CITY OF DALLAS

INTERIOR LEVEE DRAINAGE STUDY – PHASE 1

VOLUME 1 OF 2 – REPORT

September 2006

Prepared for:

The City of Dallas, Texas

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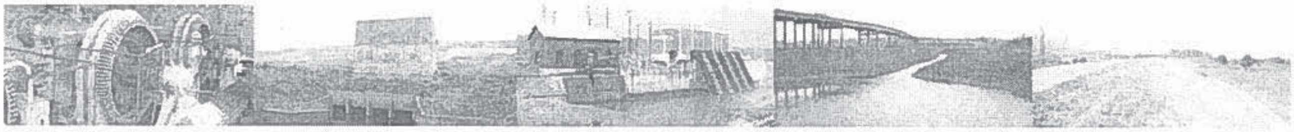

Table 4.7 - Summary of Recommendations for the East Levee Interior Drainage System

ITEM DESCRIPTION	Current Probable Cost (millions)	Escalated 5% per Year for 5 Years (millions)
multiplier =	*1.000*	*1.276*
ABLE PUMPING PLANT		
New 400,000 gpm Pump Station	$29.0	$37.0
Extend New Pump Station outfall (1,100 ft)	$3.3	$4.2
Replace Existing Culverts	$3.0	$3.8
Rehab existing Pump Stations	$6.5	$8.3
Extend existing SAX outfall (1,100 ft)	$1.1	$1.4
Extend existing LAX outfall (1,100 ft)	$1.1	$1.4
SUBTOTAL	**$44.0**	**$56.1**
BAKER PUMPING PLANT		
New 700,000 gpm Pump Station	$35.8	$45.7
Extend New Pump Station outfall (300 ft)	$0.9	$1.1
Rehab existing Pump Station (NBX only)	$0.7	$0.9
Extend existing NBX outfall (300 ft)	$0.3	$0.4
SUBTOTAL	**$37.7**	**$48.1**
HAMPTON PUMPING PLANT		
New 500,000 gpm Pump Station	$31.3	$39.9
Extend New Pump Station outfall (300 ft)	$0.9	$1.1
Rehab existing Pump Stations	$5.5	$7.0
Extend existing NHX outfall (300 ft)	$0.3	$0.4
Extend existing OHX outfall (300 ft)	$0.3	$0.4
SUBTOTAL	**$38.3**	**$48.9**
NOBLES BRANCH SUMP		
Add 3 new 60-inch gated culverts	$1.0	$1.3
SUBTOTAL	**$1.0**	**$1.3**
PRESSURE SEWERS		
Extend existing WOODALL RODGERS (1,100 ft)	$3.3	$4.2
Extend existing DALLAS BRANCH (1,100 ft)	$3.3	$4.2
Extend existing TURTLE CREEK (300 ft)	$0.9	$1.1
Extend existing BELLEVIEW (300 ft)	$0.9	$1.1
SUBTOTAL	**$8.4**	**$10.7**
TOTAL PROBABLE COST	**$129**	**$165**

Dec 2005 Baseline for Costs

for

The City of Dallas

Ms. Laura Miller
Mayor

Mr. Donald W. Hill
Mayor Pro Tem

Dr. Elba Garcia
Deputy Mayor Pro Tem

Ms. Pauline Medrano
Mr. Ed Oakley
Dr. Maxine Thornton-Reese
Mr. Steve Salazar
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Mr. Bill Blaydes
Ms. Linda Loop
Mr. Ron Natinsky
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Ms. Angela Hunt
Councilpersons

Ms. Mary K. Suhm
City Manager

Mr. David C. Dybala, P.E.
Director of Public Works and Transportation

September 2006

EXECUTIVE SUMMARY

The objective of this project was to identify and recommend upgrades to the Dallas Floodway East Levee interior drainage system so that the maximum predicted elevations in the sumps for the 100-year, 24-hour storm event do not exceed the established City of Dallas design elevations. To accomplish this objective, a comprehensive study of the East Levee interior drainage system was performed, including the development of computer models to simulate the rainfall-runoff processes of the East Levee interior drainage basins and the operation of the sumps. An HEC-HMS hydrologic computer model was developed and calibrated for the East Levee. The HEC-HMS calibration results were satisfactory for Nobles Branch, Record Crossing, and Hampton-Oak Lawn sumps. Because of the complex interaction of the individual Able Sump ponds, an XP-SWMM model was developed for Able Sump. The HEC-HMS model was used to calculate runoff hydrographs for input to the XP-SWMM model. The Able Sump XP-SWMM model calibration results were satisfactory.

For existing conditions, computer model simulations for the 100-year, 24-hour storm event predicted that all of the East Levee sumps would exceed their design elevations for that event. The calibrated computer models were used to formulate potential solutions to reduce the predicted peak sump elevations to the design elevations. Preliminary opinions of probable costs were developed for the alternates, and recommended alternates were selected in coordination with City personnel. Key results of the study were mapped using GIS, including inundation maps for existing and proposed conditions.

Summary of Recommendations

For Able Sump, alternate Able-2 is the recommended solution, consisting of the replacement of four sump pond connecting culverts along with the construction of a new 208,000 gpm pump station at Able Pumping Plant and the rehabilitation of the existing pump stations. The total probable cost for all of these improvements is approximately $44.0 million.

For Hampton-Oak Lawn Sump, alternate HOL-1 is the recommended solution, consisting of the construction of a new pump station at Baker Pumping Plant with a total capacity of 700,000 gpm. This total includes 208,000 gpm to make up for the recommended decommissioning of Old Baker Pump Station. Recommendations were also developed for the rehabilitation and modernization of the existing New Baker pump station. The total probable cost for all of these improvements is approximately $37.7 million.

For Record Crossing Sump, alternate RC-1 is the recommended solution, consisting of the construction of a new 500,000 gpm pump station at Hampton Pumping Plant. Rehabilitation and modernization of the existing Hampton pump stations is also recommended. The total probable cost for all of these improvements is approximately $38.3 million.

For Nobles Branch Sump, the construction of three additional 60-inch gated culverts at the Grauwyler Gate at Empire Central Drive is recommended (alternate NB-3). The probable cost for this imp.3rovement is approximately $1.0 million.

To avoid conflicts with the Balanced Vision Plan for the Trinity River Corridor, the outfalls of the existing East Levee pressure sewers will have to be extended. The total probable cost for these extensions is $8.4 million.

The total probable present-day cost of all recommended improvements for the East Levee interior drainage system is approximately $129 million. The probable future cost of these recommended improvements (escalated at 5% to a midpoint of construction 5 years in the future) is $165 million. These figures have been submitted to the City and were used by City personnel to develop recommendations for the City's November 2006 Bond Program.

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1. PROJECT BACKGROUND

Historically, mankind has settled near a source of fresh water – often alongside rivers. Floods would inevitably occur, and citizens would be forced to seek a means to prevent future flood damages to existing development and to allow future development to take place. Levees were often constructed to protect the community from riverine flooding. However, the levees block runoff from the interior (protected) side of the levee from reaching the river. Unless measures are taken to deal with the problem of interior drainage, the flooding on the protected side of the levee due to interior drainage may be as bad as or worse than the original riverine flooding.

Interior drainage is usually handled by allowing the stormwater runoff to pond in low areas (sumps) on the interior side of the levee. Then the water is pumped over the levee into the river, or allowed to gravity flow into the river through sluice gates in the levee. This strategy has been utilized by the City of Dallas to manage interior drainage along the Trinity River for approximately 75 years.

1.1 HISTORY OF DALLAS INTERIOR DRAINAGE

The City of Dallas was founded in the 1840's on the Trinity River, just downstream of the confluence of the West Fork and Elm Fork of the Trinity River. The Trinity River was vital to the early development of the City. However, numerous large floods, including the catastrophic flood of 1908, led the citizens of Dallas to seek protection from Trinity River floodwaters. A plan was developed to build levees along a 13-mile corridor through the City. The confluence of the Elm Fork and West Fork was relocated, and the Trinity River was channelized, creating the Dallas Floodway. Interior drainage was accommodated by a system of sumps and a number of gravity sluices, four pumping plants (two on each bank of the Floodway) and three pressure sewers. Generally, the sumps consisted of the old channels of the Elm Fork, West Fork, and Main Stem of the Trinity River, as well as borrow ditches created during levee construction. The pressure sewers are large gravity trunk lines that discharge directly into the Floodway. The inlets of the pressure sewers are located far enough upstream in the watershed to develop sufficient head to discharge against flood stages in the Floodway. The construction of the Dallas Floodway levees and associated interior drainage features was completed in 1932.

The condition of the levees had begun to deteriorate by the late 1940's, with numerous slides, cracks, and seepage failures occurring. During the period 1953-1960, the US Army Corps of Engineers (USACE) Fort Worth District reconstructed and improved the Dallas Floodway levees and pilot channel. Interior drainage was improved during the project by building a new pressure sewer, adding an additional pump station at one of the existing pumping plants, and adding two new pumping plants (one on each side of the Dallas Floodway). Interior drainage has been further enhanced since the levees were reconstructed by the construction of new pump stations at two of the pumping plants along with continuous operational improvements throughout the system.

1.2 DALLAS FLOODWAY EAST LEVEE INTERIOR DRAINAGE SYSTEM

The objective of this project was to identify and recommend upgrades to the Dallas Floodway East Levee interior drainage system so that the maximum predicted elevations in the sumps for the 100-year, 24-hour storm event do not exceed the established City of

Dallas design elevations. The East Levee protects the Stemmons Corridor and parts of downtown and the Central Business District from riverine flooding. It is estimated that the East Levee protects some $8 billion in property value from Trinity River flooding. This section presents a brief history and description of the East Levee interior drainage system. The source of much of the historical information in this section is the 2003 paper, “History of the Dallas Floodway,” by Furlong, Ajemian, and McPherson.

1.2.1 East Levee Interior Drainage Development

The original East Levee interior drainage features were installed in the early 1930’s during the construction of the Dallas Floodway and consisted of two pumping plants and two pressure sewers. Pumping Plant A (later known as Able) was built just downstream of the Houston Street Viaduct and included two 20,000 gpm pumps. Pumping Plant B (later known as Baker) was built just upstream of the current Sylvan Street crossing of the Dallas Floodway and included four 52,000 gpm pumps. Sump storage for the pump stations consisted of the old Elm Fork and Main Stem of the Trinity River channels. The Dallas Branch Pressure Sewer was constructed with an outfall located just downstream of the Continental Street Bridge, and drained most of the downtown Central Business District. The Mill Creek (later known as Belleview) Pressure Sewer outfall was located upstream of the Corinth Street Bridge and conveyed drainage from the Mill Creek watershed into the Floodway.

When the Dallas Floodway levees were reconstructed in the 1950’s, several new interior drainage features were added to the East Levee system. A new pumping plant, Hampton, was constructed just upstream of the Hampton Road Bridge and consisted of four 50,000 gpm pumps. A new pump station was constructed at Able Pumping Plant, consisting of three 46,667 gpm pumps. The newer station is known as Large Able, and the original station is known as Small Able. The Turtle Creek Pressure Sewer was constructed with an outfall near the western end of Oak Lawn Avenue, and drains the upper part of the Turtle Creek watershed. The inlet to the Turtle Creek Pressure Sewer was designed with an overflow weir such that flows in excess of the pressure sewer capacity bypass the inlet and flow down Turtle Creek into the Hampton-Oak Lawn (Baker Pumping Plant) Sump.

In 1975, the City added a new pump station at Baker Pumping Plant consisting of five 80,000 gpm pumps and one 6,000 gpm sump pump. The pump stations at Baker Pumping Plant were thenceforth named New Baker and Old Baker. Also in 1975, a new pump station was added at Hampton Pumping Plant consisting of five 80,000 gpm pumps and one 6,000 gpm sump pump, and the pump stations were accordingly named Old Hampton and New Hampton.

By the late 1970s, the capacity of the Dallas Branch Pressure Sewer had been exceeded, and a new pressure sewer, the Woodall Rodgers Pressure Sewer, was constructed. The lower part of the new pressure sewer from Woodall Rodgers Freeway to the Dallas Floodway was constructed by the City. The upper part, which runs underneath Woodall Rodgers Freeway, was constructed by TxDOT. The outfall of the Woodall Rodgers Pressure Sewer is just upstream of the Dallas Branch Pressure Sewer. The Woodall Rodgers Pressure Sewer drains much of the area previously drained by the Dallas Branch Pressure Sewer, thus relieving the burden on the Dallas Branch Pressure Sewer.

Several incremental upgrades to pumping capacity at existing pump stations have been added. In 1967, the City replaced the two 20,000 gpm pumps at Small Able pump station with two 40,000 gpm pumps. One 2,500 gpm sump pump was added at Old Hampton pump station in 1969. In 1979, the City added one 6,000 gpm sump pump to Large Able pump station. Six 10-ft x 10-ft gravity sluices were constructed at Baker Pumping Plant in the 1980s.

In addition to increasing the discharge capacity of the pumping plants, the City of Dallas has made a number of significant improvements to the East Levee interior drainage system over the years. The Dallas Floodway pilot channel was dredged from the downstream end of the Floodway to the Houston Street Viaduct. A sophisticated SCADA system incorporating closed-circuit TV cameras has been installed to control and monitor the operation of the pumping plants. Data collection has been enhanced through the installation of the ALERT system, allowing real-time measurement of precipitation and stream and sump levels throughout the watershed.

One of the primary operational problems associated with the interior levee pumping plants is dealing with the debris load in the runoff. The pump station trash racks can quickly become clogged during pumping operations, diminishing the capacity of the pump station. Over the years, the City has dealt with this problem in a number of ways, from manually scraping the bar screens with long-handled rakes to the installation of specially-designed cranes to scrape the bar screens at Old Baker and Old Hampton pump stations. More recently, the City has installed automated trash racks that periodically scrape the bar screens and lift the debris to a staging area at the top of the screen, where it can be scooped away with loading equipment. On the East Levee, these automated trash racks are installed at Large Able, New Baker, and New Hampton pump stations. Because of its location, Able Pumping Plant tends to collect the most debris of the East Levee pumping plants. Therefore, Large Able station has an automated conveyor system at the top of the trash rack which moves the debris to one side of the staging area and helps prevent large amounts of debris from accumulating at the top of the trash rack.

1.2.2 Description of East Levee Interior Drainage Features

This section identifies and briefly describes the major features of the East Levee interior drainage system and provides the framework for further discussion of the system in subsequent sections of the report. An overview of the major East Levee drainage features is shown in Exhibit 1.

1.2.2.1 Able Pumping Plant and Sump

Able Pumping Plant is located at the East Levee between the Houston Street and Jefferson Street Viaducts and consists of two separate pump stations known as Small Able (SAX) and Large Able (LAX). Photos 1.1 and 1.2 show the existing Able Pumping Plant and surrounding features.

Photo 1.1 - Able Pumping Plant, Looking Upstream Towards Able Sump

Photo 1.2 - Able Pumping Plant, Looking Downstream Towards Dallas Floodway

The pumping capacities and operational procedures of Able Pumping Plant pumps are summarized in Table 1.1.

Table 1.1 - Able Pumping Plant Properties

Small Able Pump Station (SAX) Pump floor elevation = 399.0 ft NAVD88			
Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
1	40,000	380.0	378.5
2	40,000	380.5	379.5
Large Able Pump Station (LAX) Pump floor elevation = 399.0 ft NAVD88			
Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
3	46,667	381.0	380.0
4	46,667	381.5	380.5
5	46,667	382.0	381.0
sump pump	6,000	379.0	378.0

Photos 1.3 and 1.4 show the interior of Small Able Pump Station and Large Able Pump Station, respectively.

Photo 1.3 - Small Able Pump Station Interior

Photo 1.4 - Large Able Pump Station Interior

Able Sump consists of a series of nine separate ponds divided by streets, highways, and the Belleview Pressure Sewer. The sump ponds are generally connected to each other by culverts. A detailed view of Able Sump is shown in Exhibit 2. The Belleview Pressure Sewer creates a boundary that isolates Ponds 1-5 from Ponds 6-9. A long 42-inch diameter culvert connects Ponds 4 and 7. This culvert runs under an Associated Freezers warehouse building, is approximately 1,850 ft long, and apparently requires a sag profile to pass under the Belleview Pressure Sewer. The precise alignment, profile, and condition of this pipe are unknown, and the connection does not appear on the City's storm sewer locator maps. Based on observations of Able Sump conditions after a rainfall event, this pipe is not effective in allowing Ponds 6-9 to drain towards Able Pumping Plant.

URS/Forrest and Cotton, Inc. Consulting Engineers completed an interior drainage study of the Able Pumping Plant area in November 1975. The study was conducted over a 22-month period. The primary recommendation from the study was the addition of a 6,000 gpm sump pump to reduce the number of start-up operations for the larger pumps for low flows. This pump was added to Large Able pump station in 1979.

A 2004 Master Drainage Plan Study for the Mill Creek watershed prepared by Halff Associates, Inc. found that for the 100-year, 2-hour storm event, a peak flow of 2,600 cfs and a total volume of 308 ac-ft overflowed from the Mill Creek drainage basin into Able Sump. Able Sump is undersized to handle this overflow, as was demonstrated during the May 5, 1995 and March 18-19, 2006 storm events. The Mill Creek study recommended alternatives to prevent this spillage into Able Sump; but these alternatives were not constructed at the time of this report. Further discussion of the Mill Creek overflow into Able Sump is found in Section 3.1.1.

The City of Dallas 100-year design sump elevation for Able Sump is 392.5 ft. The source of all design sump elevations listed in this report is a memo entitled “100 YR. W.S. Elevations for Sump Areas Used by City of Dallas” provided by the City of Dallas Public Works and Transportation Department. A summary table of design elevations for all East Levee sumps is found in Section 3.4.1.

1.2.2.2 Baker Pumping Plant and Sump

Baker Pumping Plant is located upstream of Sylvan Avenue, behind the City of Dallas Public Works and Transportation facility at 2255 Irving Boulevard. Baker Pumping Plant consists of two pump stations – Old Baker (OBX) and New Baker (NBX). Photos 1.5 and 1.6 show the existing Baker Pumping Plant and gravity sluices and surrounding features.

Photo 1.5 - Baker Pumping Plant and Gravity Sluices

Photo 1.6 - Baker Pumping Plant and Gravity Sluices, Looking Upstream Toward Hampton-Oak Lawn Sump

The pumping capacities and operational procedures of Baker Pumping Plant pumps are summarized in Table 1.2.

Table 1.2 - Baker Pumping Plant Properties

New Baker Pump Station (NBX) Pump floor elevation = 410.00 ft NAVD88			
Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
1	80,000	386.0	384.5
2	80,000	386.5	385.5
3	80,000	387.0	386.0
4	80,000	387.5	386.5
5	80,000	388.0	387.0
Old Baker Pump Station (OBX) Pump floor elevation = 399.88 ft NAVD88			
Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
6	52,000	390.0	389.0
7	52,000	390.5	389.5
8	52,000	391.0	390.0
9	52,000	391.5	390.5
sump pump	6,000	382.0	379.75

Photos 1.7 and 1.8 show the interior of Old Baker Pump Station and New Baker Pump Station, respectively.

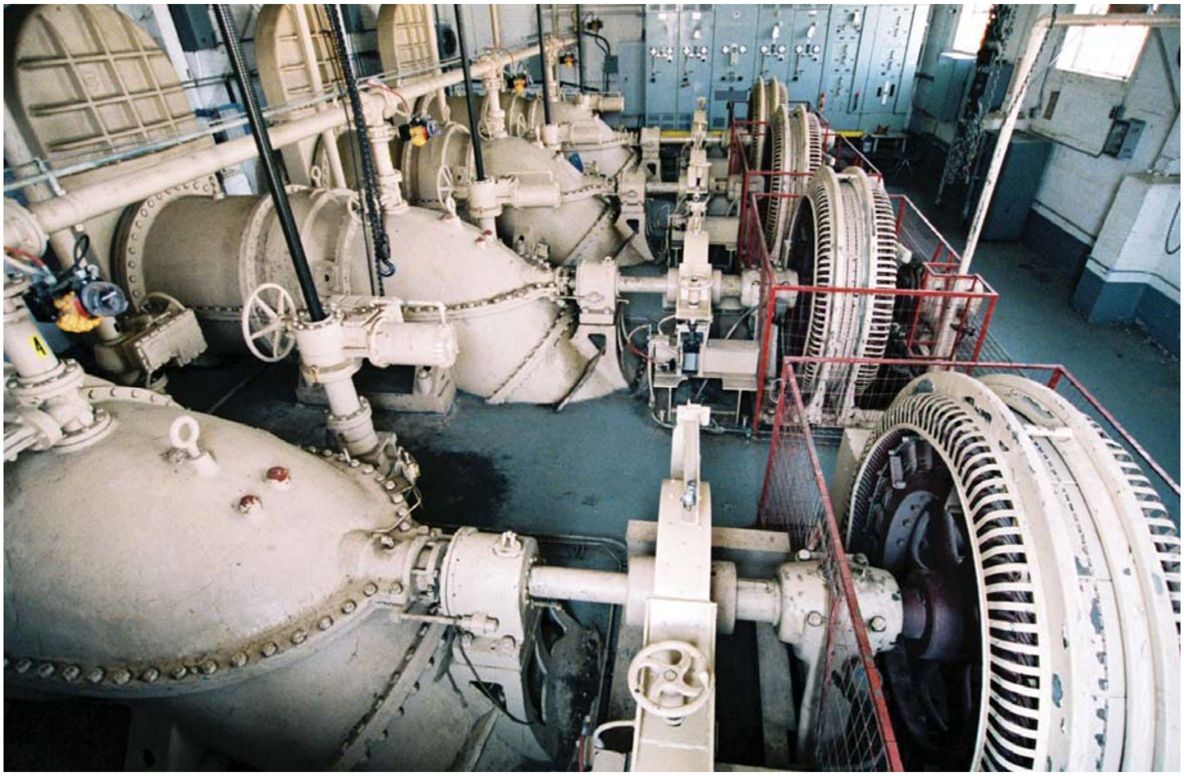

Photo 1.7 - Old Baker Pump Station Interior

Photo 1.8 - New Baker Pump Station Interior

The sump area for Baker Pumping Plant consists of the old Main Stem of the Trinity River channel and levee borrow ditches generally lying between Hampton/Inwood Road and Oak Lawn Avenue. Consequently, the Baker Pumping Plant sump is usually referred to as Hampton-Oak Lawn Sump. A detailed view of Hampton-Oak Lawn Sump is shown in Exhibit 3.

Two significant interior drainage studies have been performed for the Baker Pumping Plant area. The first was a combined study of the Baker and Hampton areas, completed by Forrest and Cotton, Inc. Consulting Engineers in 1971. The study was conducted over a period of 17 months. This study recommended the following improvements for Baker Pumping Plant and Hampton-Oak Lawn Sump:

- Construction of a levee or floodwall along the left bank of Knights Branch from the old river channel (sump) to the Rock Island Railroad embankment.
- Construction of a bridge on Irving Boulevard across the Baker Pumping Plant intake channel
- Addition of a new 400,000 gpm pump station at Baker Pumping Plant
- Addition of four 10-ft x 10-ft gravity sluices at Baker Pumping Plant

The recommendations of the 1971 study were based on proposed Dallas Floodway improvements by the Corps of Engineers which would have enlarged the Dallas Floodway pilot channel to a 200-ft bottom width trapezoidal channel. At the time of the study, it was not known when these improvements would be constructed. Consequently, the 1971 report recommended that the City consider constructing improvements to the

Dallas Floodway channel between Continental Street and the Hampton Pumping Plant. The recommendation for gravity sluices was based on the channel improvements being in place. These improvements were not constructed as envisioned, thereby negating the benefits of the gravity sluices for large flood events. The report notes that the recommended improvements would not provide protection for the 100-year storm event if the Dallas Floodway channel improvements were not constructed.

Of the 1971 report recommendations, the Irving Boulevard Bridge was constructed, and the new pump station (New Baker) at Baker Pumping Plant was constructed in 1975. The Knights Branch floodwall, Baker Pumping Plant gravity sluices, and the Dallas Floodway channel improvements were not constructed.

The second major interior drainage study for the Baker Pumping Plant area was completed in 1981 by URS Company. The study was conducted over a period of 19 months. The report recommended the construction of six 10-ft x 10-ft gravity sluices at Baker Pumping Plant and the enlargement of the Dallas Floodway channel to a 200-ft bottom width trapezoidal channel from Corinth Street to Baker Pump Station. The gravity sluices were constructed, but the channel improvements in the Dallas Floodway were never done. Thus, the benefits of gravity discharge through the sluices for larger flood events were never fully realized.

The City of Dallas 100-year design sump elevation for Hampton-Oak Lawn Sump is 402.5 ft.

1.2.2.3 Hampton Pumping Plant and Sump

Hampton Pumping Plant is located just upstream of the Hampton/Inwood Road Bridge. The pumping plant consists of two pump stations – Old Hampton (OHX) and New Hampton (NHX). Photos 1.9 and 1.10 show the existing Hampton Pumping Plant and surrounding features.

Photo 1.9 - Hampton Pumping Plant Looking Downstream Toward Dallas Floodway

Photo 1.10 - Overhead View of Hampton Pumping Plant

The pumping capacities and operational procedures of the Hampton Pumping Plant pumps are summarized in Table 1.3.

Table 1.3 - Hampton Pumping Plant Properties

Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
New Hampton Pump Station (NHX) Pump floor elevation = 410.00 ft NAVD88			
1	80,000	386.0	384.5
2	80,000	386.5	385.5
3	80,000	387.0	386.0
4	80,000	387.5	386.5
5	80,000	388.0	387.0
sump pump #1	6,000	382.0	379.75
Old Hampton Pump Station (OHX) Pump floor elevation = 410.00 ft NAVD88			
Pump No.	Capacity (gpm)	Turn-On Elevation (ft NAVD88)	Shut-Off Elevation (ft NAVD88)
6	50,000	388.5	387.5
7	50,000	389.0	388.0
8	50,000	389.5	388.5
9	50,000	390.0	389.0
sump pump #2	2,500	382.25	380.0

Photos 1.11 and 1.12 show the interiors of Old Hampton Pump Station and New Hampton Pump Station, respectively.

Photo 1.11 - Old Hampton Pump Station Interior

Photo 1.12 - New Hampton Pump Station Interior

The sump area for Hampton Pumping Plant consists of the old Elm Fork and Main Stem of the Trinity River channels between Empire Central Drive and Inwood Road and levee borrow ditches adjacent to the East Levee from the pumping plant to the Rock Island Railroad embankment. This sump area has traditionally been known as Record Crossing Sump. A gated culvert structure, Grauwyler Gate, is located in the old Elm Fork channel at Empire Central Drive and divides the lower part of the sump (Record Crossing Sump) from the upper part (Nobles Branch Sump). Nobles Branch Sump consists of the old Elm Fork Channel from Empire Central Drive to the East Levee at Stemmons Freeway, and the borrow ditch adjacent to the East Levee from Stemmons Freeway to the Rock Island Railroad. Exhibits 4 and 5 show detailed views of Record Crossing Sump and Nobles Branch Sump, respectively.

Forrest and Cotton, Inc. Consulting Engineers completed an interior drainage study of the Baker and Hampton Pumping Plant areas in 1971. The study identified the low area at the Inwood Road – DART Trinity Railway Express (TRE, formerly Chicago Rock Island and Pacific Railroad) underpass as an area of significant flooding potential and recommended the installation of a supplemental storm water pump station and a real-time flood warning system at the underpass. The report made the following primary recommendations for Hampton Pumping Plant, Record Crossing Sump, and Nobles Branch Sump:

- Enlargement of Record Crossing Sump by about 320 acre-feet
- Construction of a levee or floodwall along the left bank of Knights Branch from the old river channel (sump) to the Rock Island Railroad embankment
- Addition of a new 400,000 gpm pump station at Hampton Pumping Plant
- Addition of a single 10-ft x 10-ft gravity sluice at the Hampton Pumping Plant
- Addition of a 9,000 gpm supplemental pumping plant and related storm sewer improvements and warning system at the Inwood Road – Rock Island Railroad underpass

As previously discussed in Section 1.2.2.2, these recommendations were made with the assumption that the Dallas Floodway channel would be improved to a 200-ft bottom width trapezoidal channel from Corinth Street upstream to the Hampton Pumping Plant.

Based on the recommendations of the 1971 report, New Hampton pump station with a capacity of 400,000 gpm and a 10-ft x 10-ft gravity sluice was constructed at Hampton Pumping Plant in 1975. It is not known if the sump enlargement for Record Crossing Sump was constructed. The rest of the recommendations were apparently not implemented.

The City of Dallas 100-year design sump elevation for Record Crossing Sump is 405.0 ft. The City's 100-year design sump elevation for Nobles Branch Sump is 408.1 ft.

At the 100-year design elevation of 405.0 ft for Record Crossing Sump, water will spill out of the sump along the left bank of Knights Branch near the intersection of Irving Boulevard and Inwood Road (see Exhibit 4). The overflow will pond at the low point of Inwood Road at the TRE underpass (approximate elevation 398.0 ft). Assuming the ponded water achieved the same elevation as Record Crossing Sump, Inwood Road at the TRE underpass could be flooded up to seven feet deep for the 100-year design conditions. Besides creating a significant hazard to motorists, this ponded water would also jeopardize adjacent property.

This situation was recognized during the Forrest and Cotton, Inc. 1971 interior drainage study. The 1971 report recommended the construction of a levee or floodwall along the left bank of Knights Branch from the Record Crossing Sump channel to the TRE embankment to help curb this overflow. However, it appears that the floodwall would be ineffective in preventing overflow from Knights Branch unless Irving Boulevard and the southbound service road of Stemmons Freeway were also raised. In addition to the floodwall, the 1971 report recommended the construction of a supplemental 9,000 gpm pump station to evacuate water from Inwood Road at the TRE underpass. Also recommended was the installation of a flood warning signal light system keyed to the water level at the railroad underpass. The warning signals would have been placed at the TRE underpass and at Stemmons Freeway to the south and Harry Hines Boulevard to the north to warn motorists of high water. The report states that the ponded water should be pumped back into the Knights Branch channel, which seems unlikely to improve the situation if water cannot be prevented from spilling out of Knights Branch at Irving Boulevard and the southbound Stemmons Freeway service road.

The low area at the Inwood Road TRE underpass discharges by gravity through a 36-inch storm sewer pipe into Hampton-Oak Lawn sump just east of Inwood Road. The 1971 report recommended the addition of flap gates to the storm sewer pipe to prevent water in the sump from backing up into the low area. This would not have eliminated the need for the supplemental pump station, since the development of sufficient head to discharge against the flap gates and flood stages in Hampton-Oak Lawn Sump would still create hazardous flooding conditions at the Inwood Road TRE underpass.

Further discussion of the drainage issue at the Inwood Road/Rock Island Railroad underpass is found in Section 4.3.3.

1.2.2.4 Dallas Branch Pressure Sewer

The Dallas Branch Pressure Sewer was part of the original East Levee interior drainage system installed in the 1930s. The Dallas Branch Pressure Sewer currently drains most of downtown and the Central Business District between Woodall Rodgers Freeway and Pacific Avenue. The conduit is a hodgepodge of cross-sections and sizes. The Dallas Branch Pressure Sewer originally drained approximately 950 acres, but when the Woodall Rodgers Freeway was built in the 1970s, the Woodall Rodgers Pressure Sewer intercepted all of the Dallas Branch Pressure Sewer drainage area north of Woodall Rodgers Freeway, reducing the Dallas Branch Pressure Sewer drainage area to approximately 312 acres. The Dallas Branch Pressure outfall is located just downstream of the Continental Street Bridge. The approximate alignment of the Dallas Branch Pressure Sewer is shown on Exhibit 1.

1.2.2.5 Woodall Rodgers Pressure Sewer

The Woodall Rodgers Pressure Sewer was installed in the 1970s when the Woodall Rodgers Freeway was constructed. The upper part of the pressure sewer, which runs under Woodall Rodgers Freeway, consists of a 12-ft concrete horseshoe section and was built by TxDOT. An interior view of this portion of the pressure sewer is shown in Photo 1.13.

Photo 1.13 - Woodall Rodgers Pressure Sewer Interior at Field Street Manhole, September 2005

The lower part of the pressure sewer (from Woodall Rodgers Freeway to the Dallas Floodway) was built by the City and consists of a variety of cross-sections, including triple 96-inch RCP, double 10-ft x 9-ft box culvert, and 12-ft diameter tunnel. The Woodall Rodgers Pressure Sewer drains approximately 606 acres. The Woodall Rodgers Pressure Sewer outfall structure is located just downstream of the Continental Street Bridge and just upstream of the Dallas Branch Pressure Sewer outfall. The approximate alignment of the Woodall Rodgers Pressure Sewer is shown on Exhibit 1.

1.2.2.6 Turtle Creek Pressure Sewer

The Turtle Creek Pressure was installed in the 1950s when the Fort Worth District reconstructed the Dallas Floodway levees. The pressure sewer varies from an 18.5-ft semi-elliptical section to a 16-ft horseshoe section. The Turtle Creek Pressure Sewer intake structure was built with an adjacent spillway to allow flows in excess of the pressure sewer capacity to continue down Turtle Creek into Hampton-Oak Lawn Sump. Runoff from a portion of Central Expressway is diverted into Turtle Creek upstream of the Turtle Creek Pressure Sewer inlet. When Central Expressway was reconstructed in the 1990s, the Cole Park Detention Vault was built to provide underground storage for this diversion. The watershed upstream of the Turtle Creek Pressure Sewer inlet is approximately 5,270 acres (8.2 square miles), including approximately 1,143 acres (1.8 square miles) controlled by the Cole Park Detention Vault. The approximate alignment of the Turtle Creek Pressure Sewer is shown on Exhibit 1.

2. PROJECT METHODOLOGY

This chapter describes the techniques used to analyze the East Levee interior drainage system.

2.1 CONCEPTUAL MODELING PLAN

It was necessary to develop a comprehensive hydrologic modeling strategy capable of simulating both the surface-water rainfall/runoff process and the dynamic sump water level fluctuations associated with stormwater inflow to the sumps and outflow from the pump stations. The basic modeling concept was to compute stormwater runoff and sump water levels for selected storm events. For calibration simulations, measured rainfall data were used; for hypothetical storm event simulations, rainfall totals associated with specific storm probabilities were used. Existing conditions simulations were run to establish a baseline for comparison with proposed improvement scenarios. Then pump station capacities, sump volumes, and other parameters were varied to evaluate the effects of proposed improvements. The computer models developed for this study provide the framework for analyzing existing conditions and proposed improvements. It was essential to select modeling software flexible enough to meet these needs.

2.1.1 Computer modeling software selection

In selecting computer hydrologic modeling software for this project, some of the selection criteria included:

- Capability to simulate both rainfall/runoff and reservoir routing
- Ease of use
- Size of user base and acceptance by the engineering community
- Cost of the software and restrictiveness of license agreement.

Some of the software packages considered were HEC-1 and HEC-HMS, both developed by the US Army Corps of Engineers Hydrologic Engineering Center (HEC), and XP-SWMM, published by XP Software. Any of these software packages would have been capable of meeting the modeling requirements of this project.

HEC-HMS is part of the “next-generation” software suite developed by HEC, and is considered a functional replacement for the older HEC-1 software. The available version of HEC-HMS at the start of this project (v. 2.2.2) had some known bugs in the user interface; for this reason consideration was given to using HEC-1 for this project. Ultimately, it was decided to choose HEC-HMS over HEC-1 to take advantage of the Windows graphical interface of HEC-HMS and more importantly, to provide the capability for the models to be updated as future versions of HEC-HMS are released. No further program development will be done on HEC-1, so any models developed in HEC-1 will be stagnant. HEC-HMS Like all HEC software, HEC-HMS is in the public domain, so there is no cost to obtain the software.

XP-SWMM has roots which trace back to the development of SWMM (Storm Water Management Model) by the Environmental Protection Agency in the late 1960’s. XP-SWMM is capable of simulating hydrology, hydrodynamics, and water quality.

However, XP-SWMM was not selected as the primary modeling software for this project due to its complexity, cost, and more restrictive license agreement.

Based on these criteria, HEC-HMS was chosen as the primary modeling software for this project. This approach proved to be prescient, as a new version of HEC-HMS (v. 3.0.0) was made available by HEC in early 2006. It was possible to migrate the models developed in HEC-HMS version 2.2.2 to version 3.0.0 with little difficulty. As will be discussed in Section 3.2.3, XP-SWMM was later used to develop a model of Able Sump to simulate the complex interactions of the individual storage areas that comprise the sump.

2.1.2 Model development

The first step in the hydrologic model development was to delineate the watershed draining to the East Levee sumps and to subdivide the watershed into subbasins based on topography and the storm sewer network. Once the subbasin network was established, a detailed analysis was developed to determine hydrologic parameters for the subbasins. Topographic data were used to establish elevation-volume curves for the sumps. Pump stage turn-on/shut-off elevations were used to develop sump elevation-pump station capacity curves for the main stormwater pump stations. These were the building blocks of the hydrologic model.

The most important results of the model simulations were the predicted stage hydrographs for the sumps. The maximum predicted water surface elevations in the sumps were used to prepare flood inundation maps and were compared against design sump levels and finished floor elevations to evaluate the effectiveness of proposed alternatives in reducing flooding compared to baseline conditions.

Careful consideration was given to the hydrologic computation methods to be used to model the stormwater runoff process for this study. The primary components of a rainfall-runoff model are the loss method and the transform method. The loss method is used to compute excess precipitation and either directly or indirectly accounts for precipitation losses due to infiltration, interception, and evaporation. Alternatives considered for the loss method were the SCS (Soil Conservation Service, now known as the Natural Resources Conservation Service, or NRCS) curve number method and the initial+uniform loss rate method. The SCS curve number method was chosen because of its quantifiable approach to assigning loss parameters based on land use and soil type. The primary parameter in the SCS method is the runoff curve number – the higher the curve number, the greater the runoff potential. Each subbasin is assigned a curve number based on its hydrologic characteristics. The assignment of curve numbers for the subbasins is covered in detail in Section 2.3.2.1.

Once the excess precipitation hyetograph is computed in the model, a transform method is used to compute direct runoff from the watershed. Usually, this transform method is an empirical unit hydrograph function; however, sometimes nonlinear physically-based techniques such as the kinematic wave model are used. For this project, several methods were considered for the transform function including the SCS dimensionless unit hydrograph, the kinematic wave model, and Snyder's synthetic unit hydrograph. Snyder's synthetic unit hydrograph was chosen due to the extensive research by the USACE Fort Worth District in the estimation of Snyder unit hydrograph parameters for the Dallas-Fort Worth area. This research was based on measured streamflow data in the region collected by the USGS and others during the 1960's-1970's. Many of the gaged basins were heavily developed and fully urbanized, making this method particularly applicable to the East Levee area. This research led to the development of the NUDALLAS program by the Fort Worth District in the 1980's. Consequently, Snyder's synthetic unit hydrograph is probably one of the most commonly applied hydrologic modeling transform methods in the Dallas-Fort Worth area over the last 30 years. Further discussion of the NUDALLAS implementation of Snyder's synthetic unit hydrograph is found in Section 2.3.2.2.

2.1.3 Basic assumptions

A number of basic assumptions were inherent in the development of the hydrologic model. These assumptions are consistent with previous studies of the East Levee interior drainage basin.

First, all hydrologic parameters were derived based on existing land uses within the watershed, and proposed alternatives were evaluated based on existing conditions hydrology. This is a reasonable assumption for the near future given the very high level of development and urbanization within the East Levee watershed.

Second, it was assumed that the sumps behaved like true reservoirs; i.e., level-pool routing was applicable for the sumps. This assumption proved to be questionable for Able Sump, as discussed in Section 3.2.3, but appeared to be valid for the other sumps.

Third, no hydraulic routing was performed for storm sewer pipes or open channels conveying stormwater into the sumps. The effect of this assumption is that stormwater runoff peak attenuation and lag in the model may be less than in reality. Because much of the storm sewer network in the East Levee area predates modern design criteria and was designed for smaller storm events than those considered in this analysis (e.g., the 25-year event rather than the 100-year event), it is possible that some surface storage in the watershed created by localized ponding as a result of surcharged storm sewer systems was unaccounted for in the hydrologic model. This effect was considered in the selection of Snyder unit hydrograph parameters, as discussed in Section 2.3.2.2. A detailed analysis of the entire storm sewer network contributing to the sumps was beyond the scope of this project and would be inappropriate for a project of this scale. It was believed that the lack of routing results in a conservative prediction of sump levels due to the absence of lag and attenuation.

Fourth, pressure sewers were accounted for in the model by assuming computed pressure sewer capacities were applicable throughout an entire storm event. The pressure sewers were modeled by computing the runoff hydrographs for the pressure sewer drainage basins, then subtracting the calculated pressure sewer capacities from the runoff hydrographs. The difference between the runoff hydrograph and the pressure sewer capacity was overflow that ultimately reached the sumps.

Fifth, the rated capacities for the existing pumps at the main stormwater pumping plants were assumed to be accurate. No pump curves were available for most of the existing pumps; therefore, the most consistent approach was to model the pumps at their rated capacities. This assumption was discussed with City of Dallas Flood Control District

Manager Ron Shindoll, who suggested that a factor of 0.8 be applied to the existing pump capacity ratings to account for the age of the pumps and other unknowns. However, calibration simulations compared against measured data revealed that the use of the pumps at their full rated capacities yielded better results. Therefore, the full rated capacities of the pumps were used in all event simulations.

2.1.4 Calibration

Once the hydrologic model was assembled, the next step was to perform calibration simulations to test the validity of the modeling assumptions and to verify the model parameters. Observed precipitation, sump level, and pumping data were used in the calibration simulations. Unfortunately, since no observed flow data were available, it was necessary to calibrate against measured sump elevations. The original plan was to use the automated parameter optimization routines in HEC-HMS to refine the subbasin hydrologic parameters. This approach proved impossible since the parameter optimization method in HEC-HMS works only when calibrating against observed flow data. An attempt was made to use the sump elevation-volume curves to back-calculate inflow hydrographs for the sumps, but this approach also failed, as discussed in Section 3.2.1.

2.1.5 Hypothetical Scenario Simulations

The design criterion for most stormwater facilities in the City of Dallas is the 100-year (1% annual chance) event. Therefore, the 100-year event was the primary focus of the modeling effort for this project. Some 500-year (0.2% annual chance) event scenarios were also performed to determine the incremental difference in maximum sump elevation (flooding) between the 500-year and 100-year events. All of the hypothetical scenario simulations used a 24-hour storm duration.

The precipitation data used for the hypothetical storm scenarios came from tabulated data in the North Central Texas Council of Government (NCTCOG) iSWM (Integrated Storm Water Management) manual, September 2004 Review Draft (the latest version of the manual available at the start of this study). Precipitation data are published in the iSWM manual on a per-county basis for all the NCTCOG participating counties. The data for durations greater than 15 minutes were derived from a recent study of precipitation depth-duration frequency in Texas by the USGS. Five- and 10-minute rainfall totals were taken from the National Weather Service Hydro-35 publication. The Dallas County NCTCOG rainfall data were checked against hypothetical rainfall data from the US Weather Bureau TP-40 report and were found to agree closely. The NCTCOG precipitation data were used for this analysis because these data are more recent, are specific to the project area, and were presumably derived from a longer period of record than TP-40, since TP-40 was published in 1961. The 100-year and 500-year precipitation data used in this analysis is shown in Table 2.1.

Table 2.1 - Hypothetical Storm Precipitation Data

Duration	100-year Precipitation (in)	500-year Precipitation (in)
5 min	0.93	---
15 min	2.00	3.00
1 hr	3.86	4.86
2 hr	4.90	6.12
3 hr	5.55	6.99
6 hr	6.72	8.76
12 hr	8.04	11.04
24 hr	9.60	13.68

2.2 DATA SOURCES

For a project of this size and complexity, many different data sources were required. The goal of the data collection effort was to identify and acquire the best and most recent data available. This section lists the major data types used for this project and their sources.

2.2.1 City of Dallas Public Works and Transportation Department

Almost all of the data manipulation and mapping for the Interior Drainage Study was performed in GIS. The project sponsor, the City of Dallas Public Works and Transportation Department, was a major source of GIS data used in the project.

2.2.1.1 <u>Topographic Data</u>

Consistent topographic mapping was needed for the entire East Levee watershed. The City provided topographic data in the form of seamless digital contours. The contours were developed using an airborne Light Detection and Ranging (LiDAR) system flown in winter 2000-2001. The data collection was facilitated by NCTCOG. This was the most recent area-wide digital topographic data available for use in this project. Table 2.2 summarizes the specifications of the contour data.

Table 2.2 - 2001 LiDAR Contour Data Specifications

Sensor Type	Airborne LiDAR
Altitude of Capture	8,000 ft above mean terrain
Capture Period	November 2000-January 2001
Control Sources	Ground survey, airborne GPS and inertial measurement
DEM Point Spacing	3-5 m (9.8-16.4 ft)
DEM Point Accuracy	15-20 cm (5.9-7.8 in) vertical on clearly defined ground features
Contour Interval	2 ft
Coordinate System	Texas State Plane, North Central Zone
Horizontal Datum	NAD 83
Vertical Datum	NGVD 88
Units	US Survey Feet
DEM Format	ASCII
Contour Format	ArcInfo

The City supplied the contour data in the form of tiled ArcInfo shapefiles. Each tile was 3,000 ft x 2,000 ft, and 29 tiles were required to cover the East Levee watershed. To make the data easier to use, the individual tile shapefiles were merged into one seamless shapefile. This master contour shapefile was used for all subsequent topographic analysis and mapping.

2.2.1.2 Aerial Photography

The City provided high-resolution aerial photography of the East Levee watershed. The collection of the aerial photography was facilitated by NCTCOG. The aerial photography was an invaluable tool for a variety of project tasks, including watershed and subbasin delineation and inundation mapping. Table 2.3 summarizes the specifications of the aerial photography.

Table 2.3 - 2001 Aerial Photography Data Specifications

Sensor Type	Aerial film camera with 6-in focal length
Negative Size	9-in x 9-in
Altitude of Capture	9,000 ft above mean terrain
Capture Period	February-March 2001
Control Sources	Ground survey and airborne GPS
Scanning Resolution	15 microns
Ground Resolution	1.0 US Survey Foot
Color Type	24-bit Natural Color
Coordinate System	Texas State Plane, North Central Zone
Horizontal Datum	NAD 83
Units	US Survey Feet
Format	JPEG, TIFF, or MrSID

The City provided the aerial photography in the form of tiled jpeg files. Each tile was 3,000 ft x 2,000 ft. To make the data easier to use, the tiles were merged into a seamless image, clipped to the East Levee watershed area, and then converted to the MrSID image format. MrSID image compression saved disk space and RAM and allowed the entire image to be displayed at once on the PCs used to perform the GIS analysis.

2.2.1.3 Streets

The City provided an ArcGIS shapefile of City of Dallas street centerlines for the East Levee watershed. The shapefile was updated in April 2005. The positional accuracy of the street shapefile was undefined, but the street centerlines matched extremely well with the aerial photography. The street shapefile coverage did not include streets in Highland Park and University Park, which are part of the East Levee watershed. The street centerlines for those cities came from a different source.

2.2.1.4 Land Use

The City provided an ArcGIS shapefile and database of land uses in the East Levee watershed. The polygon shapefile contains assigned land uses for parcels in the City of Dallas. The database has the following three levels of land use detail: general, basic, and detailed. For example, the general level might classify a parcel as “commercial,” the basic level might further classify the commercial parcel as “office” or “retail,” and the detailed level might further classify the retail parcel as “restaurant.” Most parcels have both general and basic levels of classification, while relatively few parcels have the detailed level of classification. For this analysis, the general level of land use classification was sufficient.

The land use dataset included parcel polygons for the cities of Highland Park and University Park, but did not include land use classifications for those parcels. Therefore, land uses for Highland Park and University Park parcels were assigned based on aerial photography and general knowledge of the area.

2.2.1.5 Storm Sewers

The City provided an ArcGIS shapefile of storm sewer lines in the City of Dallas. The shapefile is a representation of the storm sewer lines from the storm sewer locator maps and estimation based on the locations of inlets, manholes, and outfalls. The database contains the material type and size of the pipes, as well as data such as the condition of the pipe at the time of the last inspection. However, since the storm sewer inventory was not complete, this dataset was not used for the project except for general visualization of the storm sewer system.

2.2.2 GIS Data From Other Sources

To supplement GIS data supplied by the City of Dallas Public Works and Transportation Department, a number of additional GIS data sources were used.

2.2.2.1 North Central Texas Council of Governments

NCTCOG maintains a regional clearinghouse of GIS data for the Dallas-Fort Worth area. As mentioned previously, the origin of some of the GIS data provided by the City was NCTCOG. Some additional datasets used for this project were obtained directly from NCTCOG.

2.2.2.1.1 1991 Topography and Planimetrics

The 1991 Topography and Planimetrics dataset consists of 2-ft contours and planimetrics derived from conventional aerial photography. The original source of this dataset was the USACE Fort Worth District, but it is available for purchase from NCTCOG. The dataset does not cover the entire East Levee watershed, but does include the lower part of the watershed and provides complete coverage of the sump areas.

The file format for the 1991 data is tiled Microstation drawing files. Although the plan sheets show the proper coordinates when plotted, the drawing features are not georeferenced and cannot be moved to their proper coordinates in Microstation due to a limitation in the coordinate values allowed in the software. Therefore, the drawing features were converted to AutoCAD format and a coordinate offset was applied to georeference them to the coordinate system used for the GIS analysis (State Plane feet, North Central Texas, NAD83). Because a simple coordinate transform rather than a proper map projection was used to relocate the drawing features, it is possible that some horizontal displacement error was introduced in this process. However, the size of the project area is small enough that errors were probably minimal, and a visual comparison of the relocated drawing features to aerial photography and the 2001 topographic data revealed excellent agreement. The converted and relocated AutoCAD files were then used in ArcGIS without further manipulation.

Because the 1991 dataset did not cover the entire East Levee watershed, it was not suitable for watershed and subbasin delineation. However, careful consideration was

given to deciding which of the two available topographic datasets to use for the sump elevation-volume curves. Visual comparison of the 2001 and 1991 topographic data showed that the two sets of contours were quite similar, despite the fact that the two datasets are referenced to different vertical datums. The 1991 dataset is referenced to NGVD29 whereas the 2001 dataset is referenced to NAVD88. Spot checks of the elevation differences for these two vertical datums at coordinates spread over the East Levee watershed area were made using the Corpscon program, and it was found that the maximum elevation difference in the area that could be attributed to the difference in vertical datums was less than 0.1 ft.

Because the 1991 dataset contains planimetric data, the contours are not seamless. This would have complicated the use of automated volumetric calculations to develop the sump elevation-volume curves, and would have required a significant amount of hand editing of the contours and drawing cleanup. Because the datum differences and visually observed differences between the two datasets were small enough to be insignificant, it was decided to use the 2001 topographic data exclusively for this project. Further discussion of the comparison between the 1991 and 2001 topographic datasets is found in Section 2.3.3.

2.2.2.1.2 Other GIS Data

Other NCTCOG GIS datasets used for this project included road centerlines, railroads, city limits, and streams. All of these datasets were in the form of ArcInfo shapefiles, and were used only for mapping. For example, the NCTCOG road centerline dataset was used for street centerlines in University Park and Highland Park.

2.2.2.2 <u>Natural Resources Conservation Service</u>

The NRCS Soil Survey Geographic (SSURGO) database was the source of all soils data used for this project. The SSURGO database used was TX113, Dallas County, Version 2, dated December 29, 2004. The dataset consists of an ArcGIS shapefile (the soil survey map) and a database file that contains a large number of soil property tables. The soil survey map and the soil property tables were used to develop hydrologic parameters for the East Levee subbasins, as described in Section 2.3.2.

2.2.2.3 <u>US Army Corps of Engineers, Fort Worth District</u>

The Fort Worth District provided an HEC-FDA economic flood damage model of the Dallas Floodway. Accompanying this model was a GIS dataset that consisted of building footprints for structures in the lower part of the East Levee watershed along with a database containing pertinent economic and physical data for each structure. For example, the database contains data such as the structure's street address, market value, ground stage, and foundation height. The database classified structures into the following four categories based on their use, structural characteristics, and market value: residential structures, non-residential structures, unique structures, and tunnel (underground) structures. Unique structures are high market value non-residential structures.

For this project, the most important data in the database were the ground stage and foundation height for above-ground structures. According to the GIS metadata, the ground stages were determined by assigning elevations to the building footprint centroids using a TIN created from 2-ft contours. The contour dataset was unspecified, but the metadata indicated the source was the Fort Worth District. The foundation heights were estimated based on visual estimation of average slab heights in the field – residential structures were assigned a foundation height of 1.0 ft, and all other above-ground structures were assigned a foundation height of 1.6 ft.

The ground stage and foundation height were summed to compute an estimated finished floor elevation for each structure, except in the case of some unique structures, which had surveyed finished floor elevations in the database. The estimated finished floor elevations were then used to screen structures for finished floor elevation surveys.

2.2.3 City of Dallas Flood Control District

The City of Dallas Flood Control District provided scans of available plans for East Levee pump stations and miscellaneous structures such as pressure sewers and gravity sluices. Unfortunately, plans for most of the pump stations were unavailable, but partial plans were provided for New Hampton and New Baker pump stations.

The Flood Control District also supplied calibration data for use in this project. Flood Control personnel selected a number of significant flood events, and provided time series rainfall data for selected gages in the City's ALERT system, time series sump elevations, and pump operation records (number of pumps operating at each station) for these events. Further discussion of calibration data and the calibration process is found in Section 3.2.

2.2.4 US Army Corps of Engineers, Fort Worth District

The Fort Worth District provided an HEC-RAS model of the Dallas Floodway. The HEC-RAS project contained both existing conditions and baseline conditions geometries. Baseline conditions is an intermediate state between existing and future conditions (with all proposed Trinity Park features in place) and represents the floodway at a point during construction of the Trinity Park features. The HEC-RAS project was used to compute outfall elevations for East Levee pump stations, pressure sewers, and gravity sluices, as described in Section 2.3.4.

2.2.5 City of Dallas Public Works and Transportation Vault

Storm sewer system locator maps and plans for East Levee pressure sewers were obtained from the City of Dallas Public Works and Transportation Vault. The storm sewer locator maps and pressure sewer plans were used to refine the watershed and subbasin delineations, as described in Section 2.3.1. The pressure sewer plans were used to develop hydraulic analyses of the pressure sewers.

2.2.6 Field Surveys

Field surveys were performed to obtain sump cross sections and structure finished floor elevations. The sump cross sections are discussed in Section 2.3.3.

The GIS database from the USACE Fort Worth District economic model of the Dallas Floodway described in Section 2.2.2.3 was used to screen structures with estimated finished floor elevations less than the City of Dallas 100-year design sump elevations. A GIS map and database were created of the structures potentially threatened by flooding at the design sump elevations. From the database, approximately 60 structures were selected for finished floor elevation surveys. These structures were surveyed in November 2005.

One of the purposes of the finished floor surveys was to develop a comparison between surveyed finished floor elevations and estimated finished floor elevations from the Fort Worth District economic model. Because a limited number of structures could be surveyed, it seemed logical to select only structures with estimated finished floor elevations less than their local sump design elevations. In this way, the survey data could either confirm or eliminate the potential for flooding at the sump design elevations for these structures.

Later, finished floor elevations were surveyed at additional structures in response to the flood event of March 18-19, 2006. The data for all of the finished floor elevations were combined into one analysis, as discussed in Section 3.3.5.

2.3 HYDROLOGIC ANALYSIS

2.3.1 Watershed/subbasin delineation

The entire watershed draining to the East Levee sumps was delineated by hand using the 2001 LiDAR 2-ft contour data and aerial photography provided by the City. The watershed delineation was refined based on the City's storm sewer locator maps and pressure sewer plans obtained from the Public Works and Transportation Vault. Although many of the locator maps are old and difficult to read, they proved to be invaluable for accurate watershed delineation, particularly in ambiguous or heavily developed areas where natural drainage patterns have been altered significantly (e.g., the Central Business District).

Once the total watershed area had been established, the watershed was first subdivided into the areas draining to the individual sumps and to the pressure sewers. Again, the storm sewer locator maps and pressure sewer plans were used to refine this delineation. Another useful resource for the subbasin delineation was the 1975 Central Business District Drainage Study report by Raymond L. Goodson, Jr. Inc. Consulting Engineers.

Finally, the individual sump watersheds were subdivided into the final subbasin network based on topography and the layout of the storm sewer system. Logical starting points for the subbasin delineation included the subbasins drained by major creeks or storm sewer trunk lines. In many cases, these areas were then further subdivided as appropriate. At this level of detail, heavy emphasis was placed on the storm sewer locator maps and pressure sewer plans for subbasin delineation guidance.

The watersheds and subbasins were delineated on paper. They were then digitized into a GIS shapefile for use in subsequent GIS analyses. Exhibit 6 shows the subbasins developed for the East Levee watershed. Cross-hatched subbasins in Exhibit 6 are drained by pressure sewers.

Considerable difficulty was encountered in delineating the subbasins drained by the Woodall Rodgers Pressure Sewer and the Dallas Branch Pressure Sewer. These areas consist mostly of parts of Uptown, Downtown, and the Central Business District – parts

of the city where the natural drainage patterns and topography have been altered significantly. Furthermore, the storm sewer locator maps were of less help in these areas, since it appeared that most of the maps had been developed before or during the planning and design of the Woodall Rodgers Pressure Sewer. Therefore, it was difficult to ascertain the area drained by the Woodall Rodgers Pressure Sewer. Finally, partial plans for the Woodall Rodgers Pressure Sewer were obtained from the City of Dallas Public Works and Transportation Vault. These plans, along with the 1975 Drainage Study of the Central Business District by Raymond L. Goodson, Jr. Inc. Consulting Engineers, helped to solve the mystery of the Woodall Rodgers Pressure Sewer drainage basin. It is believed that the drainage areas developed for this report are as accurate as possible without significant field work.

The Woodall Rodgers Pressure Sewer plans contained some plan and profile sheets which seemed to indicate that the Woodall Rodgers and Dallas Branch pressure sewers were interconnected at Field Street. Because the plan drawings were somewhat ambiguous, and because it was important to know whether the two pressure sewers were connected, City of Dallas Street Services and Carter & Burgess, Inc. personnel entered the Woodall Rodgers Pressure Sewer at the Field Street manhole to investigate the potential connection. No connection was found at Field Street or for several hundred yards upstream or downstream of the manhole. This helped to solidify the delineations of the Woodall Rodgers and Dallas Branch Pressure Sewer drainage areas.

Table 2.4 summarizes the drainage areas for the East Levee sumps and pressure sewers. The Turtle Creek Pressure Sewer drainage area listed in Table 2.4 includes the drainage area controlled by the Cole Park Detention Vault.

Table 2.4 - East Levee Interior Drainage Feature Watersheds

Sump / Pressure Sewer	Drainage Area (square miles)
Nobles Branch Sump	2.84
Record Crossing Sump	9.45
Hampton-Oak Lawn Sump	5.45
Able Sump	2.89
Turtle Creek Pressure Sewer	8.24
Woodall Rodgers Pressure Sewer	0.95
Dallas Branch Pressure Sewer	0.49
Total	30.31

After the subbasins were delineated, the longest flow path from the upstream divide to the subbasin outlet was determined. In some cases, the drainage flow path followed generally natural drainage patterns (e.g., upper Turtle Creek). In most other cases, the subbasins were heavily urbanized with only remnants or no trace at all of the natural drainage remaining. For these subbasins, the storm sewer locator maps were used to help define the flow paths.

2.3.2 Hydrologic Parameter Development

GIS data analysis was used to develop the hydrologic parameters for the subbasins. The use of GIS allowed the calculation of some parameters to be automated to some extent.

All parameter values determined in GIS were checked for reasonableness before use in the model.

2.3.2.1 SCS Curve Numbers

The SCS runoff curve number for a watershed is primarily dependent upon land use, soil type, and antecedent moisture conditions. For this analysis, antecedent moisture condition (AMC) II (normal soil moisture conditions) was used. GIS spatial analysis was used to calculate the composite SCS curve numbers for the subbasins. The section provides a brief description of the process.

The land use GIS data provided by the City were used to establish the primary land uses in the East Levee watershed. There are three levels of land uses within the dataset; for this analysis, the “top level” (most general) was appropriate. The land uses considered in this analysis are listed in Table 2.5 and shown graphically on Exhibit 7.

The other major variable that affects the runoff curve number is the hydrologic soil type. The NRCS classifies soils into one of four groups depending on their runoff potential as follows:

- Group A: deep sand, deep loess, aggregated silts – lowest runoff potential
- Group B: shallow loess, sandy loam
- Group C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay
- Group D: soils that swell significantly when wet, heavy plastic clays, and certain saline soils – highest runoff potential

A matrix of reference SCS curve numbers relating each land use and each soil type was created using data from NRCS publication TR-55, *Urban Hydrology for Small Watersheds*. The reference curve number matrix is tabulated in Table 2.5.

Table 2.5 - Reference Curve Number Matrix

Land Use	SCS CN by Hydrologic Soil Type				Comment
	A	B	C	D	
Commercial	89	92	94	95	Commercial/business/industrial land use; average 85% impervious
Infrastructure	83	89	92	94	DART rails, utility easements, electrical substations, etc.; average 75% impervious
Institution	77	85	90	92	Includes schools, hospitals, churches, etc.; average 65% impervious
Multi-family Residential	83	89	92	94	Average 75% impervious
Protected Open Space	44	65	77	82	Open space, average of fair and good condition CNs
Park/ Recreation	39	61	74	80	Open space, good condition (grass cover > 75%)
Streets/Roads	95	96	97	98	Paved roads with curbs and storm drains - area includes ROW, average 95% impervious
Single-family Residential	61	75	83	87	1/4 acre lots, average 38% impervious
Vacant	68	79	86	89	Open space, poor condition (grass cover <50%)
Love Field	74	83	88	91	Average 60% connected impervious area

The NRCS Soil Survey Geographic (SSURGO) dataset for Dallas County was the source of all soils data used in this analysis. The spatial component of the SSURGO dataset defines the geographic extent of soil map units. A map unit is defined as "a collection of areas defined and named in terms of their soil components or miscellaneous areas or both." Map units are typically depicted on NRCS soil survey maps by polygons, and each map unit is assigned a unique number or map symbol to differentiate it from surrounding map units.

The tabular database component of SSURGO contains many different soil properties both for the soil map units and the individual soil components which form the map units. In general, each soil component has an associated hydrologic soil type in the tabular database. Some soil components do not have a hydrologic soil type; e.g., the "Urban Land" component, which makes up a large portion of the East Levee watershed, does not have an associated hydrologic soil type. For soil components without a hydrologic soil type, the worst case for runoff potential was assumed, and hydrologic soil type D was assigned to those components.

Some soil map units are composed of more than one soil component, with each component potentially having a different hydrologic soil group. The SSURGO database lists the percent composition of each component in the map unit. In these cases, the predominant hydrologic soil type based on the percentages of the individual components was assigned to the entire map unit.

These rules were used to assign a hydrologic soil type to each map unit. The SSURGO spatial dataset was modified to include these hydrologic soil types as part of its feature attribute table. The SSURGO spatial dataset is shown on Exhibit 8.

In ArcGIS, the subbasin shapefile, the land use shapefile with associated reference SCS curve numbers, and the soil map unit shapefile with associated hydrologic soil types were overlaid and merged. GIS spatial analysis was used to compute a composite curve number for each subbasin based on the area-weighted percentages of both individual land uses and individual soil map units within each subbasin. The computed curve numbers are listed in Table 2.6.

In HEC-HMS, the implementation of the SCS method requires the following parameters to be specified for each subbasin: the curve number, the initial abstraction, and the percent of impervious cover in the subbasin. No excess precipitation (runoff) occurs until the initial abstraction has been satisfied. The initial abstraction is a function of the curve number, and is computed by the following equation:

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right) \qquad \textbf{Equation 1}$$

where: I_a = initial abstraction in inches
CN = SCS curve number

In HEC-HMS, the percent impervious value is used only if the definition of the curve number for the subbasin does not account for impervious area. Since the curve number calculations for this analysis included impervious area, the percent impervious field in the HEC-HMS input was zero for all subbasins.

2.3.2.2 Snyder's synthetic unit hydrograph parameters

The two parameters required for the implementation of Snyder's synthetic unit hydrograph in HEC-HMS are the Snyder standard lag (t_p), or the difference in time between the centroid of the unit excess rainfall hyetograph and the unit hydrograph peak, and the Snyder peaking coefficient (C_p), a coefficient which is a regional watershed characteristic.

The Fort Worth District has collected a large volume of research on the determination of Snyder hydrograph parameters in the North Central Texas region. The basic formulation of Snyder's synthetic unit hydrograph as implemented in the Fort Worth District's NUDALLAS program was used for this analysis.

2.3.2.2.1 Snyder Lag time, t_p

The method used to determine t_p for a subbasin involves the use of urbanization curves developed for clay and sandy soils in the Dallas-Fort Worth area. The urbanization curves relate t_p to the function $\frac{L \times L_{ca}}{\sqrt{S_{st}}}$, where L is the distance in miles along the flow path from the upstream divide to the watershed outlet, L_{ca} is the distance in miles along the flow path from the centroid of the watershed to the watershed outlet, and S_{st} is the weighted slope of the flow path in feet per mile. Curves are developed for estimates of urbanization ranging from zero to 100 percent. For a given soil type, the curves plotted on a log-log scale take the form of a series of parallel lines for each urbanization level. For the Dallas-Fort Worth area, the two sets of urbanization curves used are the

Blackland Prairie (clay soil) curves and the East-West Cross Timbers (sandy soil) curves.

The equation described by the urbanization curves is

$$t_p = I_p \left(\frac{L \times L_{ca}}{\sqrt{S_{st}}} \right)^{0.3833} \times 10^{-BW\left(\frac{\%URB}{100}\right)} \qquad \textbf{Equation 2}$$

where: t_p = the lag time in hours from the centroid of unit excess rainfall to the peak of the unit hydrograph

L = the distance in miles along the flow path from the upstream divide to the watershed outlet

L_{ca} = the distance in miles along the flow path from the centroid of the watershed to the watershed outlet

S_{st} = the weighted slope of the flow path in ft/mi

%URB = percent value of the degree of urbanization of the watershed

BW = the bandwidth, or the log of the width between each 20% urbanization line on the plot

I_p = the calibration point, defined as t_p where $\frac{L \times L_{ca}}{\sqrt{S_{st}}} = 1$ and *%URB*=0

For the Dallas-Fort Worth urbanization curves, *BW* is 0.266 and I_p is 0.94 for clay and 1.76 for sand. The percent urbanization for all subbasins in the East Levee watershed was taken to be 100%.

The equation for the weighted flow path slope is

$$S_{st} = \left(\frac{el_{85\%} - el_{10\%}}{0.75L} \right) \qquad \textbf{Equation 3}$$

where: S_{st} = weighted slope of the flow path in ft/mi

$el_{85\%}$ = the elevation in feet at the point 85% of the flow path length (L) upstream from the outlet

$el_{10\%}$ = the elevation in feet at the point 10% of the flow path length (L) upstream from the outlet

L = the distance in miles along the flow path from the upstream divide to the watershed outlet

The weighted stream slopes were computed for each subbasin in the East Levee watershed using this equation. The 2001 LiDAR 2-ft contour data were used to establish the flow path elevation at the 10% and 85% points. For fully urbanized subbasins where most or all of the flow path consists of storm sewer conduits, the ground elevations at the 10% and 85% points were used – the presumption being that the average storm sewer pipe slope would be similar to the average ground slope along the flow path.

The equation for t_p yields values for soils that are purely sand or purely clay, depending on what value of I_p is used in the equation. However, most soils fall between these extremes. The approach for these cases is to compute a weighted average of the sand and clay values based on the relative percentages of the two soil types as follows:

$$t_{p,weighted} = t_{p,sand}(\%sand) + t_{p,clay}(\%clay) \qquad \textbf{Equation 4}$$

The SSURGO database for Dallas County lists representative percentages of sand, silt, and clay for most soil components. In order to condense these percentages from three categories (sand-silt-clay) to two categories (sand-clay), the silt category was distributed between sand and clay by assuming that "silt" is 67% clay and 33% sand.

Some soil components in the SSURGO database do not have representative percentages of sand, silt, and clay listed. For these cases, the soil component was assumed to be 100% clay to be conservative.

Because some soil map units consist of more than one soil component, a weighted average sand-clay percentage was computed for the map unit based on the individual soil components that make up the map unit. Once the sand and clay percentages were calculated for every soil map unit, GIS spatial analysis was used to compute a weighted sand-clay percentage for each subbasin based on the map units that make up the subbasin. Then the weighted Snyder lag time (t_p) was computed for each subbasin using the above equations. The computed weighted Snyder lag times are listed in Table 2.6.

2.3.2.2.2 Snyder Peaking Coefficient, C_p

The Snyder peaking coefficient is usually taken to be a regional value. The general interpretation of this parameter is that it is related to the storage capacity of the watershed. In these terms, the higher the value of the peaking coefficient, the less storage in the watershed. The relationship between q_p, the peak discharge per unit of drainage area, t_p, and C_p is given by the following equation:

$$q_p = \frac{640C_p}{t_p} \qquad \textbf{Equation 5}$$

where: q_p = peak discharge per unit of drainage area in cfs/mi^2
C_p = Snyder peaking coefficient
t_p = Snyder lag time in hours

Because $640C_p$ appears in the above equation, many hydrologists and engineers think in terms of the value of $640C_p$ rather than the value of C_p itself.

For many years, the generally accepted value for $640C_p$ in the Dallas-Fort Worth area has been 460, yielding a C_p value of 0.719. This value is a result of the research conducted by the Fort Worth District using stream gage data in the Dallas-Fort Worth area. Most of the watersheds used to develop this value were larger than 10 square miles. However, a research study by Steven Veal on some smaller urban basins in the region (less than 10 square miles) indicated that a $640C_p$ value of 370 would be more appropriate for these smaller basins, resulting in a C_p value of 0.578. Veal concluded that the lower value of C_p for heavily urbanized areas might indicate that unintended storage was occurring in the watershed due to clogged or undersized bridges, culverts, or storm drainage systems. Because all of the subbasins developed for this analysis are significantly smaller than 10 square miles, a C_p value of 0.578 was used for this project.

Table 2.6 - East Levee Subbasin Hydrologic Parameters

Basin	Subbasin	Area (mi^2)	SCS CN	L (mi)	L_{ca} (mi)	S_{st} (ft/mi)	% sand	% clay	t_p (hr)	C_p
Nobles Branch Sump										
	Nobles Branch	1.46	88.69	2.86	1.27	28	16	84	0.502	0.578
	Nobles Branch Sump Local	1.38	89.59	0.98	0.78	33.9	18	82	0.271	0.578
Record Crossing Sump										
	Knights Branch	4.8	91.5	4.24	1.94	36.5	8	92	0.615	0.578
	Record Crossing Sump Local #1	1.29	95.4	2.21	1.38	6.02	15	85	0.624	0.578
	Record Crossing Sump Local #2	0.78	94.09	0.83	0.26	81.84	7	93	0.128	0.578
	Record Crossing Sump Local #3	2.58	92	1.84	0.76	2.89	20	80	0.554	0.578
Hampton-Oak Lawn Sump										
	Wycliff	0.8	92.5	2.06	0.99	55.71	23	77	0.372	0.578
	Cedar Springs	1.91	90.3	3.59	1.82	48.59	17	83	0.572	0.578
	Hampton-Oak Lawn Sump Local	2.74	93.3	1.21	0.84	49.59	15	85	0.274	0.578
Able Sump										
	Industrial	1.26	94.59	1.95	0.66	52.8	10	90	0.285	0.578
	Town Branch	0.81	93.93	1.49	0.75	40.37	8	92	0.28	0.578
	R.L. Thornton	0.2	97.3	1.78	0.92	17.21	4	96	0.369	0.578
	Able Sump Local	0.62	92	0.52	0.28	110.04	10	90	0.107	0.578
Turtle Creek Pressure Sewer										
	Upper Turtle Creek	6.45	88.19	6.6	3.27	28.69	18	82	1.001	0.578
	Cole Park Diversion	1.79	92.19	3.19	1.57	33.06	9	91	0.522	0.578
Woodall Rodgers Pressure Sewer										
	Woodall Rodgers	0.95	93.69	2.61	1.46	32.18	30	70	0.549	0.578
Dallas Branch Pressure Sewer										
	Dallas Branch	0.33	95.59	1.36	0.75	41.03	1	99	0.254	0.578
	Pacific	0.16	95.69	1.11	0.64	39.54	0	100	0.221	0.578

2.3.3 Sump Elevation-Volume Curves

Because the most important model outputs for a given modeling scenario are the sump water surface elevation hydrographs, accurate sump elevation-volume curves were a critical component of the hydrologic model. Therefore, the final elevation-volume curves were carefully compared with the curves used for previous studies. Furthermore, sump cross-sections cut from two different sets of topographic data were compared with field survey data to select the best available topographic data to use to develop these curves.

The two sets of topographic data compared were the 2001 2-ft LiDAR contours (provided by the City, sourced from NCTCOG) and the 1991 2-ft contours from conventional aerial photography (also from NCTCOG). The two topographic datasets were compared by evaluating cross sections cut from both datasets with each other and with surveyed cross sections at the same locations. The cross sections were spaced throughout the sumps to provide a complete overview of the differences in the data over the entire sump area.

Figures 2.1-2.4 show two representative cross section plots for each sump. The cross section locations are shown on Exhibit 9.

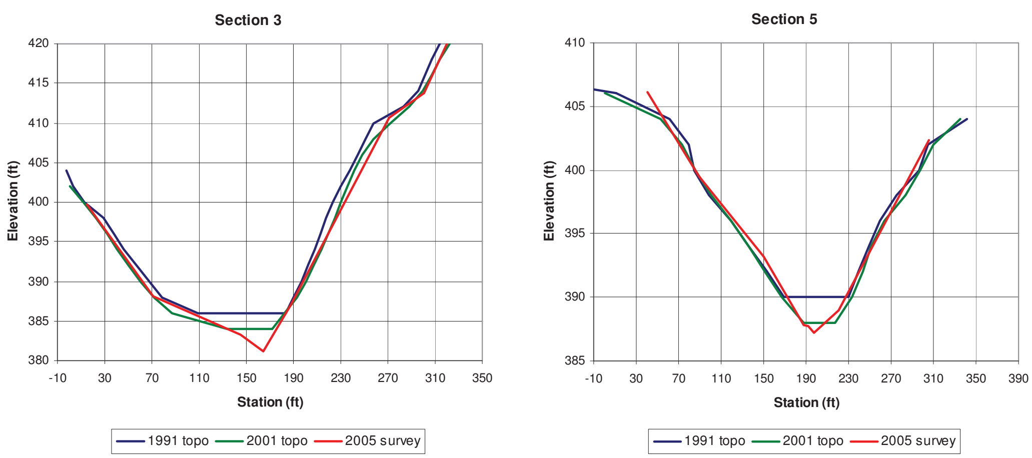

Figure 2.1 - Representative Record Crossing Sump Cross Sections

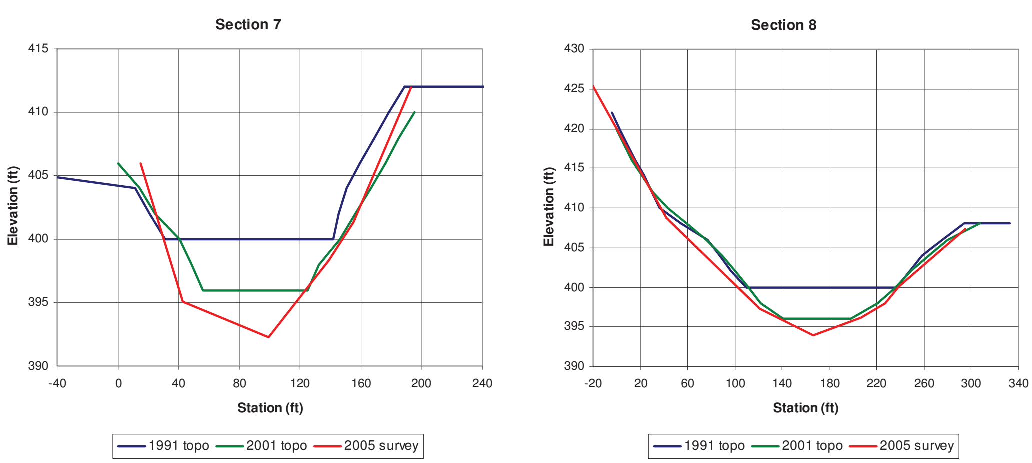

Figure 2.2 - Representative Nobles Branch Sump Cross-Sections

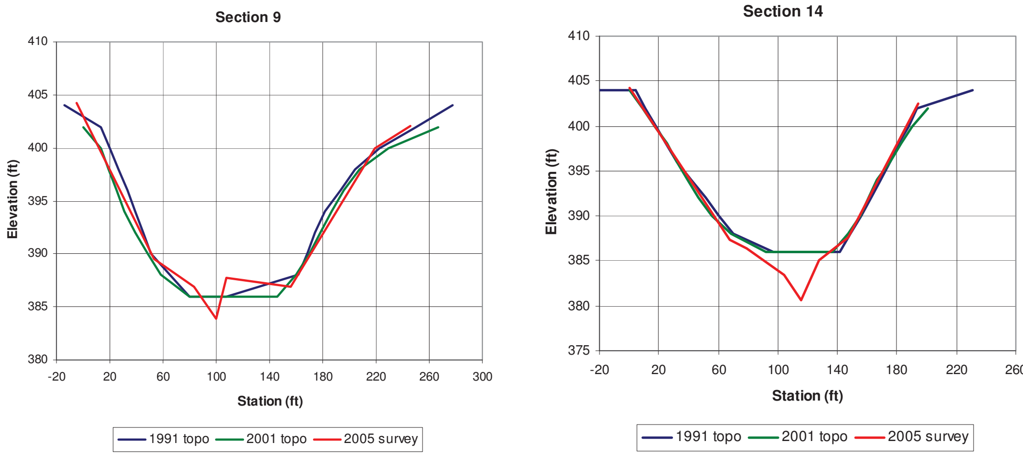

Figure 2.3 - Representative Hampton-Oak Lawn Sump Cross Sections

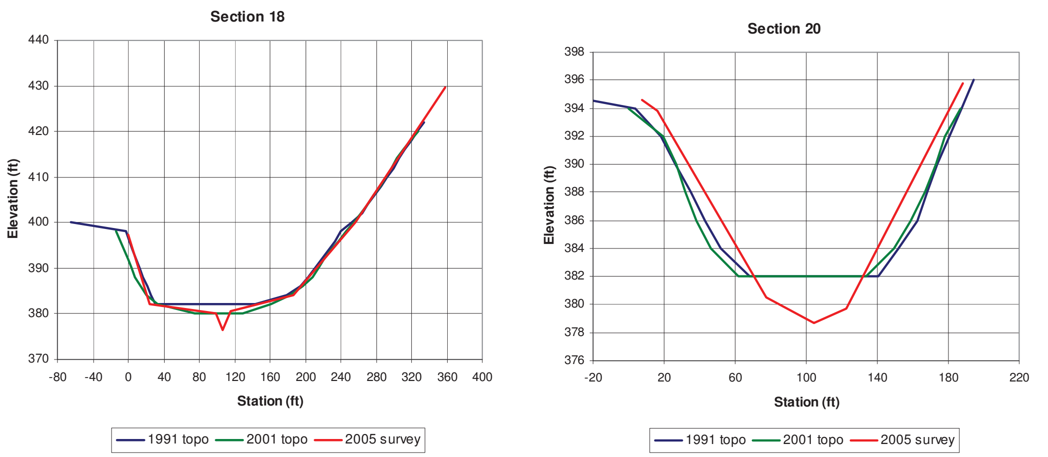

Figure 2.4 - Representative Able Sump Cross-Sections

Figures 2.1-2.4 reveal that, in general, the two topographic data sources show a flat bottom for the sumps at a higher elevation than the cross section surveys. This is because of the presence of water in the sumps during the time of the aerial surveys that were used to develop the topographic data. Neither conventional aerial photography nor LiDAR is capable of penetrating water.

Visual comparison of the topographic datasets revealed relatively minor differences between the datasets. As discussed in Section 2.2.2.1.1, the differences between the topographic datasets cannot be attributed entirely to the different vertical datums used for the datasets. Some of the differences between the datasets are caused by changes in the topography that occurred over the ten-year interval between the two surveys. The majority of the differences are likely caused by the different methods used to prepare the datasets (conventional aerial photography used for the 1991 data; LiDAR imaging used for the 2001 data). However, in no cases were the differences deemed great enough to significantly affect the results of this analysis. Therefore, the 2001 topographic data was chosen for exclusive use for this project because it was developed more recently, is based on the more modern NAVD88 vertical datum, and its seamless contours make it more amenable to GIS analysis methods.

The sump elevation-volume curves were developed by isolating the sump areas in AutoCAD Land Development Desktop (LDD). For this process, minor contour edits were done to ensure that disconnected low areas outside the sumps were filled so as to prevent their storage from being added to the sump storage. Then, a TIN surface of the edited contours was created. LDD was used to compute cut/fill volumes at each contour elevation starting at the minimum elevation in each sump. The computed fill volume for each elevation was the sump storage volume at that elevation.

Figures 2.5 – 2.8 compare the sump elevation-volume curves developed from the 2001 LiDAR 2-ft contours with elevation-volume curves from previous interior drainage studies. These plots show generally excellent agreement between the old and new data. For Hampton-Oak Lawn Sump (Figure 2.6), it can be seen that for elevations above the City of Dallas design elevation of 402.5, the new elevation-volume curve indicates more available storage than the 1971 curve.

For Nobles Branch Sump (Figure 2.8), the curves indicate that less storage is available for 2001 conditions than 1971 conditions over the entire range of elevations. It is possible that this loss of storage is due to the buildup of silt in Nobles Branch sump over the 30 years between the datasets. The magnitude of the differences between the curves is about the same over the range of elevations – if the differences were primarily due to siltation, it could reasonably be expected that the difference would be greater at lower elevations.

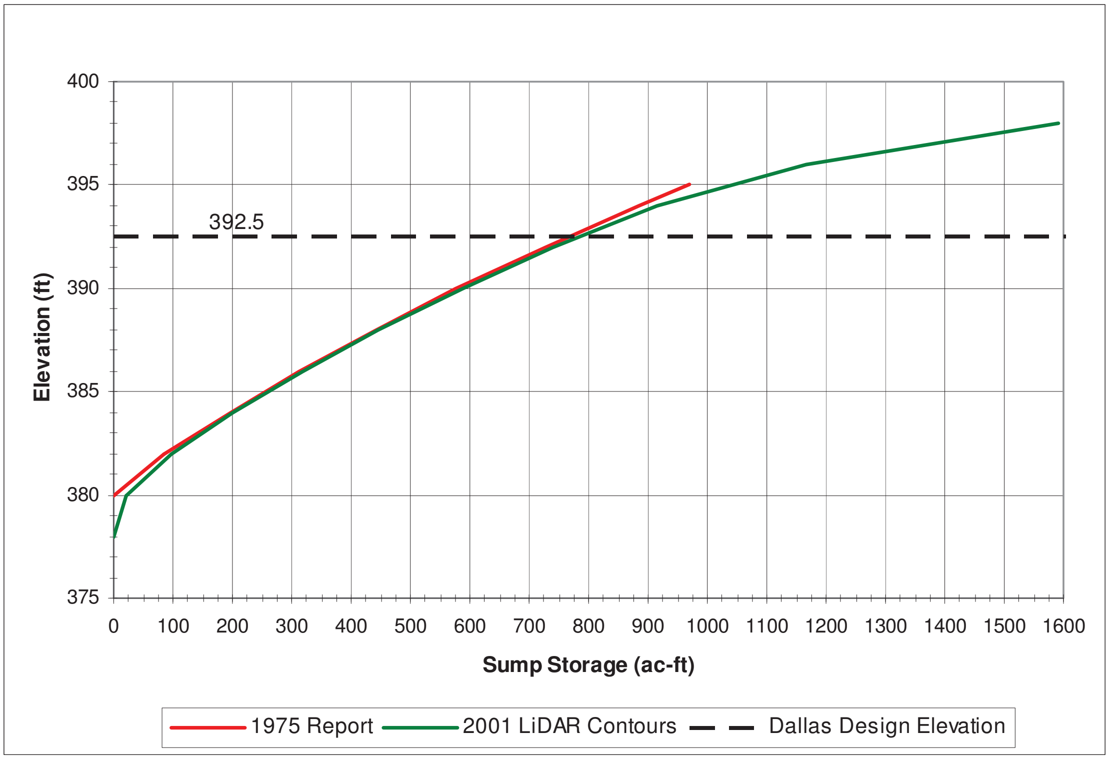

Figure 2.5 - Able Sump Elevation-Volume Curves

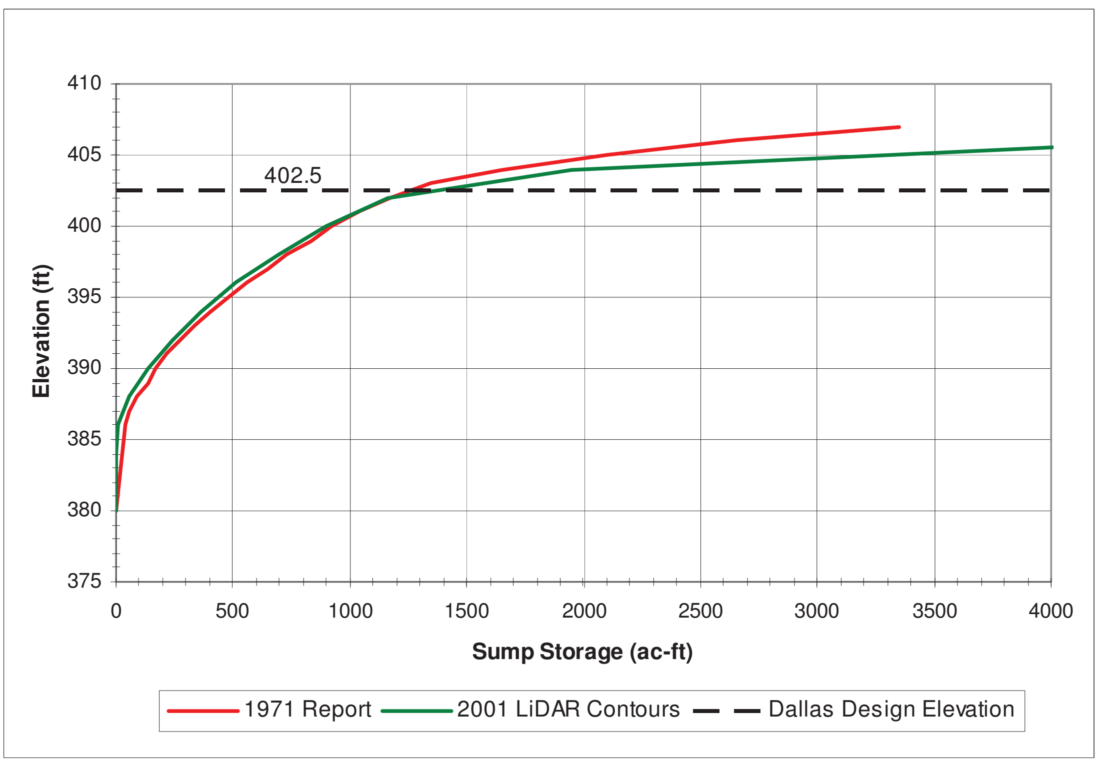

Figure 2.6 - Hampton-Oak Lawn Sump Elevation-Volume Curves

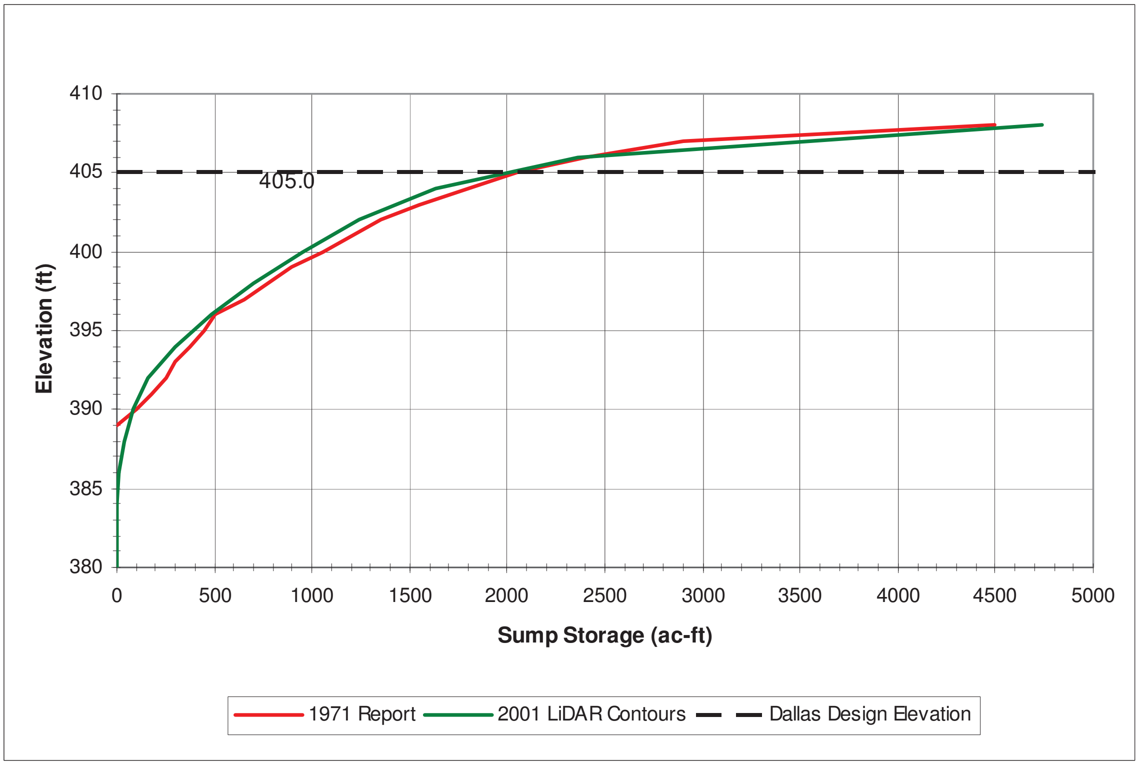

Figure 2.7 - Record Crossing Sump Elevation-Volume Curves

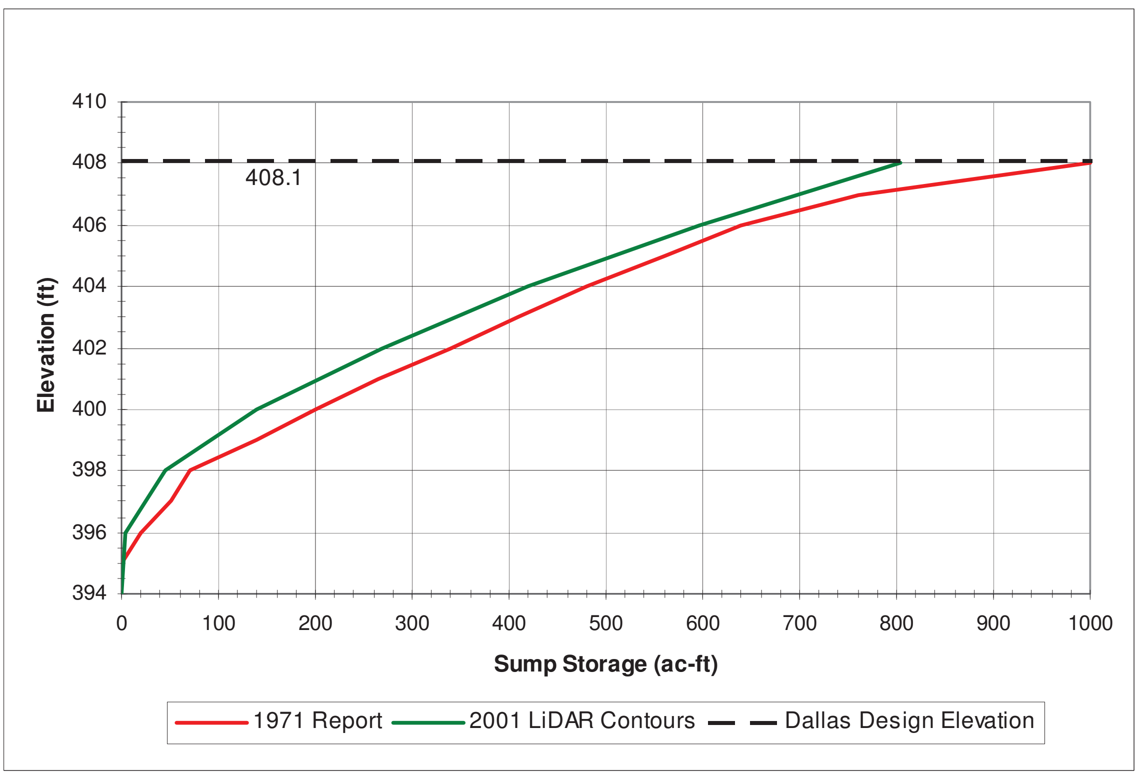

Figure 2.8 - Nobles Branch Sump Elevation-Volume Curves

2.3.4 Dallas Floodway Outfall Elevations

The East Levee pump stations discharge into the Dallas Floodway. Thus, the water surface elevation in the Floodway at the pump station outfalls can have a significant effect on the capacity of the pump stations. The higher the water surface elevations in the Floodway at the pump station outfalls, the greater the effective head the pump stations must discharge against. Of course, the water surface elevations in the Floodway are primarily determined by the flow rate in the river. Therefore, the flow rate in the Dallas Floodway has a direct impact on the pumping capacity of the East Levee pump stations.

To calculate Dallas Floodway water surface elevations at the pump station outfalls, the flow rates in the Floodway had to be established. Also, since significant changes to the Floodway geometry throughout the reach that borders the East Levee are planned as part of the ongoing Dallas Floodway Extension and Trinity Parkway projects, the proper Floodway geometry to use in the calculations had to be decided.

The USACE Fort Worth District provided their most current HEC-RAS model of the Dallas Floodway. The model included both existing conditions and baseline conditions geometries. The baseline condition is an intermediate state between existing conditions and the complete Trinity Park geometry. The baseline conditions geometry includes the proposed Trinity Parkway and proposed new bridges across the Floodway, and includes the borrow areas for roadway fill which will ultimately become lake features in Trinity Park. However, in the base conditions geometry, the borrow areas are not yet dammed off to create the lakes. The final proposed conditions HEC-RAS model including all Trinity Park features was not yet available. It was deemed appropriate to use the baseline conditions HEC-RAS geometry to compute water surface profiles for the Interior Drainage Study, since the baseline geometry is the closest match to proposed Dallas Floodway conditions in the near future.

Another topic of discussion with the Fort Worth District was the Dallas Floodway flow rates to be used to compute outfall elevations for the Interior Drainage Study. This same issue confronted the Fort Worth District as part of the design of the Dallas Floodway Extension levees and interior drainage features for the Cadillac Heights Levee and Lamar Levee areas. For these designs, the District used a steady uniform flow in the Trinity Floodway of 20,000 cfs, slightly less than a 2-year event on the Dallas Floodway at the East Levee area.

In the Dallas Floodway Extension General Reevaluation Report, Appendix A, the Fort Worth District describes the coincident peak analysis developed for the Dallas Floodway Extension study. The District prepared a statistical correlation between Trinity River flows and localized precipitation at Dallas for the period of May 1957 to September 1994. This period was used since most of the major flood control reservoirs which impact Trinity River flows at Dallas were in place by May 1957. A generally weak correlation was found between localized storms at Dallas and high mean flows on the Trinity River. The explanation given for the lack of correlation is that substantial rainfall in the central and upper portions of the Clear, West and Elm Forks of the Trinity is required to produce high sustained flows at Dallas. The report notes that “runoff from the small localized interior basins watersheds at Dallas is often fully evacuated prior to the arrival [of] significant flows on the river itself.”

Based on these findings, the Fort Worth District elected to use the prevailing steady-state release rate used in evacuating water from USACE reservoir flood control pools (15,000 cfs) plus an assumed 5,000 cfs from uncontrolled Trinity River inflows to yield a total design tailwater flow rate of 20,000 cfs for the Dallas Floodway Extension project. Based on the District's analysis, this approach was adopted for the East Levee Interior Drainage Study.

With a steady uniform flow of 20,000 cfs in the Floodway, the base conditions HEC-RAS model was executed and outfall elevations at all East Levee pump stations and pressure sewers were computed, as shown in Table 2.7. At this flow, water surface elevations in the Floodway exceed the interior sump design elevations at all the gravity sluices; therefore, gravity discharge from the sumps is not possible under these conditions.

Table 2.7 - East Levee Interior Drainage Feature Outfall Elevations

Feature	River Station	Flow (cfs)	Outfall WSEL (ft)
Hampton Pumping Plant	145117.5	20,000	410.58
Baker Pumping Plant	129668.0	20,000	406.03
Turtle Creek Pressure Sewer	125368.5	20,000	405.00
Dallas Branch Pressure Sewer	122559.5	20,000	404.11
Woodall Rodgers Pressure Sewer	121842.5	20,000	403.93
Able Pumping Plant	115967.5	20,000	402.11

2.3.5 East Levee Pressure Sewer Hydraulics

A hydraulic analysis of the East Levee pressure sewers was developed using fundamental fluid mechanics applied to pressure pipe flow. The physical characteristics of the pressure sewers were obtained from available as-built plans in the City of Dallas Public Works Vault. For the hydraulic analysis of a given pressure sewer, the Bernoulli equation was utilized between points of interest along the length of the conduit. The computations started at the downstream end of the pressure sewer where the outfall conditions are known and proceeded upstream. Points of interest were typically locations where the cross section of the conduit changed, as well as the entrance and exit of the pressure sewers. The Bernoulli equation between two points of a pressure conduit with steady incompressible flow (with flow in the direction from point 1 to point 2) is

$$\frac{p_1}{\gamma} + z_1 + \frac{v_1^2}{2g} - H_L = \frac{p_2}{\gamma} + z_2 + \frac{v_2^2}{2g} \qquad \textbf{Equation 6}$$

where: p = pressure in the conduit
γ = specific weight of water
z = height of centroid of conduit above the elevation datum
v = average flow velocity in conduit
g = local acceleration of gravity
H_L = total head loss between points 1 and 2

The term $\frac{p}{\gamma}$ is the pressure head. The locus of points described by $\frac{p}{\gamma}+z$ is the hydraulic grade line (HGL) of the conduit. The term $\frac{v^2}{2g}$ is the velocity head. The locus of points described by $\frac{p}{\gamma}+z+\frac{v^2}{2g}$ is the energy grade line (EGL) of the conduit. Thus, it is apparent that the HGL and EGL are separated from each other by the velocity head along the length of the conduit. The total head loss term H_L includes both friction losses and minor losses such as entrance/exit losses, bend losses, and contraction/expansion losses. Despite the terminology, the "minor losses" may be more significant than the friction loss.

The friction loss component of the total head loss may be evaluated by a number of different empirical equations such as the Hazen-Williams equation or Manning's equation. The Hazen-Williams equation for the friction slope of a conduit of general cross section is

$$S_f = \frac{0.6v^{1.852}}{C^{1.852}R_H^{1.167}}$$ **Equation 7**

where: S_f = friction slope
v = average flow velocity in conduit
C = Hazen-Williams friction coefficient
R_H = hydraulic radius of the conduit

For comparison, the friction slopes were also calculated using Manning's equation, which may be written

$$S_f = \left(\frac{vn}{1.486R_H^{2/3}}\right)^2$$ **Equation 8**

where: S_f = friction slope
v = average flow velocity in conduit
n = Manning's friction coefficient
R_H = hydraulic radius of the conduit

Both Manning's n and the Hazen-Williams C are functions of the pipe material. For this analysis, a C value of 120 and an n value of 0.013 were used. The computed friction slopes from the two different equations with these coefficients were very close. The Hazen-Williams friction slope was used for the analysis. The friction loss between two points is obtained by multiplying the friction slope by the length of conduit between the points.

Minor losses are generally expressed as a loss coefficient (K) times the velocity head. For this analysis, minor losses were applied to the EGL. The entrance loss coefficient used was K_{entr} = 0.5. For exit conditions where the velocity of the receiving waters is zero or negligible, the exit loss coefficient is generally K_{exit} = 1.0, meaning that the entire velocity head is lost. For this analysis, since the velocity heads in the Trinity River at the pressure sewer outfalls are known from the HEC-RAS model described in Section 2.3.4, the exit loss was taken as the difference in velocity heads between the conduit and the Trinity River at the outfall.

Two other types of minor losses were included in this analysis – sudden expansion and sudden contraction. For a sudden expansion, the loss coefficient is given by the following equation, with the flow in the direction from point 1 to point 2:

$$K_{\exp} = \left(1 - \frac{A_1}{A_2}\right)^2 \quad \textbf{Equation 9}$$

where: K_{exp} = expansion loss coefficient
A_1 = cross-sectional area of upstream (smaller) conduit
A_2 = cross-sectional area of downstream (larger) conduit

To compute the expansion loss, the loss coefficient given by Equation 9 is applied to the upstream (point 1) velocity head (larger velocity, smaller conduit).

The contraction loss coefficient varies depending upon the ratio of the conduit areas. The following table was used for the contraction coefficient:

Table 2.8 - Expansion Loss Coefficients (adapted from *Fluid Mechanics*, Eighth Edition, Streeter and Wylie)

Area Ratio A_2/A_1	Expansion Loss Coefficient K_{exp}
0.1	0.363
0.2	0.339
0.3	0.308
0.4	0.268
0.5	0.219
0.6	0.164
0.7	0.105
0.8	0.053
0.9	0.015
1.0	0.000

To compute the contraction loss, the loss coefficient determined from Table 2.8 is applied to the downstream (point 2) velocity head (larger velocity, smaller conduit).

2.3.5.1 Turtle Creek Pressure Sewer Hydraulic Analysis

The Turtle Creek Pressure Sewer inlet is located at the intersection of Turtle Creek Boulevard and Park Bridge Court. The concrete overflow spillway at the inlet is 175 feet long with a crest elevation at 419.5 feet. The non-overflow section of the dam is at elevation 426.0 feet. However, water ponded above elevation 424.0 feet will bypass the dam by spilling into Turtle Creek Boulevard and back into Turtle Creek downstream of the dam. Therefore, elevation 424.0 feet may be considered the maximum ponded elevation at the pressure sewer inlet. All of the discharge over the spillway continues down the Turtle Creek channel to Hampton-Oak Lawn Sump.

According to the 2003 paper, “History of the Dallas Floodway,” by Furlong, Ajemian, and McPherson, the rated capacity of the Turtle Creek Pressure Sewer is 3,850 cfs. From the hydraulic analysis developed for this project, the headwater elevation for this discharge in the pressure sewer would be 424.5 feet. At the maximum effective headwater elevation of 424.0 feet, the pressure sewer discharge is approximately 3,800 cfs. Based on this analysis, it is believed that 3,800 cfs is the maximum practical discharge that the pressure sewer can convey. At elevation 424.0 feet, the discharge over the spillway would be approximately 5,000 cfs. According to the hydrologic modeling developed for this analysis, the peak 100-year runoff approaching the Turtle Creek Pressure Sewer inlet is approximately 8,500 cfs. Assuming that the entire 100-year peak flow is conveyed by the combination of the pressure sewer and the overflow spillway, the 100-year peak water surface elevation at the pressure sewer inlet would be approximately 423.9 feet.

Figure 2.9 illustrates the variation of pressure sewer and overflow spillway discharge as the headwater elevation increases from the spillway crest (419.5 feet) to the maximum headwater elevation (424.0 feet).

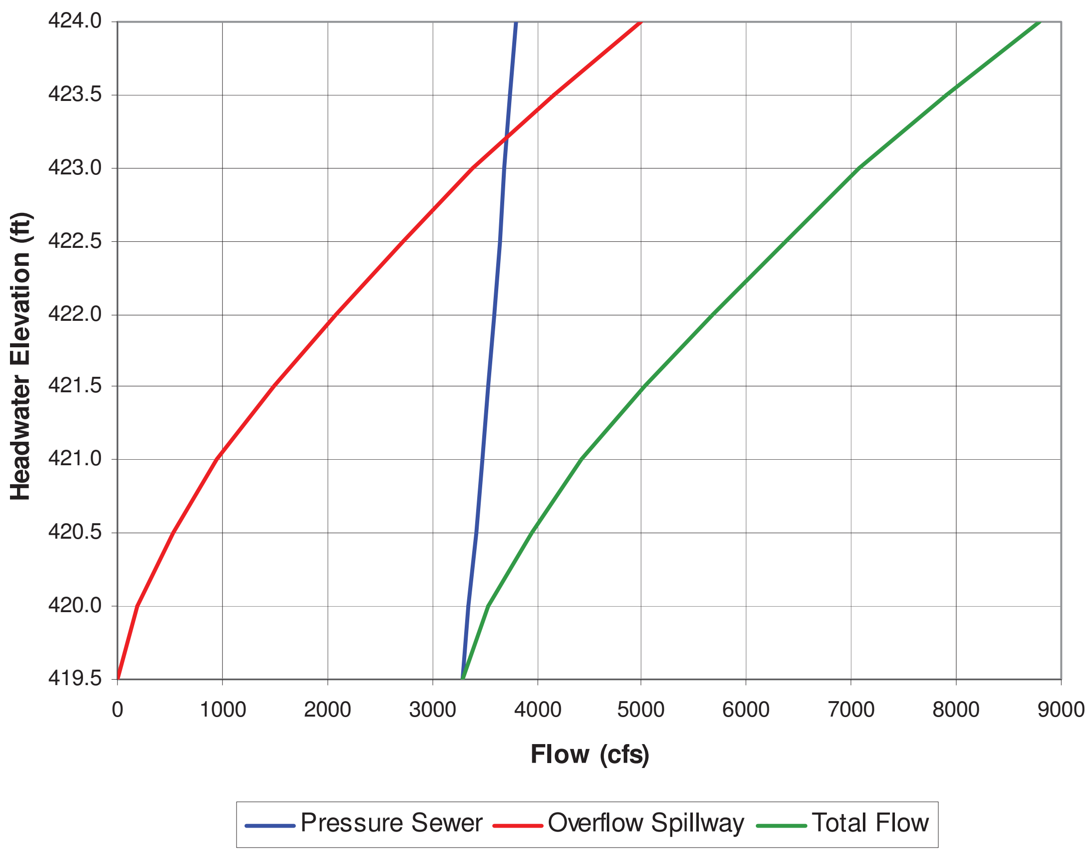

Figure 2.9 - Variation of Pressure Sewer and Overflow Spillway Discharge, Turtle Creek Pressure Sewer

A profile plot of the Turtle Creek Pressure Sewer showing the computed HGL and EGL for a steady uniform discharge of 3,800 cfs is shown in Exhibit 10. In the profile, there are various locations (particularly near the downstream end) where expansions occur in the cross-sectional area, resulting in a “hydraulic recovery,” an increase in the HGL in the downstream direction.

2.3.5.2 Woodall Rodgers Pressure Sewer Hydraulic Analysis

No plans for the upstream extension of the Woodall Rodgers Pressure Sewer (upstream of the Field Street manhole) were available. All of the plans obtained from the City of Dallas Public Works Vault show the pressure sewer upstream of the Field Street manhole labeled “future pressure sewer.” It is possible that the plans for the upstream extension of the pressure sewer were archived by TxDOT, but the project team was not able to obtain any plans from TxDOT. However, the fact that the pressure sewer was built upstream of Field Street was verified by an interior inspection of the pressure sewer. Because no plans for the Woodall Rodgers Pressure Sewer upstream of the Field Street manhole were available, this hydraulic analysis extends from the pressure sewer outfall in the Dallas Floodway to the Field Street manhole.

According to the 2003 paper, “History of the Dallas Floodway,” by Furlong, Ajemian, and McPherson, the capacity of the Woodall Pressure Sewer is 1,679 cfs. This figure agrees with the 100-year flow at the downstream end of the pressure sewer (from Stemmons Freeway to the outfall) shown on the plans obtained from the Public Works Vault. The Woodall Rodgers Pressure Sewer subbasin in the hydrologic modeling developed for the interior levee drainage study predicts a 100-year peak flow of 1,845 cfs for a slightly larger drainage area (606 acres vs. 544 acres shown on the pressure sewer plans). There are a few lateral storm drains that connect to the pressure sewer between Industrial Boulevard and Field Street. At Field Street, the plans show the 100-year flow to be 1,565 cfs, with a hydraulic grade line elevation of 416.1 feet.

The top of the manhole at Field Street is at elevation 419.0 feet. In the hydraulic analysis of the pressure sewer, this elevation was considered the maximum allowable headwater for the pressure sewer. With a steady uniform flow of 1,679 cfs in the entire conduit, the hydraulic analysis predicted a headwater elevation at the Field Street manhole of 417.3 feet. The headwater elevation for a steady uniform flow of 1,845 cfs in the conduit exceeds 419.0 feet. The maximum steady uniform flow in the conduit with a headwater elevation of 419.0 feet is 1,790 cfs. To be conservative, the capacity of the pressure sewer was assumed to be 1,679 cfs to compute the overflow from the pressure sewer drainage basin into Able Sump.

Figure 2.10 shows the variation of pressure sewer discharge with headwater elevation at the Field Street manhole.

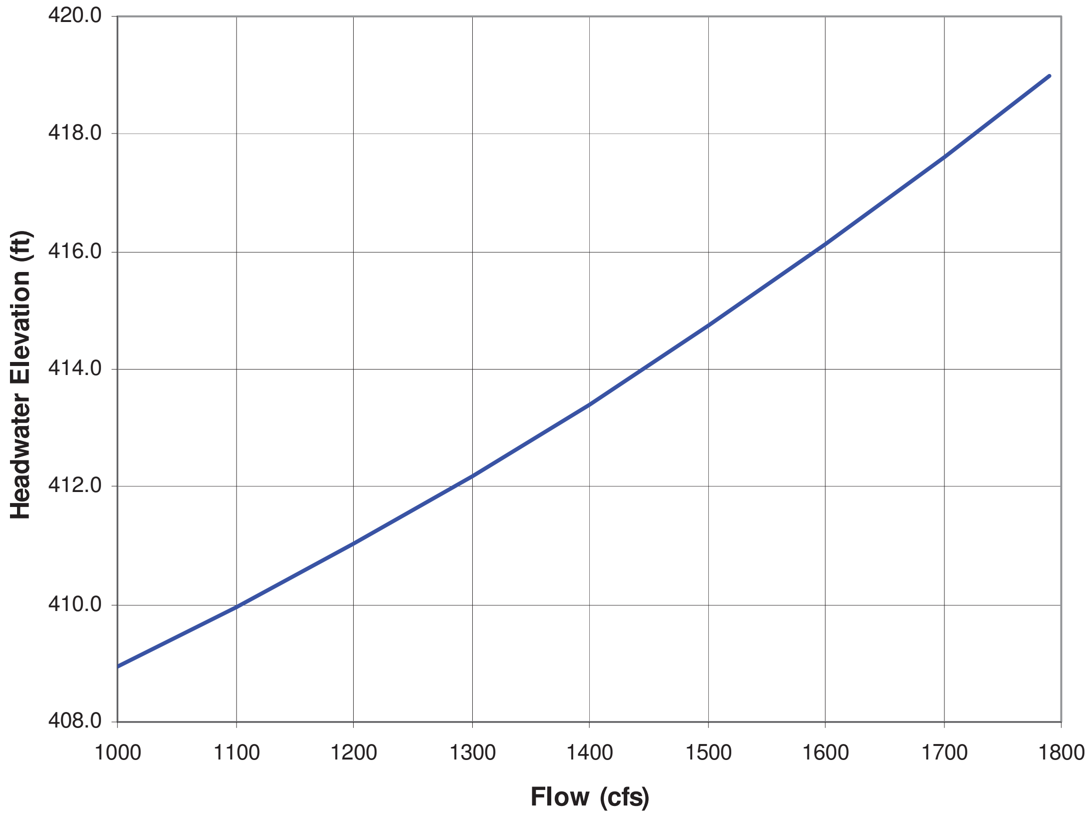

Figure 2.10 - Variation of Pressure Sewer Discharge with Headwater Elevation at Field Street Manhole, Woodall Rodgers Pressure Sewer

A profile plot of the Woodall Rodgers Pressure Sewer showing the computed HGL and EGL for a steady uniform discharge of 1,679 cfs is shown in Exhibit 11.

2.3.5.3 Dallas Branch Pressure Sewer Hydraulic Analysis

The Dallas Branch Pressure Sewer is one of the oldest pressure sewers in Dallas. Although the original Dallas Branch storm sewer system runs from the Dallas Floodway all the way upstream to the east side of Central Expressway at Haskell Avenue, the “pressure sewer” portion of the system extends from the outfall in the Dallas Floodway to an intake structure just east of Field Street. Until the Woodall Rodgers Freeway and Pressure Sewer were constructed, the Dallas Branch system drained much of the Central Business District and the Uptown area. When the Woodall Rodgers Pressure Sewer was constructed, all of the Dallas Branch drainage basin north of Woodall Rodgers Freeway was diverted to the Woodall Rodgers Pressure Sewer. Currently, the Dallas Branch Pressure Sewer drains the portion of downtown bordered approximately by Elm Street, Stemmons Freeway, Woodall Rodgers Freeway, and Central Expressway, with a total drainage area of approximately 312 acres. Approximately 103 acres of the Dallas Branch Pressure Sewer drainage basin is drained by the Pacific Avenue Lateral, which joins the Dallas Branch Pressure Sewer near Stemmons Freeway.

According to the 2003 paper, “History of the Dallas Floodway,” by Furlong, Ajemian, and McPherson, the capacity of the Dallas Branch Pressure Sewer is 572 cfs. From the original pressure sewer plans, the top of the intake structure east of Field Street is at elevation 416.7 feet. This elevation was taken as the maximum headwater elevation for the pressure sewer. According to the hydrologic modeling developed for the interior levee drainage study, the 100-year peak flow for the Pacific Avenue system is 493 cfs. The Pacific Avenue system flow is added to the Dallas Branch Pressure Sewer flow. With a maximum headwater elevation of 416.7 feet, the maximum Dallas Branch Pressure Sewer capacity at the intake structure is 857 cfs, resulting in a total pressure sewer discharge downstream of the junction with the Pacific Avenue Lateral of 1,350 cfs.

Figure 2.11 illustrates the variation of pressure sewer discharge upstream of the Pacific Avenue Lateral with headwater elevation at the intake structure. Figure 2.11 was developed assuming an inflow of 493 cfs from the Pacific Avenue Lateral.

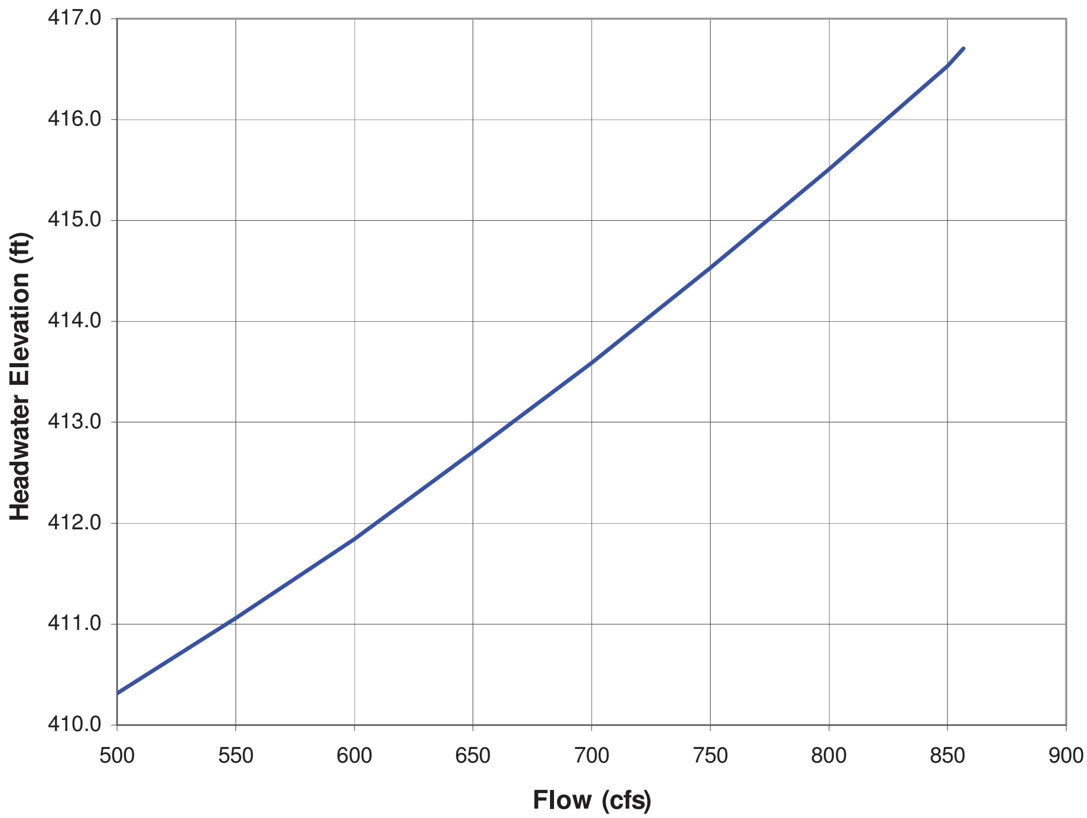

Figure 2.11 - Variation of Pressure Sewer Discharge with Headwater Elevation, Dallas Branch Pressure Sewer

A profile plot of the Dallas Branch Pressure Sewer showing the computed HGL and EGL for a steady discharge of 857 cfs plus 493 cfs from the Pacific Avenue Lateral is shown in Exhibit 12.

3. EXISTING CONDITIONS ANALYSIS

After the hydrologic parameters were computed, the HEC-HMS model was developed. The initial model development was done with each sump area in its own separate HEC-HMS model. Preliminary model simulations revealed that this approach was inappropriate for existing conditions since the predicted sump levels were high enough to cause the sumps to interact for large flood events such as the 100-year flood. Therefore, all the East Levee sumps and their contributing watersheds were combined in one HEC-HMS model to allow the calculation of overflow from one sump to another.

3.1 HEC-HMS INNOVATIVE APPROACHES

The development of the combined sump model in HEC-HMS required a number of innovative approaches to account for the interaction between the sump areas and to model some of the unique features of the East Levee watershed.

3.1.1 Pressure Sewer Overflows

Pressure sewer overflows were computed by developing the runoff hydrographs for the subbasins contributing to the pressure sewer inlet, then subtracting the constant pressure sewer capacity from those hydrographs. The difference between the inflow hydrograph and the pressure sewer capacity was considered overflow that continued downstream into the East Levee sumps. This approach assumed a constant capacity for the pressure sewer over the duration of the storm event.

A separate HEC-HMS model was developed for the Turtle Creek Pressure Sewer. The inflow hydrograph for the Turtle Creek Pressure Sewer was developed using the Upper Turtle Creek subbasin as the contributing drainage area. This approach assumed that the Cole Park Detention Vault would detain the runoff from its contributing drainage area such that the peaks were not coincident and the stored volume would be released slowly over time; therefore, the peak flows into the Hampton-Oak Lawn Sump at the downstream end of Turtle Creek would come from overflows from the Upper Turtle Creek subbasin.

Since overflows from both Woodall Rodgers and Dallas Branch Pressure Sewers contribute to Able Sump, the inflow hydrographs for these two pressure sewers were developed in a single HEC-HMS model. The contributing area for the Woodall Rodgers Pressure Sewer was the Woodall Rodgers subbasin. Dallas Branch and Pacific subbasins made up the contributing area for the Dallas Branch Pressure Sewer.

Once the HEC-HMS models for the pressure sewer contributing subbasins were executed, the computed hydrographs were read into Microsoft Excel using the HEC-DSS Data Exchange Add-In for Excel. Excel was used to subtract the pressure sewer capacities from the inflow hydrographs to compute the overflow hydrographs, then the HEC-DSS Add-In was used to store the computed overflow hydrographs in HEC-DSS. The pressure sewer overflows were incorporated into the East Levee combined HEC-HMS model using Source elements – each pressure sewer overflow was represented by a Source element. A discharge gage was created for each pressure sewer overflow linked to the appropriate path containing the computed overflows in HEC-DSS, and the corresponding Source element and discharge gages were linked in the HEC-HMS model. In this way, changes could be made to the pressure sewer overflows if necessary and the East Levee combined model would be updated with the latest computed overflow hydrographs.

Overflows from the Mill Creek drainage system were incorporated into the East Levee HEC-HMS model in a similar manner, but the source of the overflow hydrograph was different. The Halff Associates Master Drainage Plan Study of Mill Creek report provided by the City indicated that a peak flow of 2,600 cfs and a total volume of 308 acre-feet spilled over from the Mill Creek watershed into Able Sump for the 100-year, 2-hour event due to lack of capacity in the Belleview Pressure Sewer and its contributing storm sewer network. The computed overflow hydrograph was obtained from Halff Associates and used as an input to the HEC-HMS model. In the model, the timing of the overflow hydrograph was adjusted to coincide with the peak inflow to Able Sump, for a "worst-case" scenario. For calibration simulations, the magnitude of the Mill Creek overflow hydrograph was adjusted by multiplying the hydrograph by a factor, effectively making the Mill Creek system overflow a calibration parameter. One limitation of the use of the computed Mill Creek system overflow hydrograph is that the storm duration for the overflow hydrograph is 2 hours, whereas the storm duration for hypothetical event simulations for the Interior Levee Drainage Study is 24 hours. The discrepancy in storm durations probably does not make a big difference in the peak flow, but the volume would be considerably different for a 2-hour event as opposed to a 24-hour event. Because the primary design criterion for the interior drainage system is peak sump elevation, this minor limitation does not have a significant impact on the conclusions of this study.

3.1.2 Pump Outflows

The City provided pump capacity and turn-on/shut-off elevations for all the pumps at the East Levee pumping plants. These data were used to develop an elevation-outflow curve for each sump. In HEC-HMS version 2.2.2, it is not possible to specify different outflows at the same elevation for different conditions, meaning that separate turn-on and shut-off elevations cannot be described in the model. Therefore, only the turn-on elevations were used in the elevation-outflow curve. This approach is reasonable since the primary goal of the model is to compute peak elevations in the sumps; computed elevations on the falling limb of the hydrograph or at lower sump elevations are not as important.

HEC-HMS interpolates between points on the elevation-outflow curve, which is inappropriate in this case since the curve represents pumps turning on as the water level rises. The elevation-outflow curves had to be modified to prevent this interpolation from affecting the outflow, so points were added to the curves 0.01 ft lower than each subsequent pump turn-on elevation so that the outflow would be constant between pump turn-on elevations. This ensured that the curves retained the characteristic stair-step shape of a pump outflow curve in spite of the interpolation. This approach is the only way to model pumping from a reservoir in HEC-HMS version 2.2.2, since other reservoir outflow methods use orifice or weir flow calculations.

With the release of HEC-HMS version 3.0.0, a new pump station reservoir outflow method was added. With this method, actual pump curves can be used to specify the pump discharge based on the headwater and tailwater conditions. Additionally, separate turn-on and shut-off elevations can be specified. When the HEC-HMS version 2.2.2 models were converted to version 3.0.0, the pump outflows were re-configured to use the new method, with actual pump curves for the pumps when available. However, the results in version 3.0.0 were not significantly different than those obtained with version 2.2.2, implying that the original approach described above was valid.

3.1.3 Sump Overflows

From the 2001 LiDAR 2-ft contours, it can be seen that as the water level in Record Crossing Sump approaches its design elevation of 405.0 ft, water will spill out of Record Crossing Sump into Hampton-Oak Lawn Sump. Likewise, water will spill out of Hampton-Oak Lawn Sump into the Able Sump drainage area if Hampton-Oak Lawn Sump exceeds its design elevation of 402.5 ft. These conditions will be addressed in the recommendations developed from this study. However, for existing conditions simulations, the overflow must be accounted for in the model. This required an innovative approach, since the total outflow from the sump under these conditions consists of a combination of pump outflow and weir overflow, but the two types of flow must be separated since they occur at different locations and contribute to different receiving waters.

The modeling approach developed for this situation was to calculate a rating curve for the weir overflow separately and incorporate the weir flow into the sump elevation-outflow curve. In all cases, the weir flow begins at an elevation higher than the highest pump turn-on elevation at the sump's pumping plant, so the addition of the weir flow curve does not affect the representation of the pump outflow. In the model, the combined pump/weir outflow from a given sump discharges into a Diversion element, where the weir overflow can be subtracted from the total and diverted into the appropriate downstream sump.

The overflow weir from Record Crossing Sump to Hampton-Oak Lawn Sump was approximated by a 90-ft long rectangular weir section at elevation 403.0 ft. This weir overflow occurs at Inwood Road south of Irving Boulevard. The overflow weir from Hampton-Oak Lawn Sump into the Able Sump watershed (specifically, low-lying portions of the Industrial subbasin) was approximated by a 400-ft long weir at elevation 403.0 ft. Overflow from Hampton-Oak Lawn Sump can occur at numerous locations along the south/west bank of the sump.

Because Hampton-Oak Lawn Sump does not overflow directly into Able Sump, a separate reservoir was coded into the model to receive this weir overflow. This low-lying area is not actually a sump or a reservoir and should not be considered as additional sump storage, since it consists of streets and developed property. However, HEC-HMS can be used to compute an elevation hydrograph for the area since it can be represented by an elevation-volume curve developed from the 2001 LiDAR 2-ft contours. It would be inappropriate to assume that the water level in the low-lying area achieved the same elevation as Hampton-Oak Lawn sump, since they are connected only by weir flow occurring above elevation 403.0 ft. Hence, mapping flooding in this area at the same elevation as Hampton-Oak Lawn sump would have resulted in an unrealistically high estimate of flooding in the area. For existing conditions, this low-lying area between Hampton-Oak Lawn and Able Sumps was mapped with its own computed peak elevation. Recommendations to prevent this area from flooding will be presented in a subsequent section of this report.

The weir overflow hydrograph from Hampton-Oak Lawn Sump had to be routed through two separate Diversion elements in the model. The first Diversion element extracted double the computed weir flow from the combined sump outflow. The second Diversion element split the flow between the low-lying Industrial Subbasin area and Able Sump. This approach of doubling the weir flow and then splitting it back in half had to be used since there was no way to pass the flow from the low-lying Industrial Subbasin area into Able Sump, which it would ultimately reach by overland flow or the storm sewer system (an elevation-outflow curve for the low-lying area would not have been practical since there are multiple ways the water reaches Able Sump). Even though this approach "created" volume in the model, it did not affect the model results for the sumps since the extra water was diverted into the low-lying Industrial Subbasin area, which effectively acted as a sink in the model. The primary drawback of this approach was that the weir overflow from Hampton-Oak Lawn Sump into the low-lying Industrial Subbasin area was allowed to pond and not flow out of the low-lying area, which undoubtedly overestimates the flooding in the area, but not to the degree of assuming the area achieved the same elevation as Hampton-Oak Lawn Sump.

3.2 CALIBRATION

The City of Dallas Flood Control District provided calibration data for the following three storm events: May 5-7, 1995; October 18-20, 2002; and July 28-30, 2004. The data consisted of measured incremental precipitation data for ALERT sensors sufficiently distributed to provide good coverage of the East Levee watershed, measured water levels in the sumps at the stormwater pumping plants, and pump records indicating how many pumps were on at each station during a given time period (15-minute increments). Measured water levels were available at Hampton Pumping Plant (Record Crossing Sump), Baker Pumping Plant (Hampton-Oak Lawn Sump), and Able Pumping Plant (Able Sump). No measured sump data were available for Nobles Branch Sump; however, the outflow from Nobles Branch Sump at the Grauwyler Gated Culvert was a component of the calibration for Record Crossing Sump.

The basic philosophy of the calibration process was to attempt to match the timing and magnitude of the peak sump stages as closely as possible. More emphasis was put on matching the peak, since that is the elevation used for inundation mapping. Other parts of the hydrograph, particularly the falling limb, were not as much of a concern. It is desirable but usually not practical to match all parts of the hydrograph equally well, since at lower flows and sump levels, the multitude of system processes unaccounted for in the model have a relatively more significant affect than they do at higher flows and sump levels. Thus, the criteria for judging the success of calibration were first and foremost matching the peak sump stage in magnitude and timing, and secondly matching the overall shape of the hydrograph.

Of the three events available for calibration, the May 1995 event was the most similar to a hypothetical storm such as the 100-year event, because it had a burst of very intense rainfall, resulting in a rapid peak response from the watershed. The other two storm events were less intense and the rainfall was distributed over a longer period. Therefore, it was decided to focus more attention on attempting to match the peak sump elevation of the May 1995 event than the other events.

The July 2004 event exhibited some problems with the measured sump stage for Able and Hampton-Oak Lawn sumps. For both of these sumps, the measured sump stage hydrograph had an unnatural drop to the minimum sump elevation during the peak of the storm, but later recovered to apparently correct levels. It is unknown whether the problems were a result of missing data or a temporary gage malfunction. In spite of these problems, the July 2004 event was still used for calibration, since portions of the sump stage hydrographs appeared to be good for Able and Hampton-Oak Lawn sumps, and no problems were apparent for Record Crossing Sump.

Because the maximum precipitation intensity for the May 1995 event is similar to the maximum intensity for the 100-year hypothetical storm event, the Mill Creek overflow hydrograph used for the May 1995 event was the same as the assumed 100-year hydrograph developed from the Halff Associates report as described in Section 3.1.1. This may have overestimated the volume of the Mill Creek overflow for the May 1995 event, but not enough to make the computed Able Sump peak stage exceed the measured peak stage. For the other two calibration events, the peak flow and total volume of the Mill Creek runoff were assumed to be one-half of the 100-year hypothetical storm event values.

3.2.1 Calibration Methodology

It was not possible to use the automatic parameter optimization method in HEC-HMS for calibration because those routines work only when calibrating to a flow hydrograph, not a stage hydrograph. An attempt was made to perform spreadsheet water-balance calculations using the measured sump stage data and the sump elevation-volume curves to back-calculate a composite inflow hydrograph for each sump that could be used with the automatic calibration method. The problem with this approach is that it includes reservoir routing, introducing possible sources of error from the measured stages, the elevation-volume curve, and the measured outflows.

For each time period, the change in measured sump stage was converted to a change in sump volume using the sump elevation-volume curve. The change in sump volume during a time period was divided by the time increment to compute an average flow into the sump for the period – the computed flow was positive if the sump level rose over the period, and negative if the sump level fell. Similarly, the average outflow from the pumping plant for a time period was computed by multiplying the number of operating pumps by their respective capacities. The sign of the pump outflow was negative, since this flow was leaving the sump. The pump outflow hydrograph was then subtracted from the net inflow hydrograph to compute a composite runoff hydrograph into the sump.

This approach was problematic, because there were many time periods for a given calibration event for which the computed runoff was a large negative value (negative values near zero are to be expected given the averaging effect and approximations inherent in these calculations). Negative computed runoff values indicate that the sump level was falling at a rate greater than the pumping rate. For these time periods, the runoff was assumed to be zero.

The composite runoff hydrographs computed in this manner for the sumps exhibited an irregular and unnatural sawtooth shape for some time periods, with adjacent time increments of large inflows and zero inflow. Thus, the computed composite inflow hydrographs for the sumps were not usable for calibration.

It is believed that this approach failed because there were too many unaccounted-for effects in the analysis. For example, fluctuations in measured sump water levels could be caused by waves or other phenomena not associated with a net change in sump volume. Furthermore, there could have been sources of outflow which were not accounted for in the analysis, such as flow through gravity sluices or other losses. Also, averaging sump volume changes over a relatively long time increment (15 minutes) may have created too much error in computed sump inflows. Certainly any of these factors could have introduced significant error in the analysis.

Because it was not feasible to use the back-calculated sump inflow hydrographs for the calibration events, it was decided to attempt to calibrate the model to measured sump levels instead. Of course, this approach involves the same assumptions previously discussed, but it eliminates the step of back-calculating a composite inflow hydrograph for each sump. The primary difficulty with this approach was that the reservoir routing could not be performed in HEC-HMS, since only an elevation-outflow curve can be specified for a reservoir, not the desired time series of outflows. Therefore, HEC-HMS was used to compute the inflow hydrographs for the sumps and a spreadsheet was used for reservoir routing to compute the sump stage hydrographs. This approach resulted in more satisfactory results.

3.2.1.1 Calculation of Sump Inflow Hydrographs Using HEC-HMS

The first step in setting up the HEC-HMS model for calibration was to modify the connectivity of the subbasins to include a Junction element upstream of all the sumps to be calibrated. All of the contributing subbasins, pressure sewer overflows, and sump overflows entering a given sump were connected to its Junction element. The Junction element sums all of the individual entering hydrographs to compute a composite inflow hydrograph to the sump. Then this composite inflow hydrograph was used to calculate the spreadsheet reservoir routing for the sump.

Next, the model had to be set up to use measured rainfall data for the calibration events. The ability of the HEC-HMS software to compute a spatial precipitation distribution automatically is one of the major advantages it has over HEC-1. For this analysis, the HEC-HMS inverse-distance gage weighting option was used. This approach requires the coordinates of the precipitation gages to be entered in the model, as well as the coordinates of one or more precipitation nodes for each subbasin. One precipitation node was established for each subbasin, located at the centroid. The HEC-HMS model creates a coordinate system at each precipitation node and determines the closest precipitation gage in each quadrant. Then a weighted average precipitation is computed for the precipitation node based on the inverse squared distance between the node and the closest gage in each quadrant.

For each calibration event, the measured precipitation data provided by the City were written to an HEC-DSS database to allow HEC-HMS to access the data. Then, precipitation gages were created in HEC-HMS for each ALERT precipitation gage with data provided by the City. The HEC-HMS precipitation gages referenced the appropriate HEC-DSS file and pathname containing the precipitation data, and included the latitude-longitude coordinates of the gage. Then, for each subbasin, the coordinates of the centroid were used to create a precipitation node. This was a convenient method of computing an accurate spatial precipitation distribution for the East Levee watershed.

Finally, the HEC-HMS model was executed to compute the composite inflow hydrographs for Record Crossing Sump, Hampton-Oak Lawn Sump, and Able Sump. These computed hydrographs were read into Excel using the HEC-DSS Add-In.

3.2.1.2 Spreadsheet Reservoir Routing

For each sump, the composite inflow hydrograph was used to compute a net change in sump volume for each time increment. Then the elevation-volume curve for the sump was used to convert the sump volume to a stage for each time increment. The computed sump stage hydrographs were then compared to the measured sump stage hydrographs.

3.2.2 Calibration Results

For all the calibration events, the computed sump stage hydrographs using the computed hydrologic subbasin parameters as described in Section 2.3.2 matched the shape of the measured stage hydrographs reasonably well. This was encouraging, and seemed to validate the modeling approach. However, there were differences in the magnitude of the computed and measured sump stage hydrographs. The differences were greater for some sumps and some events than others. Unfortunately, it was found that for a given sump, the results were neither consistently high nor consistently low for different events. This implied that it would be difficult or impossible to improve the calibration for all events by adjusting the subbasin hydrologic parameters, since increasing the runoff may have improved the results for one storm but would have degraded the results for others. From a modeling standpoint, this implies that different hydrologic parameters would be applicable for different storm events. This is often found to be the case in computer modeling of physical phenomena, since models usually cannot account for all of the processes that affect the modeled system. Nevertheless, a systematic approach was used to modify the subbasin hydrologic parameters to determine their effects on the computed results in an attempt to more closely match the magnitude of the measured sump stage hydrographs.

As stated previously, the primary goal of the calibration simulations was to match the timing and magnitude of the peak sump stages for the May 1995 storm event. Using the original computed subbasin parameters, the timing and magnitude of the computed peak sump stage matched the measured peak sump stage extremely well for Record Crossing Sump, but the slope of the receding limb of the computed hydrograph was less than that of the measured hydrograph. This implies that the measured sump level was falling faster than the computed pumping outflow rate. The reason for this is unknown – it is unlikely that the gravity sluice at the Hampton Pumping Plant was operating for this event. Nevertheless, the computed results for the May 1995 event using the computed hydrologic parameters for Record Crossing Sump were judged to be acceptable.

The magnitude of the computed peak stages was less than the measured peak stages for Hampton-Oak Lawn and Able sumps. Therefore, the computed SCS curve numbers for subbasins in the Hampton-Oak Lawn and Able sump watersheds were increased in an attempt to increase the runoff into the sumps, thereby increasing the peak stages. Unfortunately, it was not possible to improve the results much, since the computed SCS curve numbers described in Section 2.3.2.1 were already quite high. The highest possible SCS curve number in HEC-HMS is 99.0, which is generally used to represent a fully impervious surface such as standing water. Even increasing all of the SCS curve numbers to 99.0 for the Hampton-Oak Lawn and Able sump watersheds was not sufficient to increase the computed peak sump stages to the measured peaks for the May 1995 event. The Snyder unit hydrograph parameters were also perturbed in conjunction with the SCS runoff parameters, and no suitable combination of parameters could be found to match the measured peaks or to significantly improve the shape of the computed hydrographs. The original computed hydrologic parameters were judged to yield the best overall match to the measured stage hydrographs. Thus, the selected East Levee watershed "best-fit" parameters for the May 1995 event were the original computed parameters.

It should be noted that it was possible to improve the fit of the May 1995 calibration results by adjusting parameters such as the sump volumes and pump outflow rates. A relatively high degree of confidence was placed in the sump elevation-volume curves, particularly given how well they matched curves from previous studies. It was found that adjusting the pump capacities did not have much effect on the peak stages, since maximum pumping did not begin until right at or just after the peak. However, the overall shape of the hydrographs, particularly on the falling limb, was found to be improved in some cases by <u>increasing</u> the pump capacities at some of the pumping plants. This approach was rejected because it was deemed inappropriate to increase the pumps beyond their rated capacities.

After completing this process for the May 1995 event, the other two calibration events were used as verification events, since any combination of hydrologic parameters that may have improved the calibration results for those events would have been detrimental to the calibration of the May 1995 event. Therefore, the calibration results for these events are presented with the original computed hydrologic parameters.

For the October 2002 event, the computed sump stage hydrographs show remarkably good agreement with measured hydrographs for Able Sump and Hampton-Oak Lawn Sump. For Record Crossing Sump, the computed stages are too high and the shape of the computed hydrograph exhibits too much smoothing. Overall, the calibration results for the October 2002 storm event were judged to be very good.

For the July 2004 event, the shapes of the computed hydrographs matched the measured hydrographs well, but in general, the computed hydrographs were higher than the measured. It is impossible to make any judgment about Able Sump because of the missing data during the peak of the event. Overall, the calibration results for the July 2004 were judged to be acceptable.

The calibration results are presented graphically in Figures 3.1 – 3.9.

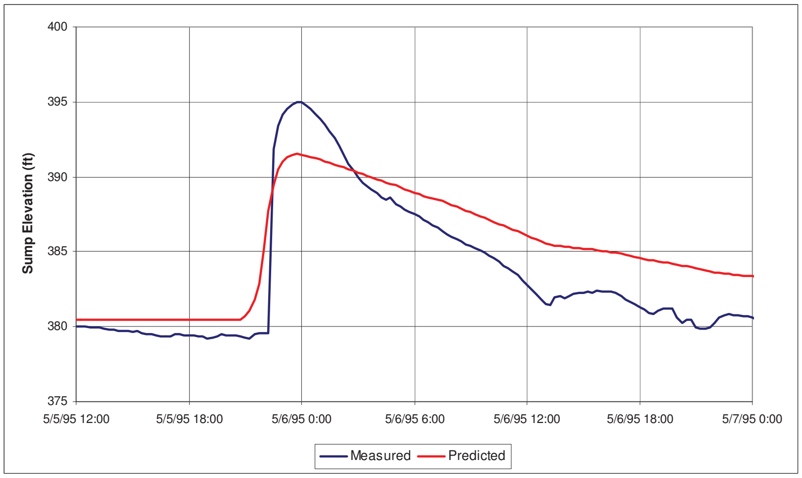

Figure 3.1 - Able Sump Calibration, May 1995 Event

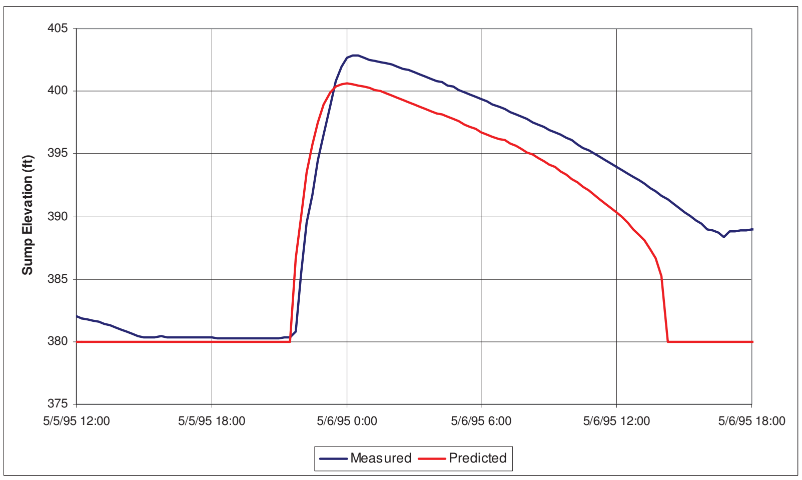

Figure 3.2 - Hampton-Oak Lawn Sump Calibration, May 1995 Event

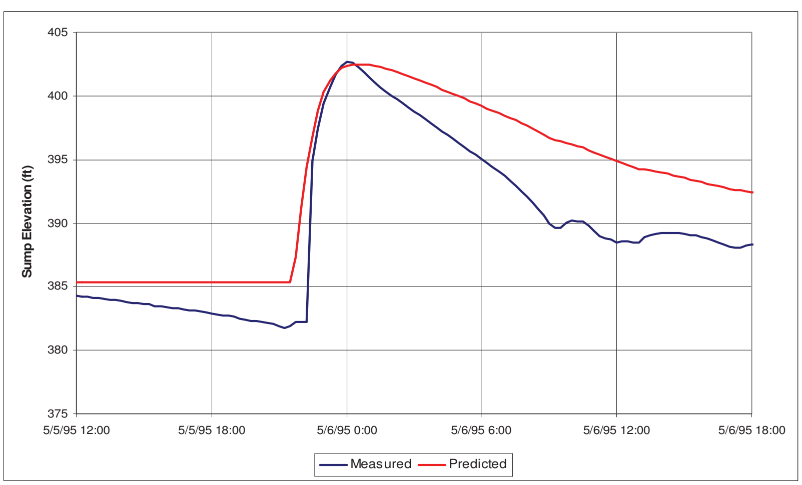

Figure 3.3 - Record Crossing Sump Calibration, May 1995 Event

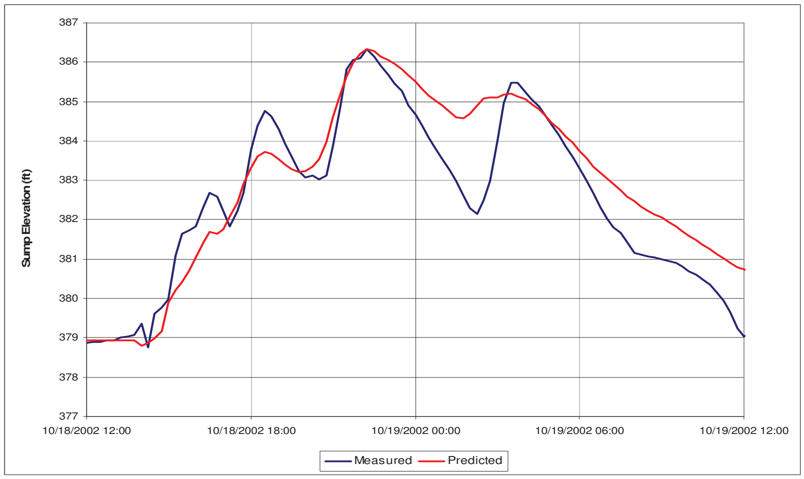

Figure 3.4 - Able Sump Calibration, October 2002 Event

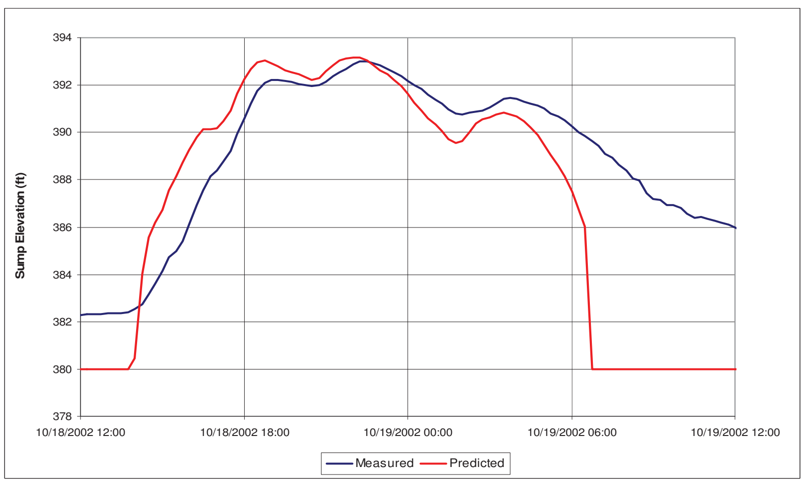

Figure 3.5 - Hampton-Oak Lawn Sump Calibration, October 2002 Event

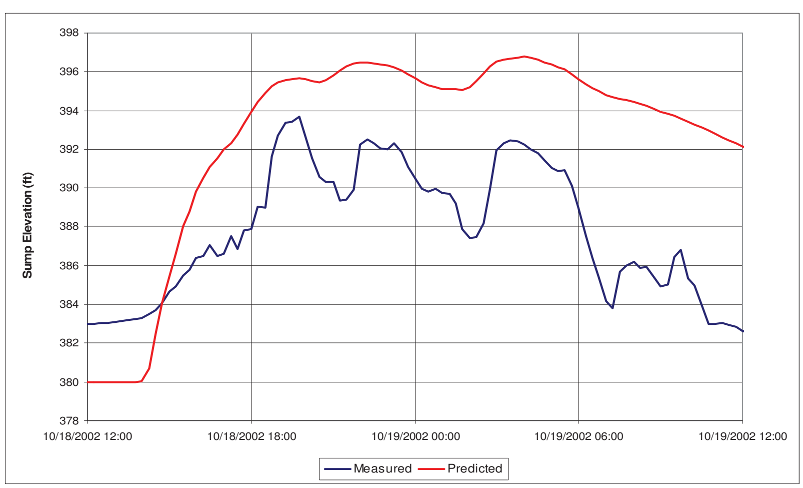

Figure 3.6 - Record Crossing Sump Calibration, October 2002 Event

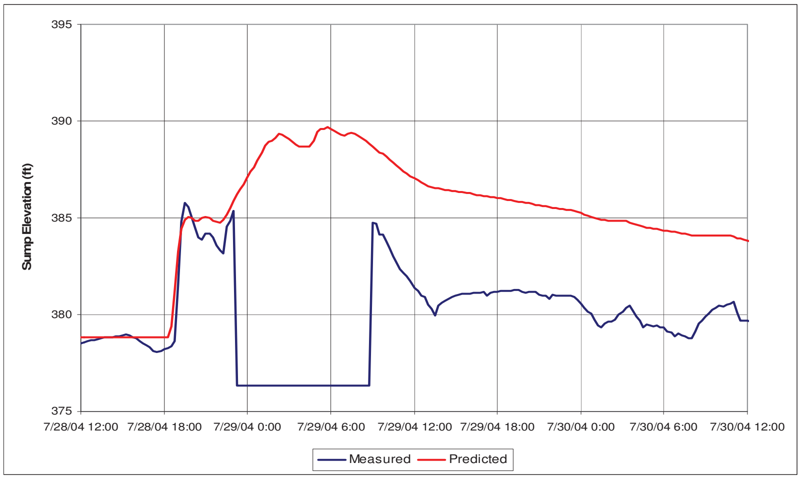

Figure 3.7 - Able Sump Calibration, July 2004 Event

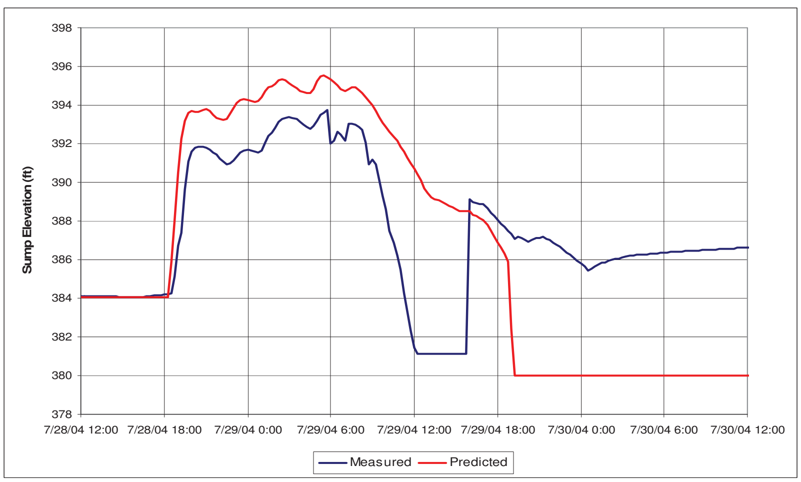

Figure 3.8 - Hampton-Oak Lawn Sump Calibration, July 2004 Event

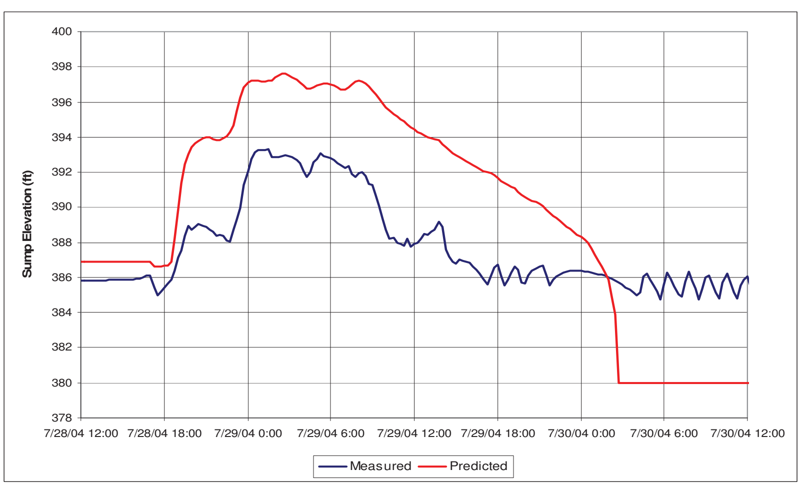

Figure 3.9 - Record Crossing Sump Calibration, July 2004 Event

The calibration results validated the hydrologic modeling approach and methodology developed for this analysis. As discussed previously, it was anticipated that it would be difficult to calibrate to stage hydrographs using the hydrologic model due to the inclusion of routing in the calibration process. Although the modeling approach involves many assumptions and simplifications, the shapes of the computed hydrographs match the measured hydrographs quite well. It would be desirable to match the peaks more accurately, particularly for the May 1995 event, but no reasonable way was found to do so in HEC-HMS within the constraints of the model and engineering judgment.

3.2.3 Able Sump XP-SWMM Model

Because the Able Sump HEC-HMS calibration results for the May 1995 event were unsatisfactory, a model of Able Sump was developed using XP-SWMM. The XP-SWMM model was able to include more of the complex processes that affect the water surface elevation in Able Sump, such as culvert and weir flow between the individual ponds that make up the sump.

As shown in Figure 3.1, the measured Able Sump stage hydrograph at Able Pumping Plant for the May 1995 calibration event exhibited an extremely steep rising limb, rising over 12 feet in a 15-minute period. During the May 1995 event, Able Sump exceeded its 100-year design elevation and overtopped Industrial Boulevard near Able Pumping Plant. Based on the HEC-HMS calibration results for Able Sump, it was apparent that for larger storm events, there are processes affecting the sump stage near the pumping plant that are not represented in the HEC-HMS model. In the HEC-HMS model, the elevation-volume curve for Able Sump was developed as though the entire sump were a single contiguous reservoir. This approach appeared to work well for smaller events, as evidenced by the HEC-HMS model's generally excellent reproduction of the observed stage hydrograph for the October, 2002 calibration event (Figure 3.4). For the May 1995 event, the HEC-HMS model fell short of predicting the observed peak sump stage elevation. Therefore, a more thorough examination was made of Able Sump to determine which factors may have contributed to the rapid rise in sump stage for the May 1995 event.

Able Sump is a system composed of nine separate pond areas divided by streets and highways and the Belleview Pressure Sewer, as shown on Exhibit 2. Box culverts, bridges, and a weir created by the Belleview Pressure Sewer connect the individual ponds. Simulation of the dynamic and complex interaction of the individual ponds is beyond the capabilities of HEC-HMS. Therefore, an XP-SWMM model of Able Sump was developed to investigate the interaction of the individual ponds. The HEC-HMS model with original computed hydrologic parameters was used to compute inflow hydrographs for Able Sump, but the reservoir routing and culvert and weir flow calculations were performed with XP-SWMM.

The same technique described in Section 2.3.3 was used to develop elevation-volume curves for the nine individual Able Sump ponds. As the water level in the ponds rises, weir flow over Industrial Boulevard and Corinth Street will eventually occur between the ponds. The 2001 LiDAR 2-ft contours were used to determine the weir profiles between the ponds. Surveyed culvert sizes and invert elevations were used in the model. The surveyed box culvert sizes include some nonstandard sizes, which is possible since the culverts appear to have been cast in place.

As shown in Exhibit 2, Able Pumping Plant is located in Pond 1. Therefore, the measured sump stage data for calibration events is for Pond 1. Pond 1 receives runoff from a large storm sewer trunk line along Industrial Boulevard. In addition, Pond 1 receives overflow from the Woodall Rodgers and Dallas Branch pressure sewers and runoff from the Town Branch subbasin. Therefore, Pond 1 receives a large volume of flow from some of the most heavily developed portions of the East Levee watershed. The majority of the runoff from the Able Sump watershed is intercepted by Ponds 1, 2, and 3. Ponds 4-9 receive mostly local runoff.

The Belleview Pressure Sewer barrier effectively isolates Ponds 6-9 from the rest of Able Sump. Besides the 42-inch culvert under the Associated Freezers property connecting Ponds 4 and 7, the only exchange between Ponds 6-9 and the rest of Able Sump occurs via weir flow over the Belleview Pressure Sewer. The culvert between Ponds 4 and 7 is approximately 1,850 ft long and has an overall slope of 0.000016 ft/ft from Pond 4 to Pond 7 based on surveyed invert elevations from the 1975 Pumping Plant “A” Interior Drainage Study. The alignment, profile, and condition of the pipe are unknown. It is possible that the pipe has a sag profile to pass underneath the Belleview Pressure Sewer. The capacity of this pipe is too limited to allow significant exchange of flow between Ponds 4 and 7.

The Belleview Pressure Sewer weir between Ponds 5 and 6 overtops at approximately elevation 387.0 ft. Ponds 1-5 and 6-9 are effectively isolated from each other until the weir overtops. This barrier presents a significant operational problem for Able Sump, since the volume in Ponds 6-9 below elevation 387.0 ft is not available for storing the large volume of flow entering the sump downstream of the weir. It is possible that this effect, coupled with possibly inadequately-sized culverts connecting the ponds, was responsible for the extremely rapid rise in Pond 1 for the May 1995 calibration event. It is believed that as Pond 1 filled, the water was not able to flow out into the rest of the sump quickly enough to prevent Pond 1 from exceeding the Able Sump 100-year design elevation.

The Able Sump XP-SWMM model generated markedly better calibration results for the May 1995 event than the HEC-HMS model, as shown in Figure 3.10. Figure 3.11 compares XP-SWMM computed stage hydrographs for selected Able Sump ponds for the May 1995 event. Figure 3.11 shows that the peak pond water surface elevations are lower in the ponds farther away from Pond 1, which implies that the water is unable to back up from Pond 1 into the rest of the sump quickly enough to equalize the water levels in the ponds. This is due to a combination of the barrier created by the Belleview Pressure Sewer and undersized culverts connecting the ponds. Recommendations to address these problems are presented in Chapter 4 of this report. Ponds 1 and 2 achieve the same peak elevation, implying that the culvert between those ponds is not the primary bottleneck in backing water up from Pond 1 to the rest of the sump. This does not necessarily mean that the culvert does not need to be improved, as discussed in Section 4.2.3.

In Figure 3.11, Ponds 6 and 9 reach their peak elevation and do not decrease. This is because the 42-inch culvert connecting Ponds 4 and 7 is not included in the XP-SWMM model. As discussed previously, this culvert is ineffective in allowing Ponds 6-9 to drain.

Because the culvert is not in the model, these ponds have no way to drain because of the Belleview Pressure Sewer barrier. However, this does not affect the validity of computed peak stage in those ponds.

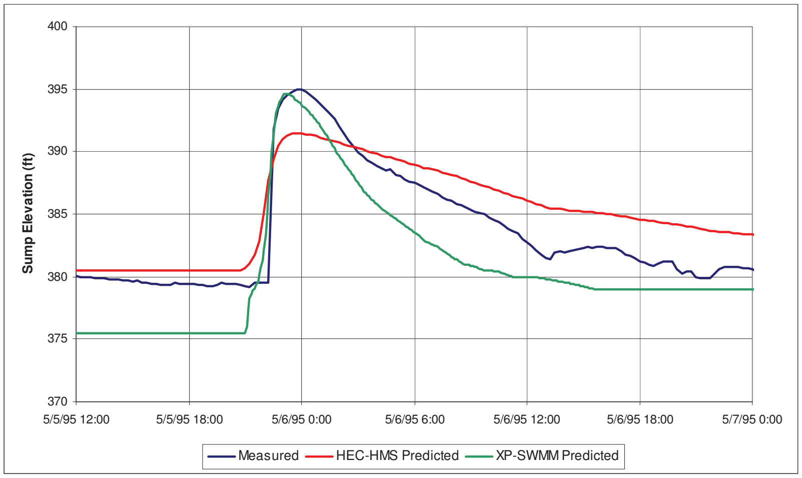

Figure 3.10 - Able Sump Pond 1 XP-SWMM Model Calibration, May 1995 Event

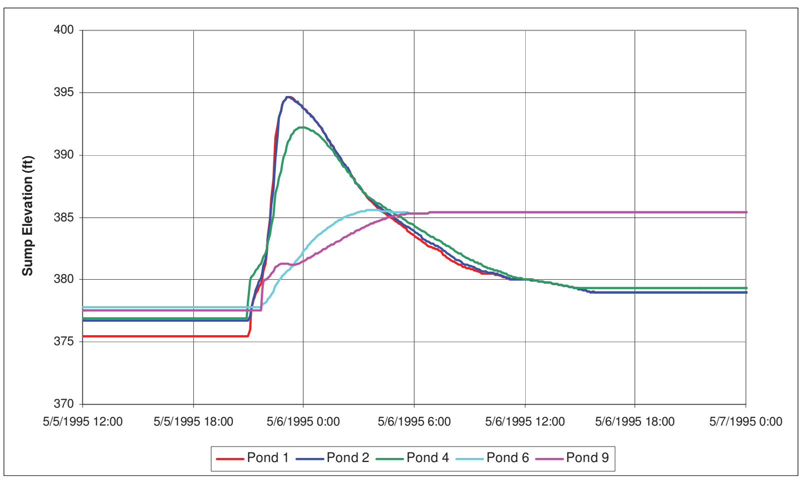

Figure 3.11 - Comparison of XP-SWMM Computed Able Sump Stages for May 1995 Event

3.3 MARCH 18-19, 2006 FLOOD EVENT

On March 18-19, 2006, a major rainfall event caused widespread flooding in the City of Dallas, resulting in substantial property damage and one fatality. Because this event occurred during the ongoing Phase I Interior Levee Drainage Study, heightened emphasis was placed on the qualitative and quantitative analysis of the event and its consequences. Also because of the timing of the event, it was possible to document the flooding with photographs and field observations for incorporation into this report. Measured data from the event were used to verify the computer models that had been previously developed for the East Levee area.

3.3.1 Overview and Chronology of the Flood Event

The first measurable precipitation of the event began around midnight on Saturday, March 18. Several bands of thunderstorms moved across the City from late morning to early evening on Saturday. Typical rainfall totals around the City for the day of Saturday, March 18 were about 1 inch. Typical precipitation intensities during periods of rainfall on Saturday were about 0.2 to 0.3 inches/hour.

In the early morning hours of Sunday, March 19, heavier thunderstorms moved into Dallas. The rain continued for most of the day, with many gages across the City reporting nearly continuous rainfall for 24 hours. By noon on Sunday, typical 36-hour rainfall totals around the city were approaching 4 inches. Many gages experienced peak rainfall intensities greater than 0.5 inches/hour on Sunday morning.

On Sunday afternoon between 1:00 and 4:00 P.M., the heaviest rains hit the area. Some gages recorded peak rainfall intensities greater than 2.5 inches/hour. Fortunately, these areas were isolated. Peak rainfall intensities greater than 1.0 inches/hour were widespread throughout the City during the afternoon of Sunday, March 19.

The rain continued throughout the afternoon and evening of Sunday, March 19. By the time the rain stopped in the early morning hours of March 20, over 10 inches of rain had fallen in isolated areas of the City. The average 48-hour rainfall total across the City was nearly 7 inches.

Figure 3.12 shows a typical incremental precipitation hyetograph and cumulative precipitation curve for the event. The gage data in Figure 3.12 are from ALERT Gage 671, located at the Public Works and Transportation Facility at 2255 Irving Boulevard near Baker Pumping Plant. In Figure 3.12, the heavy precipitation that occurred on the afternoon of Sunday, March 19 is readily apparent.

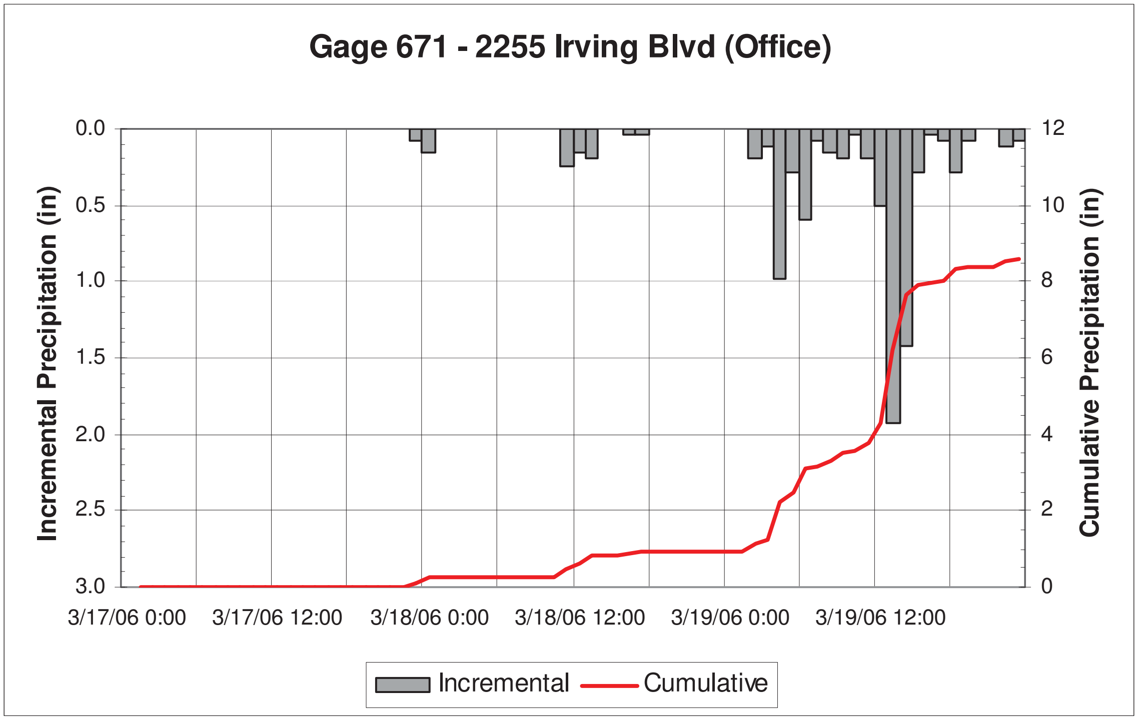

Figure 3.12 - Incremental and Cumulative Precipitation at the Public Works and Transportation Office at Baker Pumping Plant

The storms followed a typical southwest-to-northeast track across the City. The area of maximum rainfall in the City generally consisted of a swath from the Loop 12/Interstate 30 interchange to the southwest to White Rock Lake to the northeast, lying between Love Field and the downtown Central Business District.

The highest rainfall totals in the City occurred near White Rock Lake, and numerous homes were flooded near the lake or its tributaries. A retaining wall downstream of the White Rock Lake spillway was undermined and damaged during the flood, as shown in Photo 3.1.

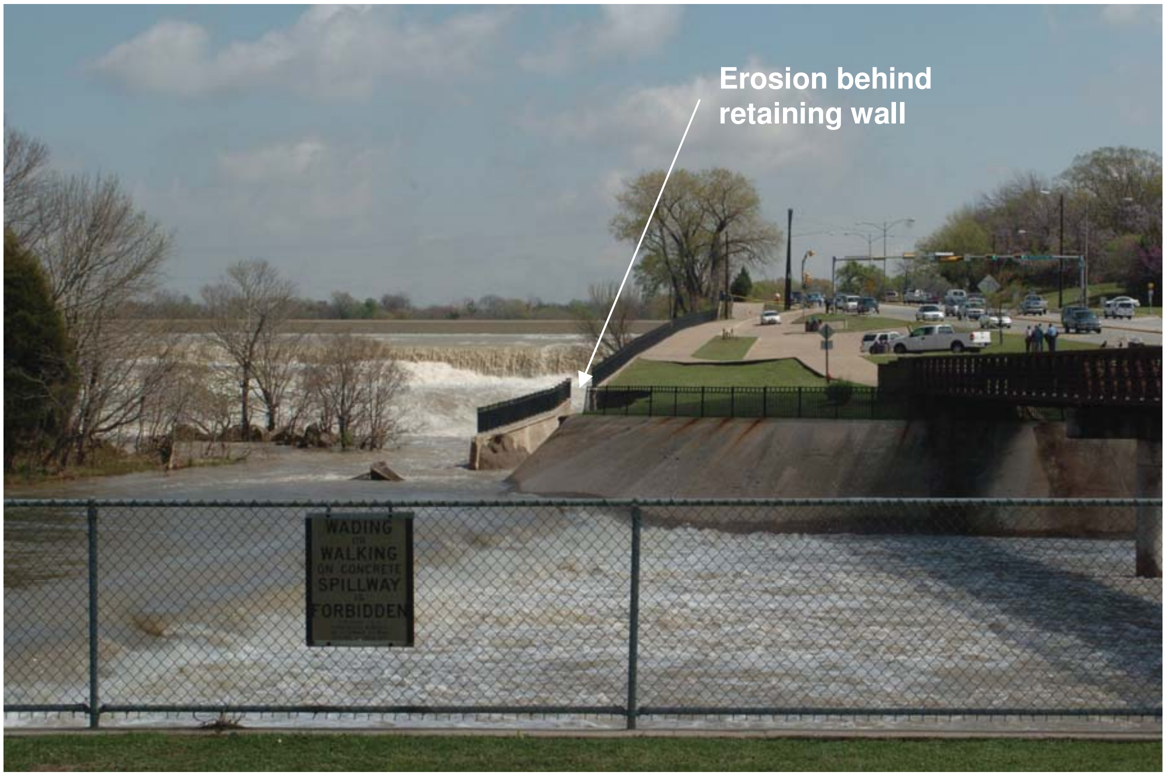

Photo 3.1 - Damage to Retaining Wall at White Rock Lake Spillway - March 20, 2006 (source: Dallas Morning News)

Severe flooding also occurred on Turtle Creek, inundating several streets and intersections. One fatality occurred when a woman was swept downstream by Turtle Creek as she attempted to escape her flooded car at the intersection of Wycliff Avenue and Turtle Creek Boulevard.

Photo 3.2 shows the overflow spillway at the Turtle Creek Pressure Sewer inlet. In Photo 3.2, the pressure sewer inlet is in the main channel of Turtle Creek at the rear of the photo, and all the flow passing over the spillway flows down the Turtle Creek channel to Hampton-Oak Lawn Sump.

Photos 3.3 and 3.4 show Turtle Creek near the intersection of Turtle Creek Boulevard and Hall Street. There is an in-channel spillway on Turtle Creek just upstream of this intersection. Both of these photos were taken in the late afternoon of Sunday March 19, after the peak of the flood had passed. Photo 3.3 shows the flow over and around the in-channel spillway. Photo 3.4 shows Turtle Creek passing under Hall Street. Hall Street and the Turtle Creek Boulevard/Hall Street intersection are flooded in Photo 3.4, but earlier during the peak of the event Hall Street was overtopped by Turtle Creek.

Photo 3.2 - Turtle Creek at Pressure Sewer Inlet/Spillway - March 19, 2006


Photo 3.3 - Turtle Creek Spillway Upstream of Hall Street - March 19, 2006

Photo 3.4 - Turtle Creek at Turtle Creek Boulevard and Hall Street - March 19, 2006

The Dallas Police and Fire-Rescue Departments responded to hundreds of emergency rescue calls from stranded motorists and residents on March 19. A dramatic motorist rescue is shown in Photo 3.5.

Photo 3.5 - Dallas Police Motorist Rescue on East Mockingbird Lane Near White Rock Lake - March 19, 2006 (source: Dallas Morning News)

Significant flooding occurred in both the East and West Levee areas due to high sump elevations. In the East Levee area, the worst flooding was in the Record Crossing and Hampton-Oak Lawn Sumps, particularly along the Stemmons Freeway corridor.

In the Record Crossing sump area, the Stemmons Freeway service roads from Commonwealth Drive to Inwood Road were inundated. Typical flooding along Stemmons Freeway is shown in Photo 3.6. Inwood Road at the TRE overpass was flooded to a depth of at least 3 feet. Photo 3.7, taken from an office building at Stemmons Freeway and Inwood Road, shows the flooding at the Stemmons Freeway Service Roads and Inwood Road.

Photo 3.8 shows Record Crossing Sump just south of Mockingbird Lane during the flood event. The sump channel is at bankfull stage, but the sump was not overtopped at this location.

Photo 3.6 - Northbound Stemmons Freeway (IH35E) Service Road at Commonwealth Drive, Record Crossing Sump Area - March 19, 2006 (source: Dallas Morning News)

Photo 3.7 - Inwood Road at Stemmons Freeway (IH35E), Record Crossing Sump Area - March 19, 2006

Photo 3.8 - Record Crossing Sump near Mockingbird Lane - March 19, 2006

In the Hampton-Oak Lawn sump area, numerous parking lots and/or structures along Stemmons Freeway were flooded by high sump elevations, including the Renaissance Hotel, Market Hall, Dallas World Trade Center, and the Hilton Anatole Hotel. The Stemmons Freeway service roads from Market Center Boulevard to Oak Lawn Avenue were inundated, and Oak Lawn Avenue under the Harry Hines Boulevard overpass was flooded. The Dallas North Tollway south of Mockingbird Lane was also closed due to flooding on Sunday afternoon. The Design District also experienced street flooding and some structure flooding, possibly due to overflow from Hampton-Oak Lawn Sump.

Photo 3.9 shows Market Center Boulevard looking south towards Stemmons Freeway on the afternoon of March 19. Photo 3.10 shows flooding in the Market Hall parking lot.

Photo 3.9 - Market Center Boulevard North of Stemmons Freeway (IH35E), Hampton-Oak Lawn Sump Area - March 19, 2006

Photo 3.10 - Market Hall Parking Lot, Hampton-Oak Lawn Sump Area - March 19, 2006 (source: Dallas Morning News)

The worst flooding in the Able Sump area was in the R.L. Thornton Freeway "canyon," which was flooded to a depth of up to 3 feet, as shown in Photo 3.11. In Photo 3.11, note the City of Dallas police cruiser in the left side of the photo. This flooding was primarily due to overflow from the Mill Creek drainage system, which experienced some of the heaviest rainfall in the City.

Photo 3.11 - R.L. Thornton Freeway (IH30) "Canyon" at South St. Paul Street, Able Sump Area - March 19, 2006 (source: Dallas Morning News)

3.3.2 Precipitation Statistical Analysis and Mapping

The precipitation event of March 18-19, 2006 was significant not only in its magnitude and impact to the City of Dallas; it was also an important opportunity to observe how the City's interior drainage facilities functioned during a large flood event. To enhance the understanding of the event, Carter & Burgess, Inc. developed statistical analyses and mapping of the precipitation event based on measured data from the City's network of ALERT precipitation gages. Hourly incremental precipitation for the gages is available in real time from the City of Dallas Flood Control District website. These data were downloaded for all of the available gages across the City. For this event, 58 gages were active and had usable data.

The two primary goals of the precipitation analyses were as follows:

- to develop a graphical depictions of the magnitude and spatial variation of the precipitation event
- to determine the frequencies or exceedence probabilities associated with the event for various storm durations

To accomplish these goals, the hourly precipitation data for all 58 gages for the 72-hour period beginning at midnight on March 17, 2006 were downloaded, reformatted, and imported into a spreadsheet. The maximum precipitation totals for the 1-, 2-, 3-, 6-, 12-, 24-, 36-, 48-, and 72-hour durations were then computed for each gage. For a given duration, this was accomplished by calculating a rolling sum of the total rainfall in all contiguous periods equal to the duration, then selecting the maximum value. For example, the maximum precipitation total at a gage for the 3-hour duration is not the sum of the three maximum hourly incremental values, but rather the maximum total of any contiguous 3-hour period during the event. With this methodology, the contiguous period associated with a duration is not necessarily the same from gage to gage.

The next step in the analysis was to determine the frequency associated with the total precipitation for each duration at each gage. This was done by comparing the precipitation totals with the NCTCOG iSWM intensity-duration-frequency (IDF) curves for Dallas County. The iSWM IDF curves were chosen for this analysis because they incorporate recent Texas precipitation depth-duration-frequency research developed by the USGS in cooperation with TxDOT, and because they are tabulated on a county-by-county basis. The iSWM IDF curves are similar to the IDF curves obtained from the traditional TP-40/Hydro-35 sources. The iSWM data are limited to durations of 24 hours or less; therefore, precipitation data for the 36-, 48-, and 72-hour durations were extrapolated. The precipitation totals at each gage for each duration were then interpolated against the iSWM tabular IDF curves to determine the frequency associated with the duration. Thus, two discrete calculated data points were associated with each gage for each duration – a total rainfall depth and a frequency.

Tables 3.1 and 3.2 summarize the maximum precipitation and computed frequency for the ALERT sensors for the period March 17-20, 2006. The ALERT sensors in Tables 3.1 and 3.2 are grouped according to the drainage basins associated with the interior drainage system.

Table 3.1 - Maximum Depth-Duration Table, March 17-20, 2006

GAGE	1-hr MAX PRECIP (in)	2-hr MAX PRECIP (in)	3-hr MAX PRECIP (in)	6-hr MAX PRECIP (in)	12-hr MAX PRECIP (in)	24-hr MAX PRECIP (in)	36-hr MAX PRECIP (in)	48-hr MAX PRECIP (in)	72-hr TOTAL PRECIP (in)
EAST LEVEE SUMP DRAINAGE AREA									
155	1.73	3.11	3.46	3.86	5.88	6.84	7.52	7.88	7.92
195	1.46	2.80	3.19	3.67	5.87	6.87	7.51	7.75	7.83
6135	1.26	2.36	3.23	3.79	5.29	6.49	7.09	7.45	7.61
6715	1.93	3.35	3.86	4.38	6.67	7.67	8.27	8.51	8.59
6775	1.93	3.47	4.06	4.61	6.90	7.94	8.74	9.18	9.26
6835	1.93	2.80	3.15	3.66	5.87	6.71	7.47	7.91	7.95
6855	2.32	3.70	4.13	4.57	5.98	6.78	7.42	7.82	7.82
6895	2.05	3.51	3.86	4.34	6.83	7.75	8.43	8.79	8.83
7725	1.22	1.73	1.97	2.49	4.26	5.33	6.01	6.25	6.29
7735	1.72	2.48	2.88	3.36	5.16	6.32	7.00	7.24	7.32
WEST LEVEE SUMP DRAINAGE AREA									
5045	1.14	2.12	3.26	3.78	5.21	6.53	7.17	7.65	7.77
5055	0.94	1.85	2.72	3.24	4.47	5.43	5.91	6.27	6.39
5235	1.10	1.81	2.52	2.68	3.39	3.75	4.35	4.67	4.67
5295	1.34	2.64	3.74	4.37	5.88	6.96	7.52	7.92	8.04
5515	2.44	3.70	4.96	5.47	7.01	7.89	8.53	8.93	9.05
6235	1.34	2.36	3.30	3.94	5.52	6.68	7.32	7.72	7.92
6355	1.61	3.15	3.98	4.46	5.72	6.72	7.20	7.48	7.56
6475	2.52	3.78	4.37	4.88	6.46	7.10	7.58	7.86	7.94
MILL CREEK DRAINAGE AREA									
1535	1.61	2.52	2.83	3.26	4.76	5.56	5.88	6.12	6.20
1655	2.32	4.25	5.08	5.75	7.75	8.82	9.58	9.90	10.02
1855	1.89	3.43	4.18	4.81	6.42	7.53	8.13	8.49	8.57
1955	2.09	3.78	4.25	4.65	6.46	7.34	8.18	8.38	8.50
2255	2.09	3.78	4.65	5.32	7.21	8.36	9.08	9.44	9.52
WHITE ROCK LAKE VICINITY									
915	2.80	4.97	5.32	6.15	8.71	9.82	10.46	10.74	10.86
935	2.01	3.19	3.54	4.21	6.73	7.73	8.41	8.65	8.73
1155	2.83	4.64	5.11	5.78	8.26	9.45	10.05	10.41	10.53
1235	2.56	4.73	5.12	5.95	8.39	9.23	9.87	10.15	10.27
1295	1.46	2.40	2.56	2.99	4.37	5.48	5.96	6.04	6.08
1515	1.50	2.64	3.07	3.59	5.48	6.51	7.03	7.19	7.27
OTHER AREAS									
345	0.55	0.79	0.99	1.74	2.18	3.41	4.21	4.25	4.33
1055	0.79	1.22	1.42	2.25	2.89	4.20	4.92	5.00	5.04
1075	0.51	0.79	0.99	1.86	2.34	3.81	4.65	4.77	4.85
1095	0.55	0.67	0.91	1.65	2.01	3.56	4.44	4.52	4.60
1715	1.57	3.11	3.66	4.49	6.43	7.81	8.49	8.81	8.89
1755	1.65	2.95	3.58	4.21	6.61	8.00	8.64	8.96	9.00
2055	0.87	1.70	2.05	2.65	4.02	5.33	5.93	6.33	6.41
2535	0.64	0.72	0.84	1.52	3.00	4.28	4.84	5.28	5.36
2555	0.28	0.44	0.52	0.88	1.48	2.12	2.76	3.12	3.16
2775	0.79	0.95	1.26	2.05	3.39	5.01	5.65	6.01	6.09
3055	0.87	1.50	1.97	2.41	3.51	4.79	5.47	5.87	5.95
3075	0.67	1.26	1.73	2.17	3.16	4.16	4.68	5.12	5.16
3775	0.71	0.94	1.30	2.01	2.95	4.77	5.53	5.93	5.97
3975	0.51	0.98	1.14	1.85	2.95	4.34	5.02	5.46	5.54
4135	1.10	1.65	2.04	2.44	3.27	4.11	4.71	5.19	5.31
4155	0.51	0.90	1.21	1.77	2.91	4.19	4.95	5.59	5.75
4515	1.06	1.61	2.04	2.44	3.28	4.32	4.84	5.44	5.56
4535	0.52	1.00	1.36	1.96	3.08	4.44	5.04	5.56	5.68
4555	0.79	1.54	2.21	2.61	3.29	3.73	4.33	4.65	4.73
4855	0.88	1.48	2.28	2.64	3.76	4.84	5.32	5.80	5.84
5535	0.91	1.54	2.41	2.97	3.99	4.87	5.43	5.95	6.15
7035	0.59	0.87	1.03	1.51	2.37	3.40	3.96	4.04	4.08
7355	0.87	1.34	1.77	2.43	3.66	4.76	5.40	5.40	5.48
7455	0.98	1.41	1.69	2.47	3.74	5.39	6.31	6.43	6.47
7535	0.63	0.94	1.14	1.62	2.72	3.64	4.12	4.24	4.32
7555	0.59	0.87	1.03	1.59	2.33	3.44	4.16	4.24	4.28
7755	1.30	1.77	2.01	2.53	3.95	5.06	5.66	5.78	5.86
7775	0.98	1.49	1.77	2.44	3.35	4.69	5.41	5.53	5.57
7955	0.43	0.67	0.91	1.58	2.02	3.37	4.32	4.36	4.40

Table 3.2 - Duration-Frequency Table, March 17-20, 2006

GAGE	1-hr FREQ (year)	2-hr FREQ (year)	3-hr FREQ (year)	6-hr FREQ (year)	12-hr FREQ (year)	24-hr FREQ (year)	36-hr FREQ (year)	48-hr FREQ (year)	72-hr FREQ (year)
EAST LEVEE SUMP DRAINAGE AREA									
155	2	9	9	7	22	21	21	19	15
195	1	5	6	6	22	22	21	18	14
6135	< 1 yr	3	7	6	15	18	16	16	13
6715	3	13	15	12	41	35	31	24	20
6775	3	16	20	16	48	40	39	34	24
6835	3	5	6	5	22	20	20	19	15
6855	6	22	21	15	23	21	20	19	14
6895	4	17	15	11	46	36	34	28	21
7725	< 1 yr	1	1	2	5	8	8	8	6
7735	2	4	4	4	13	16	15	14	11
WEST LEVEE SUMP DRAINAGE AREA									
5045	< 1 yr	2	7	6	14	18	17	17	14
5055	< 1 yr	1	4	4	7	8	7	8	7
5235	< 1 yr	1	3	2	3	2	3	3	2
5295	< 1 yr	4	13	12	22	22	21	19	16
5515	8	22	55	36	52	39	36	30	23
6235	< 1 yr	3	7	8	17	20	19	18	15
6355	2	9	18	13	20	20	17	16	12
6475	9	24	27	20	35	24	21	19	15
MILL CREEK DRAINAGE AREA									
1535	2	4	4	4	9	9	7	7	6
1655	6	45	64	45	87	67	60	47	33
1855	3	15	22	19	34	32	29	24	20
1955	4	24	24	16	35	28	30	23	19
2255	4	24	39	31	62	49	45	39	27
WHITE ROCK LAKE VICINITY									
915	16	123	83	66	189	122	102	73	42
935	4	10	9	10	43	36	34	25	21
1155	17	76	67	47	129	94	82	61	39
1235	9	84	67	54	147	85	74	52	36
1295	1	3	3	3	6	9	8	6	5
1515	1	4	5	5	17	18	16	14	10
OTHER AREAS									
345	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	3	2	2
1055	< 1 yr	< 1 yr	< 1 yr	1	2	3	4	4	3
1075	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	3	3	3	3
1095	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	3	3	2
1715	2	9	11	14	34	38	35	28	22
1755	2	7	10	10	40	42	37	31	22
2055	< 1 yr	1	1	2	4	8	7	8	7
2535	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	4	4	4	4
2555	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	1	< 1 yr
2775	< 1 yr	< 1 yr	< 1 yr	< 1 yr	3	6	6	6	5
3055	< 1 yr	< 1 yr	1	2	3	5	5	6	5
3075	< 1 yr	< 1 yr	< 1 yr	1	2	3	3	4	3
3775	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	5	6	6	5
3975	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	4	4	4	4
4135	< 1 yr	< 1 yr	1	2	2	3	4	4	4
4155	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	3	4	5	5
4515	< 1 yr	< 1 yr	1	2	2	4	4	4	4
4535	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	4	4	5	4
4555	< 1 yr	< 1 yr	2	2	2	2	3	3	2
4855	< 1 yr	< 1 yr	2	2	4	5	5	5	5
5535	< 1 yr	< 1 yr	2	3	4	5	5	6	6
7035	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	2	2	2
7355	< 1 yr	< 1 yr	< 1 yr	2	3	5	5	4	4
7455	< 1 yr	< 1 yr	< 1 yr	2	4	8	9	8	7
7535	< 1 yr	< 1 yr	< 1 yr	< 1 yr	1	2	2	2	2
7555	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	2	2	2
7755	< 1 yr	1	1	2	4	6	6	5	5
7775	< 1 yr	< 1 yr	< 1 yr	2	3	5	5	5	4
7955	< 1 yr	< 1 yr	< 1 yr	< 1 yr	< 1 yr	2	3	2	2

LEGEND:

10-year or less	10-year to 25-year	Over 25-year

GIS mapping was used to depict the spatial variation of rainfall across the city based on the rain gage data. Because the map coordinates of the rain gages are known, terrain modeling techniques were used to develop contour maps of rainfall depth and frequency for selected durations. This was done by using the rainfall depth or calculated frequency as the z-coordinate or "elevation" associated with the rain gage points. Then the contour maps were color-shaded between contour lines, such that a single color represents a range of rainfall depths between contour lines. The maps of 12- and 24-hour duration depth and frequency were deemed most significant, and these maps and associated data summary tables were provided to City of Dallas Public Works staff to assist in decision-making and assessing the impacts of the precipitation event. These maps are shown in Exhibits 13 – 16.

3.3.3 Aerial Reconnaissance

On the afternoon of Tuesday March 21, 2006, Carter & Burgess, Inc. used a helicopter to perform aerial reconnaissance to document the remaining extent of sump and Dallas Floodway flooding. By this time, water surface elevations in the Dallas Floodway had receded substantially compared to the day before. No flooding was observed on the East Levee side, and the East Levee sumps were apparently at normal levels. However, elevated sump stages were still apparent on the West Levee side at this time, implying that the West Levee sumps were not able to be pumped down as rapidly as the East Levee sumps. Selected aerial reconnaissance photos are shown in Photos 3.12 – 3.17.

Photo 3.12 - Dallas Floodway, Looking East - March 21, 2006

Photo 3.13 - Dallas Floodway, Looking South - March 21, 2006

Photo 3.14 - Able Sump, Looking South - March 21, 2006

Photo 3.15 - Hampton-Oak Lawn Sump, Looking Northwest - March 21, 2006

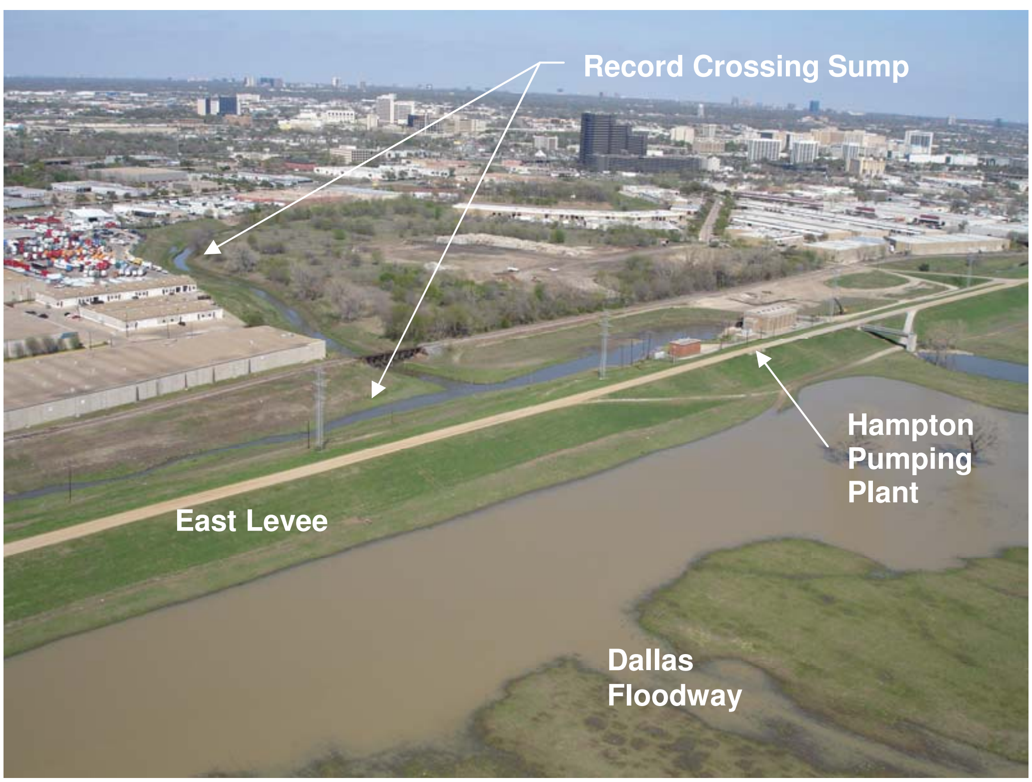

Photo 3.16 - Record Crossing Sump, Looking Northeast - March 21, 2006

Photo 3.17 - Nobles Branch Sump, Looking North - March 21, 2006

3.3.4 Hydrologic Model Verification

This significant flood event provided an opportunity to verify the hydrologic models developed for the Interior Levee Drainage Study. Verification is the process of executing a model with established parameters and comparing the model results to observed data.

Measured sump stage hydrographs and pumping records for the East Levee sumps for this precipitation event were supplied by the City of Dallas Flood Control District. The maximum measured sump elevations and the 2001 LiDAR contours were used to prepare the inundation map for the event shown in Exhibit 17.

Using these data and the precipitation data described in Section 3.3.2, verification simulations were executed using the East Levee HEC-HMS model and Able Sump XP-SWMM models described in Chapter 3. The methodology discussed in Section 3.2.1 was used for the verification simulations.

Figures 3.13 – 3.15 show measured sump stage and pump outfall hydrographs for the Able, Hampton-Oak Lawn, and Record Crossing Sumps for the period March 18-24, 2006. These figures show that Able Sump and Hampton-Oak Lawn Sump exceeded their 100-year design elevations for this event. It is also apparent that periodic pumping continued for several days after the rainfall event.

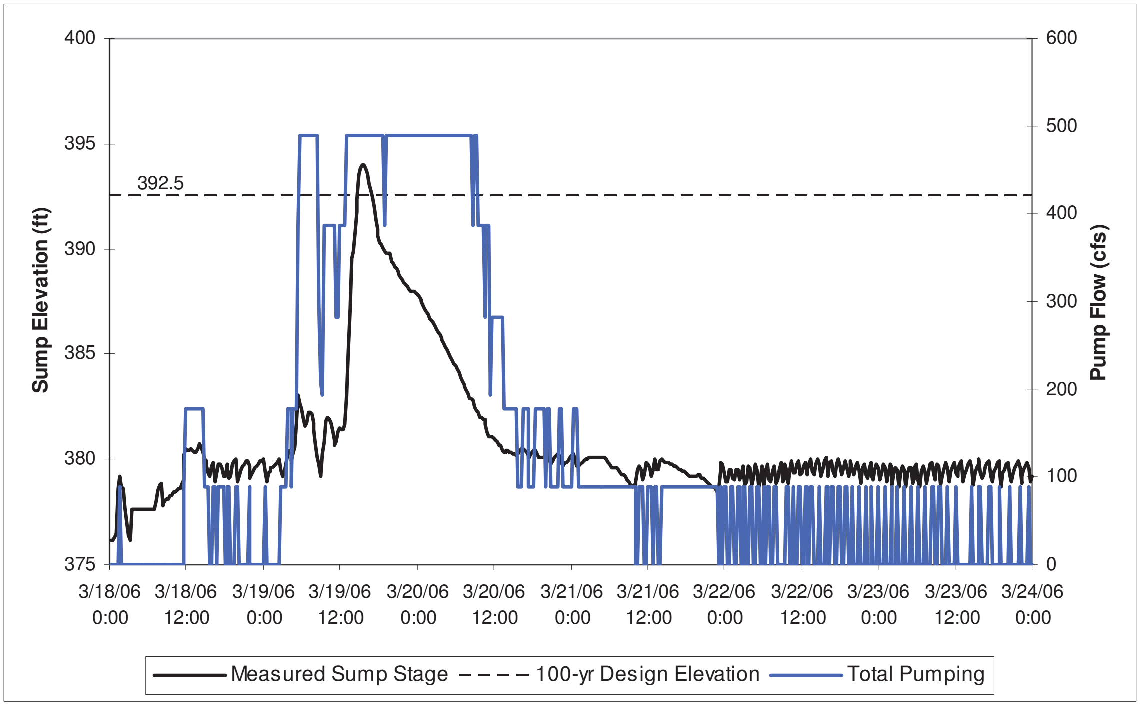

Figure 3.13 - Able Sump Stage and Pump Outflow Hydrographs, March 18-24, 2006

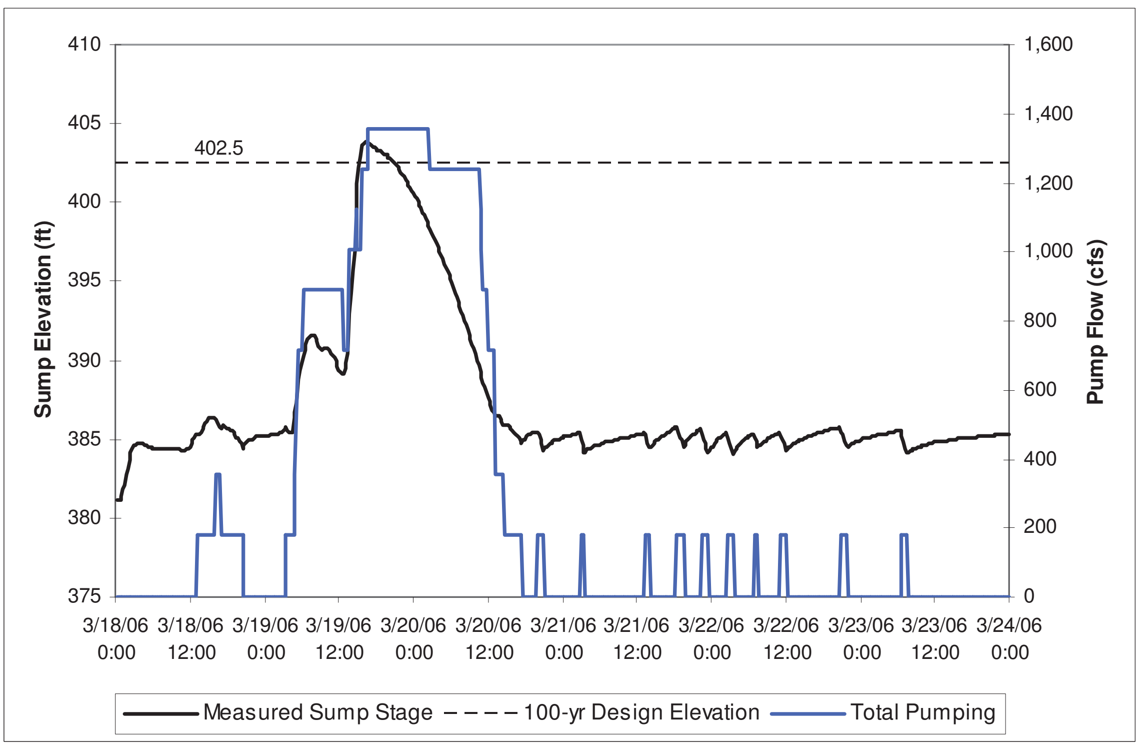

Figure 3.14 - Hampton-Oak Lawn Sump Stage and Pump Outflow Hydrographs, March 18-24, 2006

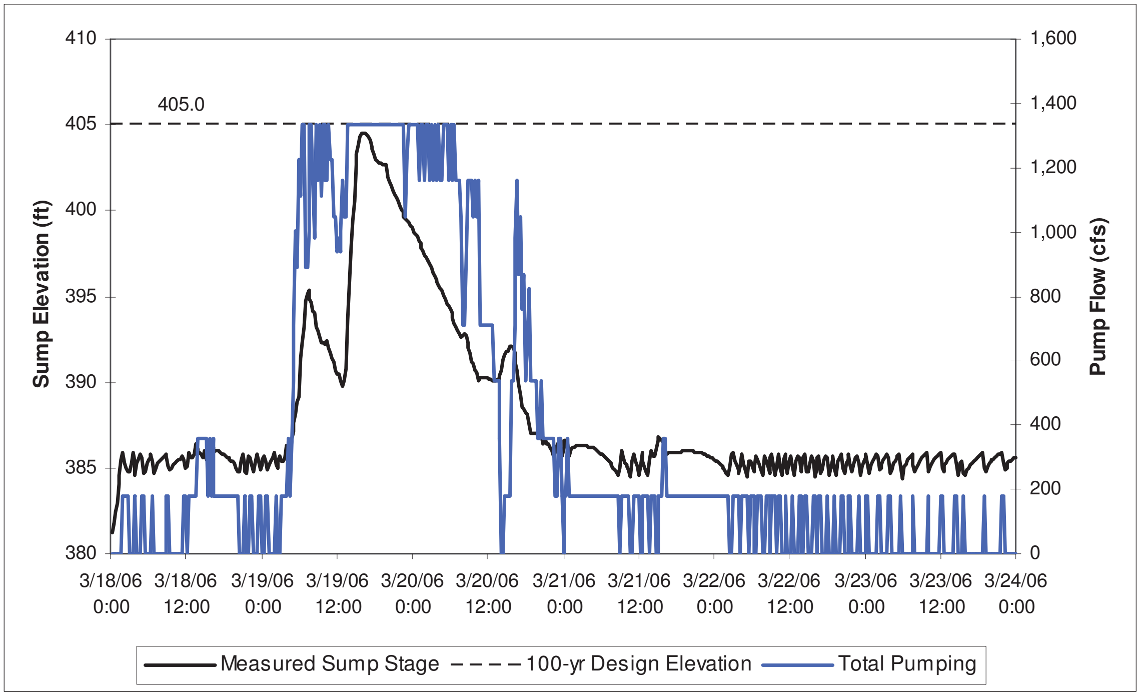

Figure 3.15 - Record Crossing Sump Stage and Pump Outflow Hydrographs, March 18-24, 2006

Figures 3.16 – 3.18 show the results of the verification simulations for this event. For Able Sump, the results shown are from the XP-SWMM model described in Section 3.2.3. For Hampton-Oak Lawn and Record Crossing Sumps, the results come from the HEC-HMS model using the same model parameters used to generate the results presented in Section 3.2.2. Once again, the predicted stage hydrographs show very good agreement with the measured stage hydrographs. These model results verify the application of the models developed for the East Levee Interior Drainage Study for major rainfall events, and lends credibility to their application to hypothetical events such as the 100-year design storm.

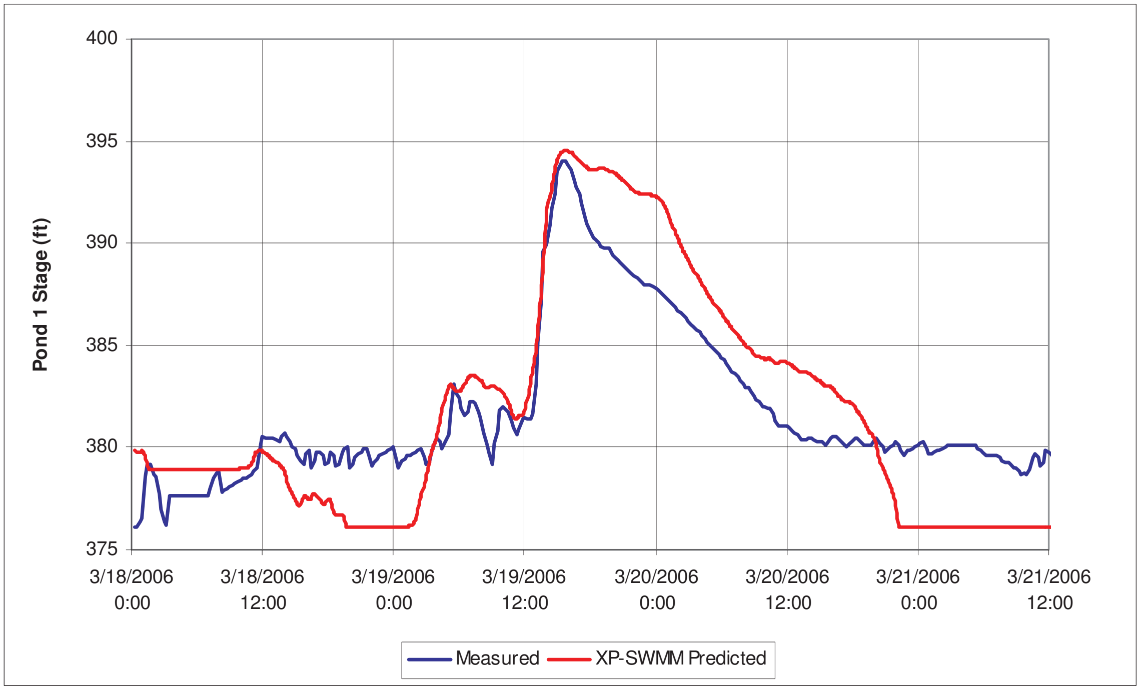

Figure 3.16 - Able Sump Verification, March 2006 Event

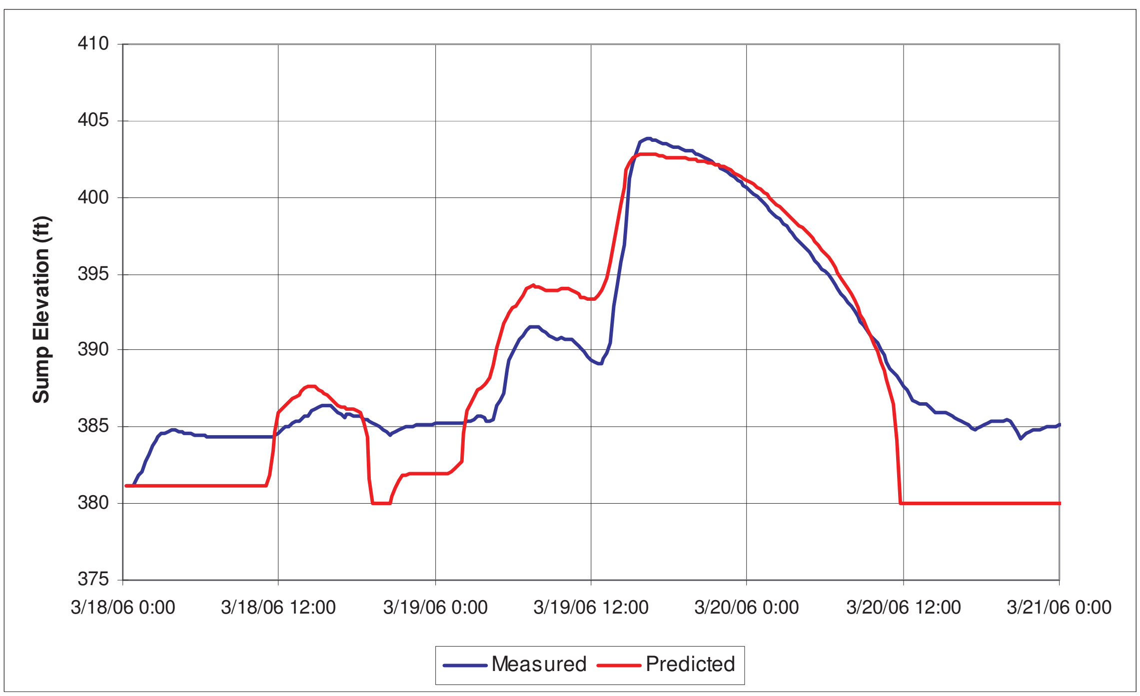

Figure 3.17 - Hampton-Oak Lawn Sump Verification, March 2006 Event

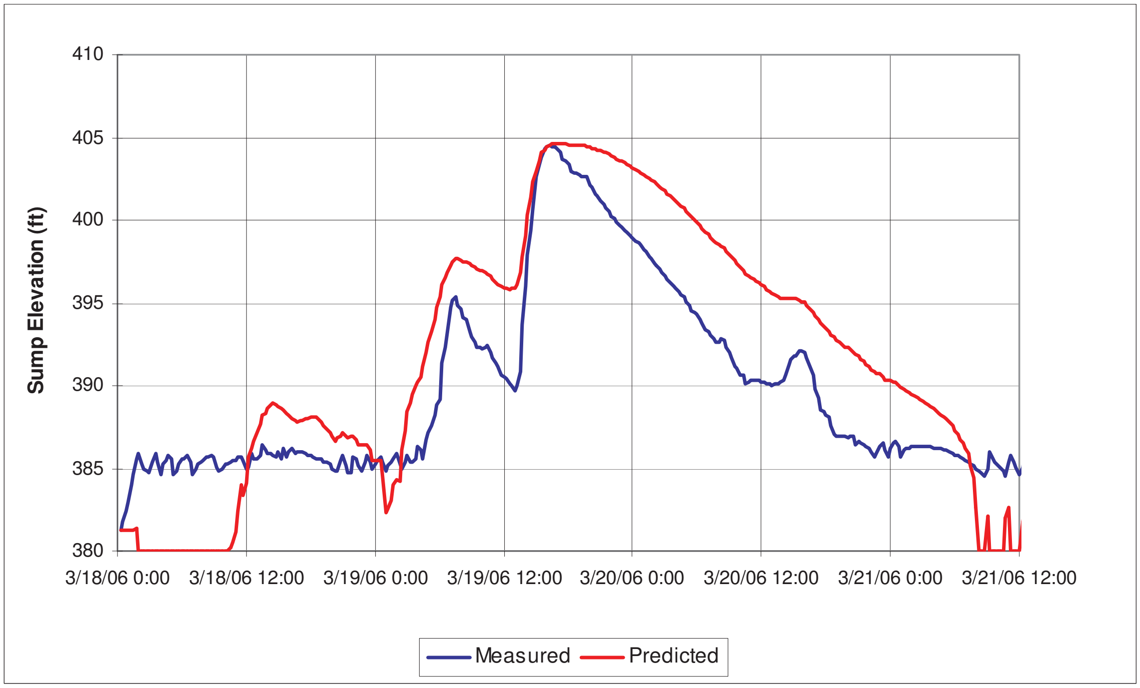

Figure 3.18 - Record Crossing Sump Verification, March 2006 Event

3.3.5 High Water Marks and Finished Floor Elevation Surveys

On Wednesday March 22, Carter & Burgess, Inc. mobilized a survey crew to survey the elevation of high water marks (debris lines) near the East and West Levee sumps. Using GPS, the crew was able to survey high water marks near the sumps while the debris lines were still intact. When the surveyed debris line elevations were compared with the maximum sump elevations from the level sensors at the pumping plants, some differences were observed. For example, the measured debris line elevations in Able Sump are consistently several feet lower than the maximum measured sump elevation at the Able Pumping Plant level sensor. In all cases, the measured maximum sump elevations from the level sensors at the pumping plants were assumed to be more reliable indicators of the maximum sump levels than the surveyed debris lines. At one location, two surveyed high water marks near one another differed by as much as 6 inches. These examples illustrate the unreliability associated with debris lines and high water marks as indicators of maximum water surface elevations.

The City of Dallas contracted with Carter & Burgess, Inc. to survey high water marks associated with the March 18-19 storm event and finished floor elevations of structures which may have been impacted by sump flooding in the East and West Levee sump areas. In the days following the storm event, City of Dallas departments such as Housing and Code Compliance as well as relief agencies such as the Red Cross performed preliminary inspections of flood-affected areas and developed databases of possible flood-damaged structures. Using these databases, the City of Dallas Housing Department compiled a master database of addresses of possible flood-affected structures. The Housing Department provided the database to Carter & Burgess, Inc. for use in the Interior Levee Drainage Study. The database was condensed to those addresses which might potentially have been affected by high sump elevations. The finished floor elevations of the structures at these addresses were surveyed along with any visible high water mark. The survey crew interviewed residents of the structures whenever possible to assess the extent of structure flooding. Because the surveys were performed in the weeks following the flood event, it is probable that some high water marks were obliterated by the time of the survey. Nevertheless, the surveyed finished floor elevations provided an important comparison to the estimated finished floor elevations from the Fort Worth District economic model database. For the East Levee, only finished floors in the Record Crossing and Hampton-Oak Lawn sump areas were surveyed in response to the March 2006 flood event, because no structures in the Nobles Branch and Able sump areas were included on the lists.

The surveyed finished floor elevations and high water marks were analyzed in a spreadsheet and mapped in GIS. Exhibits 18 and 19 illustrate the surveyed finished floor elevations for the East Levee sump areas, integrating all of the finished floor surveys from November 2005 and March 2006. These exhibits show the locations of the surveyed structures and a graphical comparison of the surveyed finished floor elevations with the estimated finished floor elevations from the Fort Worth District economic model database.

Based on an analysis of all of the East Levee area surveyed finished floor elevations, properties at the addresses shown in Table 3.3 were potentially affected by high sump elevations in Record Crossing Sump and Hampton-Oak Lawn Sump on March 19, 2006. In this case, "potentially affected" means that the surveyed finished floor elevation of the property is less than the maximum measured sump elevation at the pumping plant. Due to the limitations of the data used to develop the list, Table 3.3 is not intended to be a comprehensive list of flood-affected structures for the East Levee sump areas, and structures at the addresses listed in Table 3.3 may not have sustained flood damage as a result of this event.

There are some localized areas that may have experienced higher elevations than the maximum measured sump elevation. One such location on the East Levee was at the east end of Briar Cliff Road, just upstream of the confluence of Knights Branch and Record Crossing Sump. Surveyed high water marks in this area were approximately 406.0 feet, compared to the maximum measured Record Crossing Sump elevation at Hampton Pumping Plant of 404.5 feet. Field investigation of this area after the flood event suggested that the right bank of Knights Branch overtopped near the east end of Briar Cliff Road, and a drainage ditch blocked with brush and debris prevented water from draining out of the area, resulting in higher maximum water surface elevations in this area than Record Crossing Sump. There were more flooded structures on Briar Cliff Road than are listed in Table 3.3; because those structures were affected by factors other than the sump, they were not included in the table. This is just one example of localized phenomena that may affect water surface elevations during a flood event.

Table 3.3 - East Levee Potentially Sump Flooding Affected Structures - March 19, 2006

No.	Address	No.	Address
1	2222 N Stemmons Fwy (Renaissance Hotel)	29	1015 Levee St
2	2030 Market Center Blvd	30	1021 Levee St
3	2050 N Stemmons Fwy (Dallas Trade Mart)	31	1025 Levee St
4	2050 N Stemmons Fwy (Dallas World Trade Center)	32	1125 Levee St
5	2050 N Stemmons Fwy (International Floral & Gift Center)	33	101 Howell St
6	2201 N Stemmons Fwy (Hilton Anatole)	34	111 Leslie St
7	923 Slocum St	35	142 Cole St
8	940 N Industrial Blvd	36	1205 Levee St
9	1444 Oak Lawn Ave	37	1209 Levee St
10	1027 Dragon St	38	1211 Levee St
11	1643 Dragon St	39	1233 Levee St
12	1607 Dragon St	40	1315 Levee St
13	1525 Dragon St	41	1030 Dragon St
14	1523 Dragon St	42	1013 Slocum St
15	1001 Industrial Blvd	43	155 Cole St
16	167 Payne St	44	101 Cole St
17	149 Payne St	45	100 Glass St
18	131 Payne St	46	135 Glass St
19	1010 Levee St	47	100 Oak Lawn Ave
20	134 Pittsburg St	48	1345 Levee St
21	1101 N Industrial Blvd	49	1363 Chemical St
22	161 Pittsburg St	50	1020 Levee St
23	137 Pittsburg St	51	100 Cole St
24	107 Pittsburg St	52	1000 N Industrial Blvd
25	106 Howell St	53	209 Payne St
26	110 Howell St	54	1341 W Mockingbird Ln (Parking Garage)
27	166 Howell St	55	1612 Briar Cliff Rd
28	1001 Levee St	56	3098 N Stemmons Fwy

3.4 HYPOTHETICAL STORM EVENT SIMULATIONS

Hypothetical storm event scenarios were run for existing conditions to identify problems with the system and to establish a baseline against which proposed alternatives would be evaluated. Both 100-year (1% annual chance of occurrence) and 500-year (0.2% annual chance of occurrence) hypothetical storm events were simulated using the combined East Levee watershed HEC-HMS model. The duration of the storms was 24 hours. The 100-year event was simulated for Able Sump using the HEC-HMS model to compute runoff hydrographs and the XP-SWMM model to compute reservoir routing and culvert and weir flow in the sump. These simulations used the precipitation data described in Section 2.1.5.

3.4.1 100-year Storm Event

The 100-year, 24-hour storm event predicted sump stage hydrographs are shown in Figures 3.19 - 3.22. The inundation map for the 100-year, 24-hour storm event is shown in Exhibit 20.

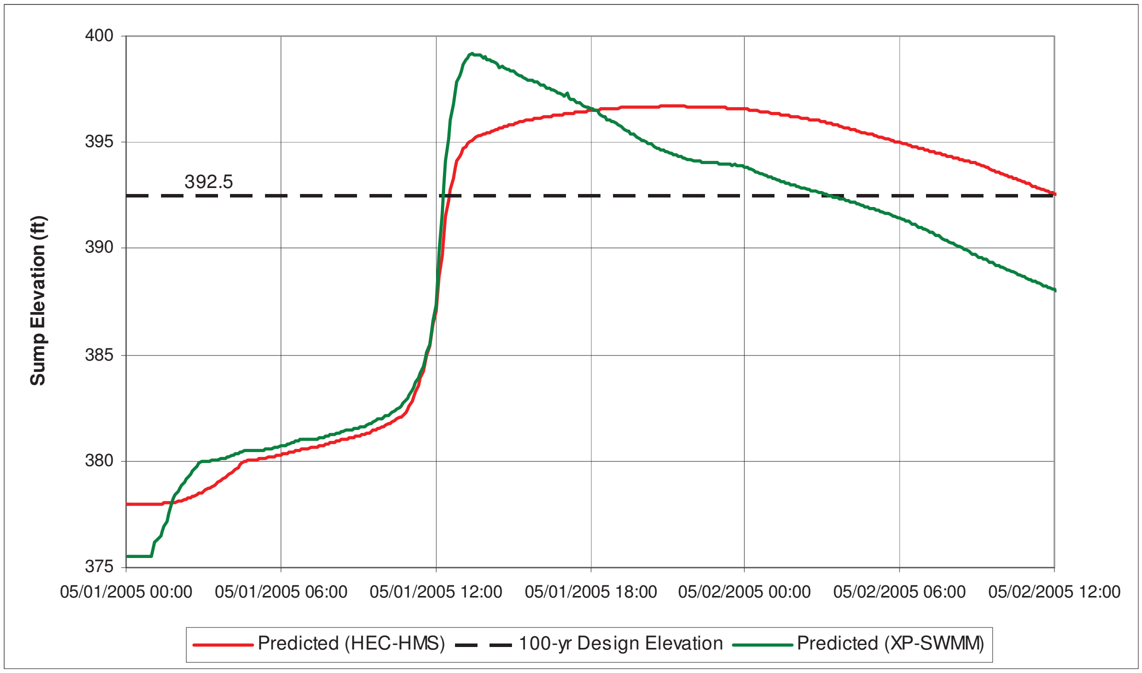

Figure 3.19 - Able Sump Predicted Stage Hydrographs for 100-yr, 24-hr Storm

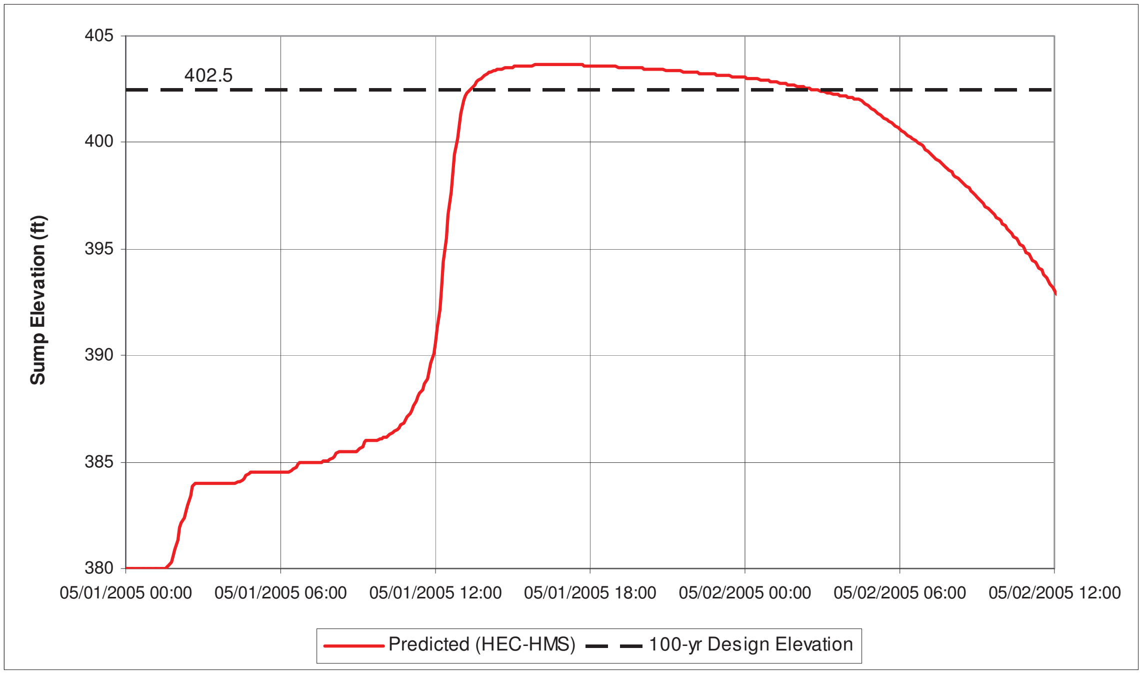

Figure 3.20 - Hampton-Oak Lawn Sump Predicted Stage Hydrograph for 100-yr, 24-hr Storm

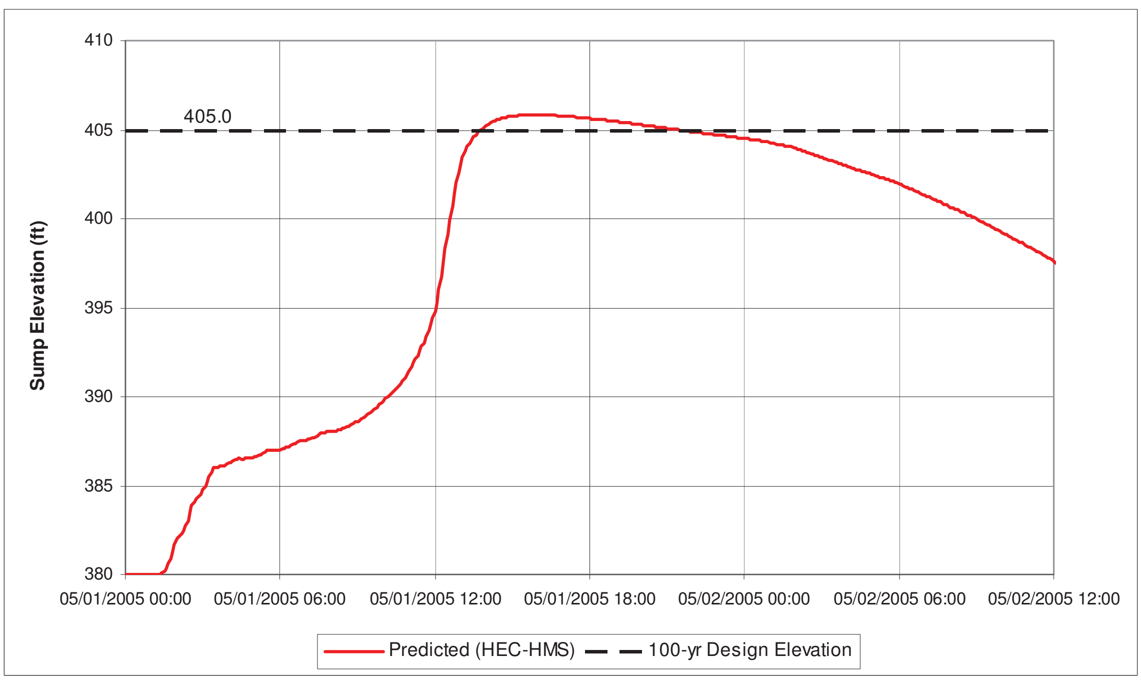

Figure 3.21 - Record Crossing Sump Predicted Stage Hydrograph for 100-yr, 24-hr Storm

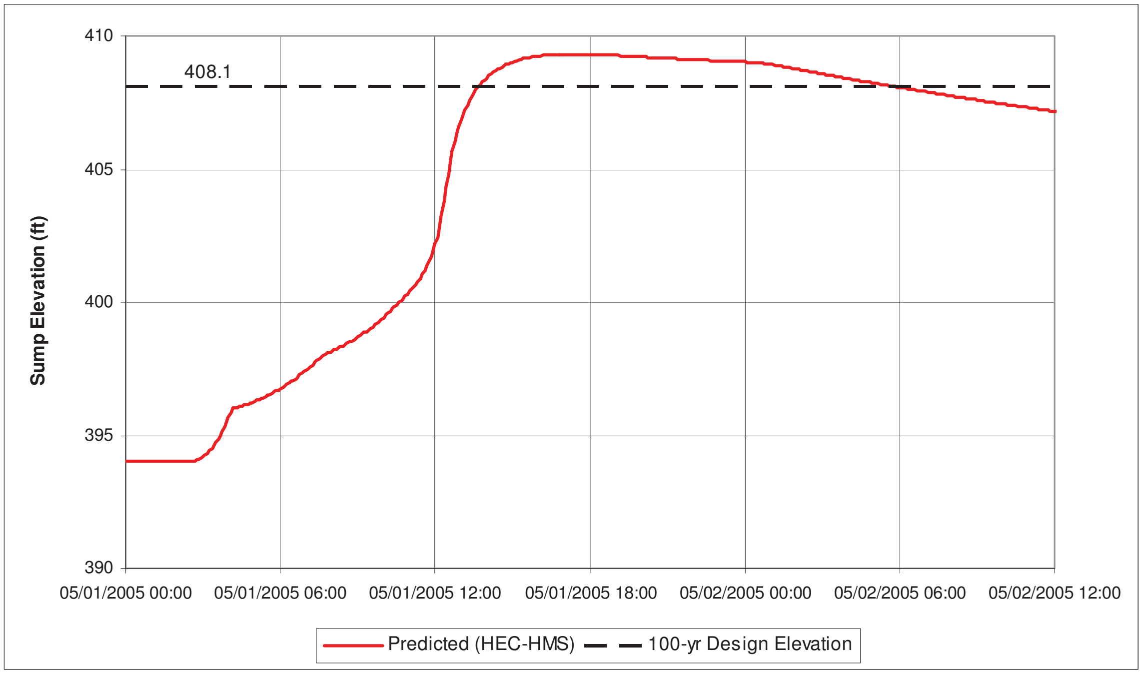

Figure 3.22 - Nobles Branch Sump Predicted Stage Hydrograph for 100-yr, 24-hr Storm

A comparison of computed peak sump stage elevations for the 100-year, 24-hour storm event and the City of Dallas sump design elevations is presented in Table 3.4.

Table 3.4 - Comparison of Design Sump Elevations and Computed 100-year Peak Sump Elevations

Sump	City of Dallas Design Sump Elevation (ft)	Computed 100-year Peak Elevation (ft)
Able	392.5	399.2
Hampton-Oak Lawn	402.5	403.7
Record Crossing	405.0	405.8
Nobles Branch	408.1	409.3

The model results for the 100-year event predict that all of the East Levee sumps will exceed their 100-year design sump elevations. Therefore, improvements will be necessary to maintain the design sump elevations for the 100-year event. The models were used to investigate a number of alternatives to reduce the peak sump stages to the design sump elevations, as described in Section 4 of this report. Based on field surveys of finished floor elevations, it was not considered feasible to recommend increasing the design sump elevations. Model simulations revealed that the improvements necessary to reduce the design elevations by 0.5 ft or more would be prohibitively expensive. Therefore, it is recommended that the current 100-year design elevations be retained.

For the 100-year event, a total of 376 East Levee structures are potentially affected by sump flooding. In this case, "potentially affected" means that the estimated finished floor elevation of the structure is less than the maximum predicted 100-year water surface elevation of the adjacent sump. This is the most conservative estimate possible of the number of affected structures with the available data. By sump area, 141 structures are potentially affected by Hampton-Oak Lawn Sump, 135 structures by Able Sump, and 100 structures by Record Crossing and Nobles Branch sumps. The 100-year potentially affected structures are mapped on Exhibit 21.

3.4.2 500-year Storm Event

The 500-year, 24-hour storm event predicted sump stage hydrographs are shown in Figures 3.23 – 3.26. The inundation map for the 500-year, 24-hour storm event is shown in Exhibit 22.

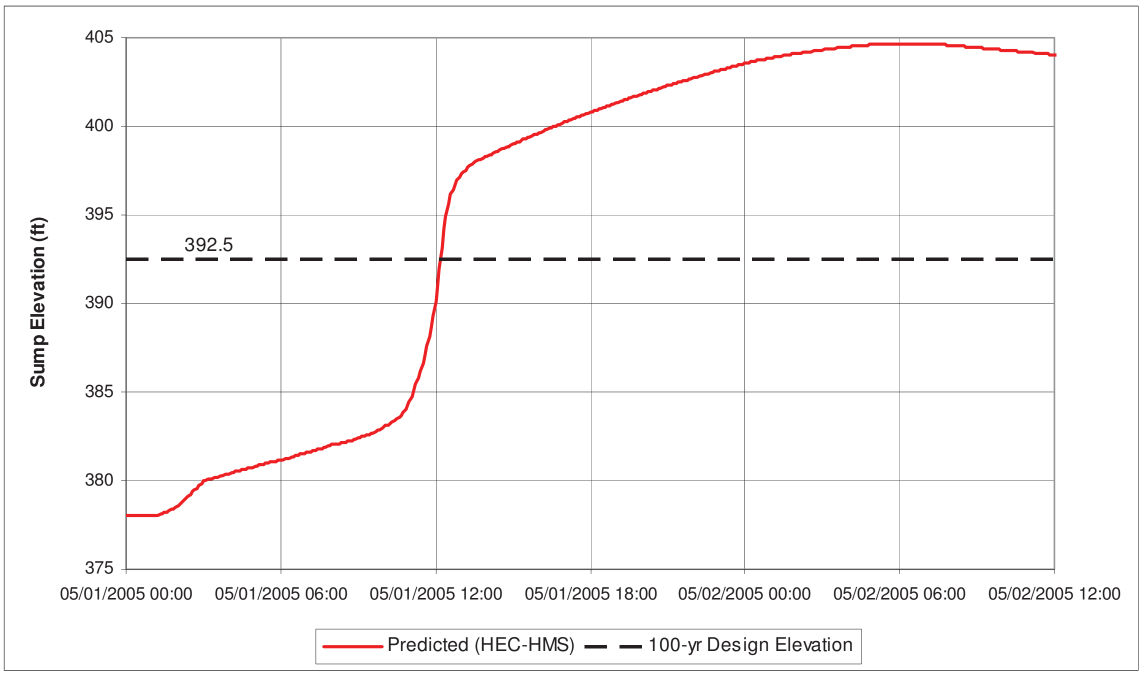

Figure 3.23 - Able Sump Predicted Stage Hydrographs for 500-yr, 24-hr Storm

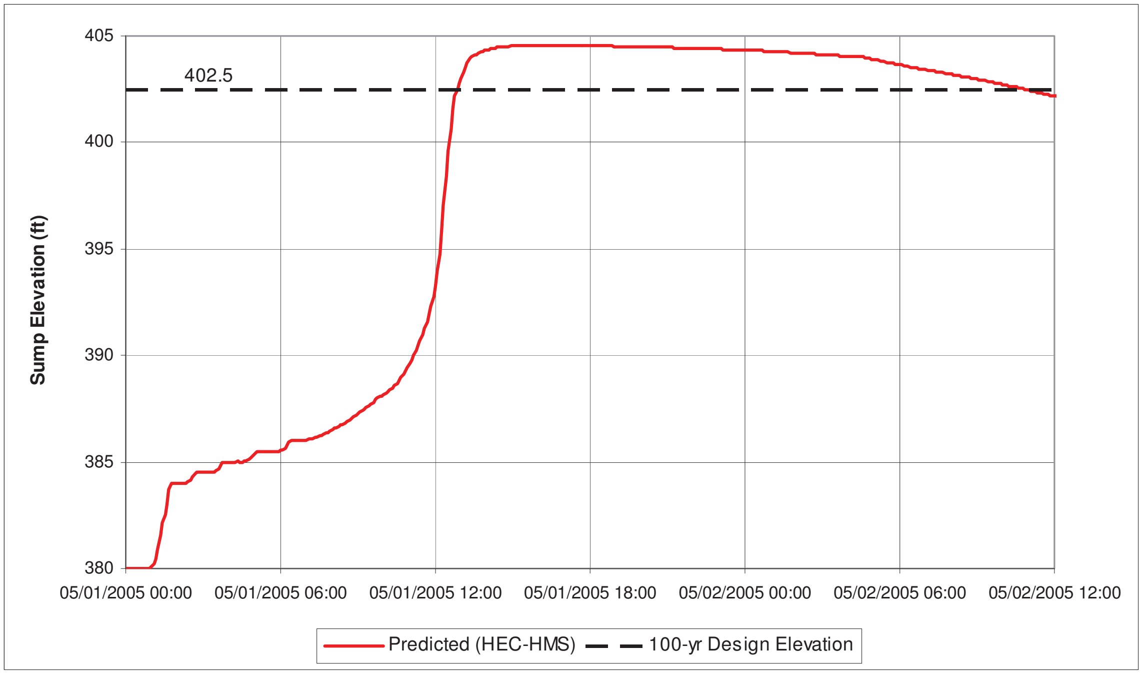

Figure 3.24 - Hampton-Oak Lawn Sump Predicted Stage Hydrograph for 500-yr, 24-hr Storm

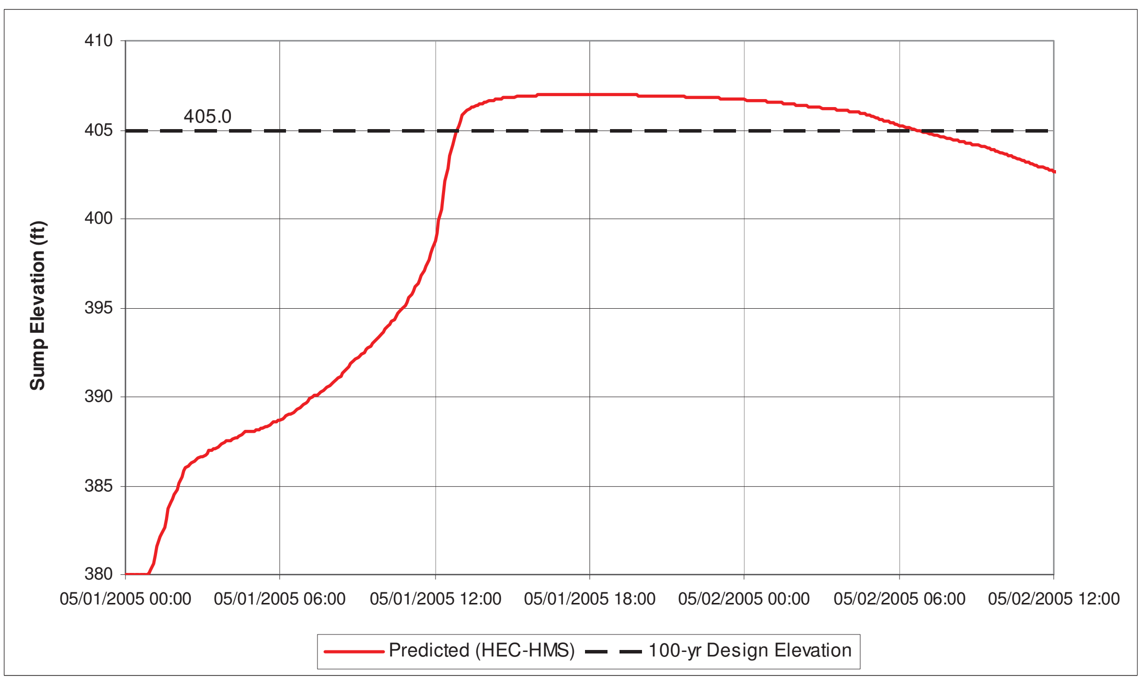

Figure 3.25 - Record Crossing Sump Predicted Stage Hydrograph for 500-yr, 24-hr Storm

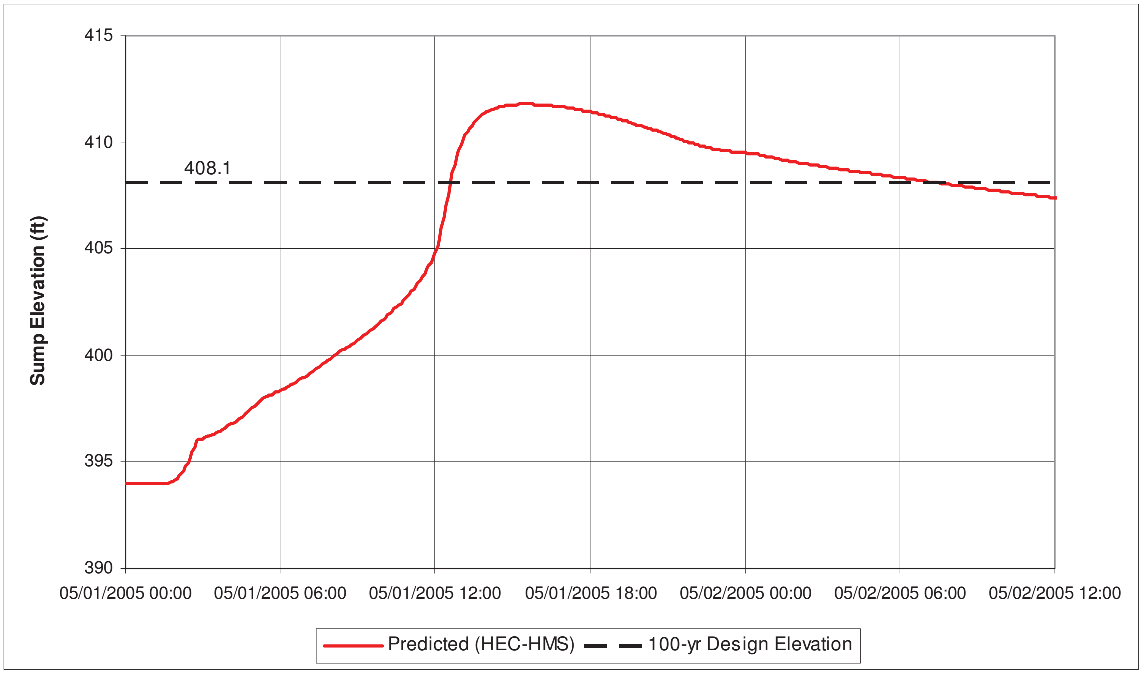

Figure 3.26 - Nobles Branch Sump Predicted Stage Hydrograph for 500-yr, 24-hr Storm

The 500-year results are somewhat questionable because it was necessary to extrapolate the sump elevation-volume curves to higher elevations to allow the simulations to run to completion. This was done by linear extrapolation, which probably results in conservatively high estimates of the computed peak sump stages for the 500-year event.

4. ALTERNATIVES AND RECOMMENDATIONS

A number of alternatives were evaluated to determine a set of recommended improvements to the City's East Levee interior drainage system. The computer models described in Chapter 3 were used to evaluate the alternatives. The goal of the alternatives was to reduce computed peak sump elevations for the 100-year, 24-hour event to the City's design elevations for all of the East Levee sumps. Exhibit 23 shows the inundation map for proposed conditions (100-year design sump elevations).

In addition to improvements necessary to reduce the peak sump stages, consideration was given to modernizing and extending the service life of the existing facilities at least another 50 years. Recommendations for rehabilitating existing pump stations were developed to accomplish these goals.

For planning purposes, the City has requested preliminary opinions of probable costs for both present-day and future conditions (costs escalated at 5% to a midpoint of construction 5 years in the future). Present-day probable costs are based on 2005 conditions, and the cost escalation factor for future probable costs is 1.2763. The detailed preliminary opinions of probable costs for the recommended alternatives found in Appendix A include both present-day probable costs and future probable costs. For simplicity, comparisons of alternates in this chapter are made on the basis of present-day probable costs. The selection of alternates based on present-day probable costs is also valid if future costs are considered, since the future probable costs are a multiple of the present-day probable costs. In the summary table of recommended alternatives (Table 4.7), both present-day and escalated probable costs are included.

When constructing new interior drainage facilities and rehabilitating existing facilities, all outfall structures in the Dallas Floodway must be compatible with the Balanced Vision Plan for the Trinity River Corridor. Some of the outfall structures will have to pass under the proposed man-made lakes to discharge into the realigned pilot channel in the Dallas Floodway. All of the existing and proposed East Levee outfall structures will be affected in some way by the Balanced Vision Plan. The recommendations from this study address the interaction of the interior drainage facilities with the Balanced Vision Plan. The required length of outfall extensions for the East Levee interior drainage features were estimated based on a conceptual drawing of the Balanced Vision Plan; these estimates and associated probable costs may be revised when the Balanced Vision Plan features are finalized.

4.1 SUMP VOLUME VERSUS ADDITIONAL PUMPING CAPACITY

Fundamentally, reduction of peak stages in a sump may be accomplished by decreasing the magnitude or altering the timing of the inflow hydrograph to the sump, increasing the discharge from the sump, or increasing the storage capacity of the sump. It is not considered feasible to decrease the magnitude or alter the timing of the existing sump inflow hydrographs significantly due to the large amount of detention storage which would be required. Certainly, future land development in the interior drainage basins should include drainage features including detention in accordance with City of Dallas development guidelines. Increasing sump storage capacity and/or pumping capacity are the only viable alternatives to reducing peak stages in the sumps.

If land is available or can be acquired at a favorable price, it could be more cost-effective to increase sump storage capacity rather than increasing pumping capacity; however, the highly developed nature and high property values in the area surrounding the existing East Levee sumps limit their potential expansion. The property maps accompanying this report show the tracts adjacent to the sumps. The utility maps included with the property maps are approximate and are not intended to be comprehensive. They were developed based on information from the City of Dallas, perfunctory cooperation of franchise utility owners, and field reconnaissance. The utility maps are not a substitute for field location of utilities or as-built plans and is not sufficient for construction purposes; they are intended only to provide an indication of the types of utilities in place near the sumps and their general locations.

It is not considered feasible to expand the footprint of the East Levee sumps except in a few locations, which would not have a major effect on the available sump storage capacity. Consequently, enhancements to sump storage were assumed to be confined to the limits of the existing sump channels, where additional volume could be gained by increasing the side slopes until they approached vertical. This would require the construction of concrete or gabion retaining walls in the sumps, substantially increasing the cost. In addition, 404 permits and associated mitigation may be required for excavation in the sumps.

The required additional pumping capacity for a sump can be expressed as a function of the total sump volume. As sump volume increases, the required additional pumping capacity decreases. Many different model simulations were executed to develop curves relating these variables for each sump. These curves, along with preliminary opinions of probable costs for the sump improvements and pump stations, were used to select a recommended alternative for each sump that minimized the total cost.

A preliminary opinion of probable cost per unit length of sump improvement was developed by the following method:

1. Based on the surveyed sump cross sections discussed in Section 2.3.3, the average existing sump side slope was estimated to be 4:1 and the average existing depth of the sumps was estimated to be 15 feet.
2. It was assumed that an approximately rectangular sump cross-section could be achieved by construction of an approximately vertical gabion retaining wall from the bottom of the sump to the top of bank and excavating the cross section.
3. Using the estimates of the average side slope and average sump depth, an average increase in cross-sectional area was computed, resulting in an estimated 95 acre-feet of sump volume increase per mile of sump improvement.
4. Preliminary opinions of probable costs associated with excavation and gabion retaining wall construction were developed to estimate a probable cost per unit length of sump improvement — approximately $12 million per mile of sump improvement.

Using the average increase in cross-sectional area from step 3, the approximate required length of sump improvements to achieve a given increase in sump volume could easily be calculated. This length of sump improvement was associated with a total probable cost using the rate developed in step 4. In this manner, probable cost analyses of sump improvements versus additional pumping were developed for each sump. In each case, the analyses revealed that the probable costs associated with sump improvements substantially exceeded the probable costs of pump station construction. Thus, for the conditions considered for this analysis, sump volume enhancement is not an economically viable alternative to the construction of new pump stations.

It should be noted that general clearing and dredging of the sumps can provide a minor enhancement to the sump capacity and improve their operation for a reasonable cost, but such improvements would not be sufficient to negate the need for or substantially reduce the required size of additional pump stations.

No alternatives evaluated for this analysis eliminated the need for additional pump stations. Therefore, a new pump station is recommended at each pumping plant. Figure 4.1 shows a conceptual plan and section view of a typical proposed pump station.

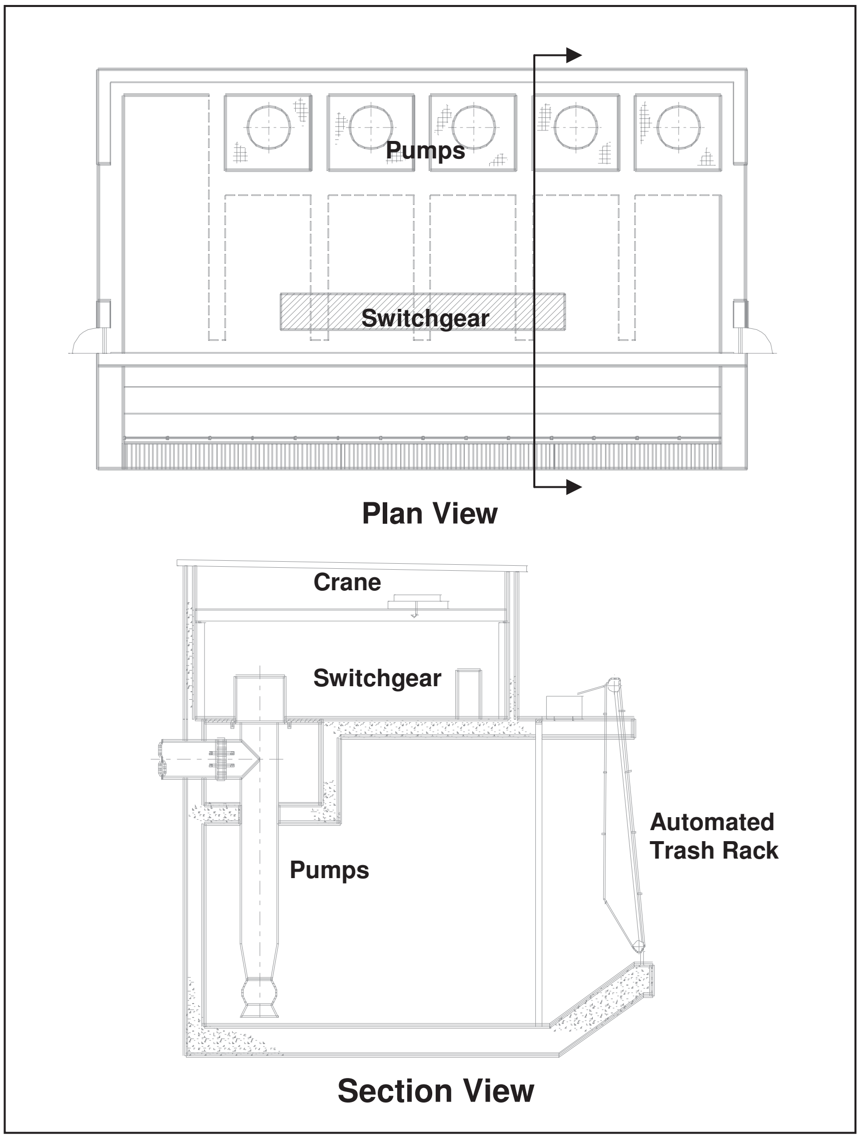

Figure 4.1 - Conceptual Plan and Section Views of Proposed Pump Station

4.2 ABLE PUMPING PLANT AND SUMP

The XP-SWMM model described in Section 3.2.3 was used to evaluate alternatives for Able Sump. A number of structural improvements were considered, including the addition of a new pump station, replacement of culverts connecting the individual ponds, the addition of a new culvert connecting Ponds 1 and 5, and the addition of an inverted siphon connecting Ponds 5 and 6. Runoff hydrographs for the Able Sump drainage basin were computed in the East Levee HEC-HMS model, and XP-SWMM was used to compute the reservoir routing and culvert and weir flows.

As noted in the 2004 Halff Associates Mill Creek Master Drainage Plan Study, overflow from the Mill Creek drainage basin into Able Sump represents a significant operational problem for Able Sump, since the Able Sump system was not designed to handle the additional inflow. Based on initial model simulations, it was found that it would be prohibitively expensive to provide the required improvements to meet the City's Able Sump 100-year design elevation of 392.5 ft if the Mill Creek overflow is included. Therefore, for the purpose of evaluating alternatives for Able Sump, it was assumed that measures will be taken to prevent overflow from the Mill Creek system into Able Sump. This assumption is reasonable, since funding for a proposed relief sewer for the Mill Creek system will be included in the City's November 2006 Bond Program package.

4.2.1 Pump Stations

The required additional pumping capacity for Able Sump depends upon other improvements made in the sump. The recommended pump station will be part of a combination of feasible alternatives selected based upon the lowest total probable cost. No alternative was developed that eliminated the need for an additional pump station at Able Pumping Plant.

The proposed new Able pump station will include five equally-sized vertical turbine pumps, one of which is a backup. The pump station will include all the necessary mechanical and electrical equipment to operate the pump station efficiently and with minimum maintenance. This will include equipment such as automated self cleaning bar screens with a trash conveyor system, closed-circuit television (CCTV) cameras, and SCADA equipment for remote operation and monitoring.

The existing Able pump stations were evaluated by an electrical engineer and architect on the project team and the following rehabilitation recommendations were made:

Small Able Pump Station (SAX)

- Replace two pumps (40,000 gpm) with new 40,000 gpm pumps
- Replace and repair tile coping and roof
- Replace steel sliding door and hardware
- Replace wall louvers: fixed and operable
- Paint interior (walls, floor, and ceiling)
- Re-brick closed openings to provide weather tight enclosure
- Provide masonry to close window currently blocked with steel plate
- Replace roof hatch hardware
- Repair vehicular gates
- Replace transformers and panelboards
- New 480V motor control center
- New conduit and wire
- New lighting
- Reconnect controls and SCADA system

The probable cost for rehabilitation of Small Able Pump Station (SAX) is approximately $2.4 million. A detailed preliminary opinion of probable cost for the pump station rehabilitation is included in Appendix A. In addition, the outfall of SAX will need to be extended approximately 1,100 feet under a proposed man-made lake in the Dallas Floodway as part of the Balanced Vision Plan for the Trinity River Corridor. The probable cost to extend the outfall is approximately $1.1 million.

Large Able Pump Station (LAX)

- Replace three pumps (47,000 gpm) with new 50,000 gpm pumps
- Replace roof and gravel stop
- Replace double doors, frame, and hardware
- Replace wall louvers: fixed and operable
- Clean brick and graffiti
- Paint Interior (walls, floor, and ceiling)
- Replace roof hatch hardware
- Repair vehicular gates
- Replace transformers and panelboards
- New 480V motor control center
- New conduit and wire
- New lighting
- Reconnect controls and SCADA system

The probable cost for rehabilitation of Large Able Pump Station (LAX) is approximately $4.1 million. Appendix A includes a preliminary opinion of probable costs for the rehabilitation of LAX. In addition, the outfall of LAX will need to be extended approximately 1,100 feet under a proposed man-made lake in the Dallas Floodway as part of the Balanced Vision Plan for the Trinity River Corridor, at a probable cost of approximately $1.1 million.

If no additional improvements are made to Able Sump, an additional pump station of 704,000 gpm capacity would be required to limit the computed 100-year, 24-hour storm peak sump stage to the 100-year design elevation. This pump station is designated Alternate Able-1. The probable cost of the pump station is $35.9 million, plus an additional probable cost of $3.3 million to construct an extended outfall under the proposed man-made lake in the Dallas Floodway. The total probable cost of Alternative Able-1 is $39.2 million.

4.2.2 Sump Capacity Improvements

No specific alternatives were developed for sump capacity improvements in Able Sump. Any enhancement to sump capacity would need to be made in Ponds 1-5 to achieve a reduction in peak stages. The biggest incremental improvement could be realized by increasing the storage capacity of Pond 1. The western side of Pond 1 is confined by the interior toe of the East Levee. Therefore, it is impossible to expand Pond 1 in that direction. Pond 1 is confined along its western bank by existing development. If the eastern bank of Pond 1 were made vertical through the installation of a concrete or gabion retaining wall, the incremental storage gained would not justify the cost of the wall and excavation. If property between Rock Island Street and Industrial Boulevard north of RL Thornton Freeway (I-30) and/or along the west side of Industrial Boulevard south of RL Thornton Freeway could be acquired at a favorable cost, the sump could be expanded to the east. An evaluation of the economic feasibility of acquiring these properties is beyond the scope of this study. However, as planning proceeds for the realignment of Industrial Boulevard and reconstruction of the Mixmaster as part of the Pegasus Project, some of this property may become available for expansion of the sump. It is essential that no net loss of sump storage occur as a result of the realignment of Industrial Boulevard, the reconstruction of the Mixmaster, or any other future development in the Able Sump area. As part of the design process for future transportation and development projects, consideration should be given to the possibility of expanding Able Sump storage and enhancing conveyance between the sump ponds. The property maps accompanying this report may be consulted to locate the tracts adjacent to the sump.

Ponds 2-5 are similarly confined by existing development. Due to the small size of the ponds, expansion of the sump by addition of vertical walls would not provide enough additional storage to offset the cost of the walls.

4.2.3 Culverts

Field observations and model results suggested that Able Sump operation could be improved if conveyance between sump ponds were enhanced. This can be done by improving existing culvert connections or by adding additional connections between ponds. Two different types of culvert alternatives were evaluated. First, replacement of existing culverts that connect the Able Sump ponds was investigated. The second alternative consisted of the addition of a new culvert connecting Ponds 1 and 5.

4.2.3.1 Replacement of Existing Culverts

One of the primary motivations for developing the XP-SWMM model of Able Sump was to account for the losses that occur in the culverts connecting the individual ponds that make up the sump. As discussed in Section 3.2.3, it is believed that undersized culverts contributed to the high sump stages (exceeding the 100-year design elevation) observed during the May 5, 1995 storm event. Of particular concern were the culverts connecting Ponds 1 and 2 under Industrial Boulevard.

XP-SWMM model results showed that losses in the Pond 1-2 culverts were not great enough to prevent Ponds 1 and 2 from equalizing during the 100-year, 24-hour storm event. However, the model revealed that improving the Pond 1-2 culverts could reduce peak sump stages. Furthermore, the model results indicated that reductions in peak sump stages were possible by improving the culverts between Ponds 2 and 3 (RL Thornton and Stemmons Freeway and ramps), Ponds 3 and 4 (Cadiz Street), and Ponds 4 and 5 (Industrial Boulevard). These proposed culvert improvements alone are not sufficient to reduce the computed 100-year, 24 hour storm peak sump stage to the design elevation. The model showed that no reductions in peak sump stages were possible by improving the culvert connections on the opposite side of the Belleview Pressure Sewer from Able Pumping Plant (i.e., culverts connecting Ponds 6-9).

The proposed culvert improvements for Able Sump are summarized in Table 4.1 and are shown on Exhibit 24. All of the proposed culverts use the same invert elevations as the existing culverts to maintain existing flowlines in the sump.

Table 4.1 - Proposed Able Sump Culverts

Culvert Connection	Street or Highway	Proposed Culvert
Ponds 1-2	Industrial Blvd	Approx 380 LF of 3 – 10'x10' RCBC
Ponds 2-3	RL Thornton & Stemmons Freeway lanes and ramps	Approx 270 LF of 3 – 10'x10' RCBC
Ponds 3-4	Cadiz St	Approx 170 LF of 3 – 10'x8' RCBC
Ponds 4-5	Industrial Blvd	Approx 170 LF of 3 – 10'x6' RCBC

The culvert improvements listed in Table 4.1 were incorporated into Alternate Able-2. The total probable cost for these four culverts is approximately $3.0 million. Because the culverts alone are not sufficient to reduce predicted peak sump stages to the design elevation, an additional pump station of 400,000 gpm capacity is also required as part of Alternate Able-2, at a probable cost of approximately $29.0 million, plus a probable cost of $3.3 million for an extended outfall under the proposed man-made lake in the Dallas Floodway. The total probable cost for Alternate Able-2 is $35.3 million.

4.2.3.2 Addition of New Culvert Connecting Ponds 1 and 5

Another alternative investigated was the connection of Ponds 1 and 5 with a box culvert. During storm peaks, this would allow excess water in Pond 1 to back up directly into Pond 5, more efficiently utilizing the existing storage in Pond 5. This new culvert would run parallel to the interior toe of the East Levee, behind the Fuel City truck stop and the electrical substation between the southbound and northbound embankments of I-35E. The proposed alignment is approximately 1,750 ft long, and is shown on Exhibit 24. Part of the proposed alignment is sometimes used as pasture for a small herd of longhorn cattle.

Several different culvert configurations were evaluated and the best compromise between culvert capacity and probable cost appeared to be a double 10-ft x 10-ft box culvert, at a probable cost of approximately $3.4 million. XP-SWMM model simulations revealed that the addition of this culvert was effective in reducing the predicted 100-year peak sump elevation, but not enough to negate the need for a new pump station at Able Pumping Plant. With the proposed culvert in place, an additional 280,000 gpm pump station would be required to reduce the computed peak sump stage to the design elevation. The probable cost of the pump station is approximately $26.3 million, plus a probable cost of $3.3 million required to construct an extended outfall under the proposed man-made lake in the Dallas Floodway. This combination of a new culvert and pump station was designated Alternate Able-3. The total probable cost of Alternate Able-3 is approximately $33.0 million.

4.2.4 Pressure Sewers

To justify constructing a new pressure sewer in a sump drainage basin, the following criteria must be met:

- A location capable of developing enough hydraulic head to operate as a pressure sewer must be available.
- The new pressure sewer subbasin must be large enough to contribute a significant amount of flow to the sump, so that the cost of the pressure sewer is offset by savings in other sump improvements such as increased sump storage or additional pumping requirements.

When these criteria were applied to the evaluation of a new pressure sewer in the Able Sump drainage basin, no suitable location meeting both of these criteria could be found. Therefore, no new pressure sewers were investigated for the Able Sump drainage basin.

4.2.5 Inverted Siphon

As discussed in Section 3.2.3, the weir created by the Belleview Pressure Sewer effectively isolates Ponds 1-5 from Ponds 6-9 until the weir is overtopped, so the volume in Ponds 6-9 below the crest of the weir is currently ineffective except in storing the local runoff received by Ponds 6-9. To use the storage volume in Ponds 6-9 more effectively, the possibility of connecting Ponds 5 and 6 with a culvert was evaluated. Because the Belleview Pressure Sewer also creates an underground barrier, a sag culvert or inverted siphon connection passing under the pressure sewer would be required to connect the ponds.

The existing 42-inch culvert beneath the Associated Freezers warehouse building connecting Ponds 4 and 7 currently does not have sufficient flow capacity to equalize the levels between Ponds 1-5 and Ponds 6-9. The condition of this culvert is unknown. The culvert apparently has a sag profile, which would be necessary to pass under the Belleview Pressure Sewer. Because of the lengthy alignment of the culvert under an existing building, it does not appear feasible to replace the culvert or augment it with additional capacity at this time. However, if the existing building were ever demolished and the property re-developed, consideration should be given to replacing the existing culvert with a larger section as part of the development.

A connection under the Belleview Pressure Sewer between Ponds 5 and 6 would result in the shortest possible alignment. Therefore, a 48-inch inverted siphon connecting Ponds 5 and 6 was investigated as an alternative in the XP-SWMM model. The model

showed that the siphon was not effective in reducing peak sump stages, but would aid in allowing Ponds 6-9 to drain towards Able Pumping Plant after the peak of a storm event. Multiple (up to six) 48-inch barrels were also investigated, and again the model results showed that no significant reductions in peak stages were achieved. Multiple barrels were shown to allow Ponds 6-9 to drain faster.

Unfortunately, the model results did not predict the anticipated reduction in peak stages associated with the addition of an inverted siphon between Ponds 5 and 6. Given the future development potential of the property around Ponds 6-9 associated with the Balanced Vision Plan for the Trinity Corridor, construction of an inverted siphon connection between Ponds 5 and 6 should be considered as part of the development plans to help prevent stagnant standing water in Ponds 6-9.

4.2.6 Recommendations

The three alternates evaluated for Able Sump/Able Pumping Plant are summarized in Table 4.2.

Table 4.2 - Summary of Able Sump/Able Pumping Plant Alternatives

Alternate	Description	Total Probable Cost
Able-1	Construct new 704,000 gpm pump station at Able Pumping Plant	$39.2 million
Able-2	Construct new 400,000 gpm pump station at Able Pumping Plant, and construct culvert improvements as specified in Table 4.1	$35.3 million
Able-3	Construct new 280,000 gpm pump station at Able Pumping Plant, and construct a double 10' x 10' RCBC connecting Ponds 1 and 5	$33.0 million

The alternate with the lowest total probable cost is Alternate Able-3 at $33.0 million. However, discussion with City of Dallas Public Works and Transportation personnel led to the conclusion that the feasibility of constructing the box culvert connecting Ponds 1 and 5 is questionable, in part because of the geotechnical challenges associated with excavation along the interior toe of the levee. Therefore, Alternate Able-3 was rejected.

Alternate Able-2 is the recommended solution for Able Sump. In addition, it is recommended that the rehabilitation of the existing Able pump stations be performed along with the construction of the required outfall extensions at a total probable cost of approximately $8.7 million. The total probable cost of all recommended improvements for Able Sump is approximately $44.0 million. Detailed preliminary opinions of probable costs for these recommendations are found in Appendix A.

As discussed previously, the area surrounding Able Sump is expected to be affected by transportation projects and private development. It is essential that no net loss of storage volume occur as a result of these projects. Furthermore, consideration should be given to conveyance between separate storage areas. If possible, it would be advantageous to connect adjacent sump ponds directly by combining them into one contiguous pond or by replacing culvert connections with bridges.

4.3 BAKER PUMPING PLANT AND HAMPTON-OAK LAWN SUMP

The HEC-HMS model of the combined East Levee watershed was used to evaluate potential improvements for the Hampton-Oak Lawn Sump area and Baker Pumping Plant. The alternates investigated for the Hampton-Oak Lawn Sump area included increased pumping capacity, increased sump storage, pressure sewers, and a floodwall. The recommended alternate reflects the lowest combined probable cost to meet the goal of limiting the predicted 100-year maximum sump stage to the City of Dallas 100-year design sump elevation of 402.5 ft.

4.3.1 Pump Stations

No alternate evaluated for Hampton-Oak Lawn sump eliminated the need for an additional pump station to be built at Baker Pumping Plant. The proposed new Baker pump station will include five equally-sized vertical turbine pumps, one of which is a backup. The pump station will include all the necessary mechanical and electrical equipment to operate the pump station efficiently and with minimum maintenance. This will include equipment such as automated self cleaning bar screens with a trash conveyor system, CCTV cameras, and SCADA equipment for remote operation and monitoring.

The existing Old Baker Pump Station has a capacity of 208,000 gpm and is approximately 75 years old. Local lore is that the horizontal split case pumps at Old Baker Pump Station were purchased as surplus from the City of New Orleans, so the pumps may be older than the station itself. This claim cannot be confirmed, but the pumps are the same type and from the same era as some of the New Orleans pumps. The exposed armatures of the AC synchronous motors used on the Old Baker pumps are a potential safety hazard. During the May 5, 1995 storm event, Old Baker Pump Station was flooded because one of the pumps was out for service, and water backed up into the pump station through the pump casing. The pump floor elevation is lower than the 100-year design elevation for Hampton-Oak Lawn Sump. For these reasons, it is recommended that Old Baker Pump Station be decommissioned. Due to its age and historical significance, consideration should be given to preserving Old Baker Pump Station as a museum. The new Baker pump station capacities considered in this analysis include an extra 208,000 gpm to account for the retirement of Old Baker Pump Station.

The existing Baker pump stations were evaluated by an electrical engineer and architect on the project team and the following rehabilitation recommendations were made. No rehabilitation recommendations were made for Old Baker Pump Station, since it is recommended to be decommissioned.

New Baker Pump Station (NBX)

- Replace roof
- Paint and repair sliding door
- Fix damage screen on louvers
- Clean and paint exterior concrete
- Paint Interior (walls, floor, and ceiling)
- Grind slope in concrete at door (water running under door during rains)
- Replace 5KV switchgear
- Replace transformers and panelboards
- New 480V motor control center
- New conduit and wire
- New lighting
- Reconnect controls and SCADA system

The total probable cost for rehabilitation of the existing New Baker Pump Station (NBX) is approximately $676,000. A detailed cost estimate for the rehabilitation of NBX is included in Appendix A. In addition, the NBX outfall will need to be extended approximately 300 feet to avoid conflicts with the proposed Trinity Parkway at a probable cost of approximately $300,000.

The curve of required additional pumping capacity versus additional sump volume for Baker Pumping Plant and Hampton-Oak Lawn Sump is shown in Figure 4.2. The additional pump station capacities shown on the y-axis of Figure 4.2 include an extra 208,000 gpm to account for the retirement of Old Baker Pump Station. Figure 4.2 shows that if no additional sump storage is provided, the required additional pumping capacity is 700,000 gpm; if 407 acre-feet of additional sump storage is provided, the required additional pumping capacity is 200,000 gpm (no net increase in the current Baker Pumping Plant capacity).

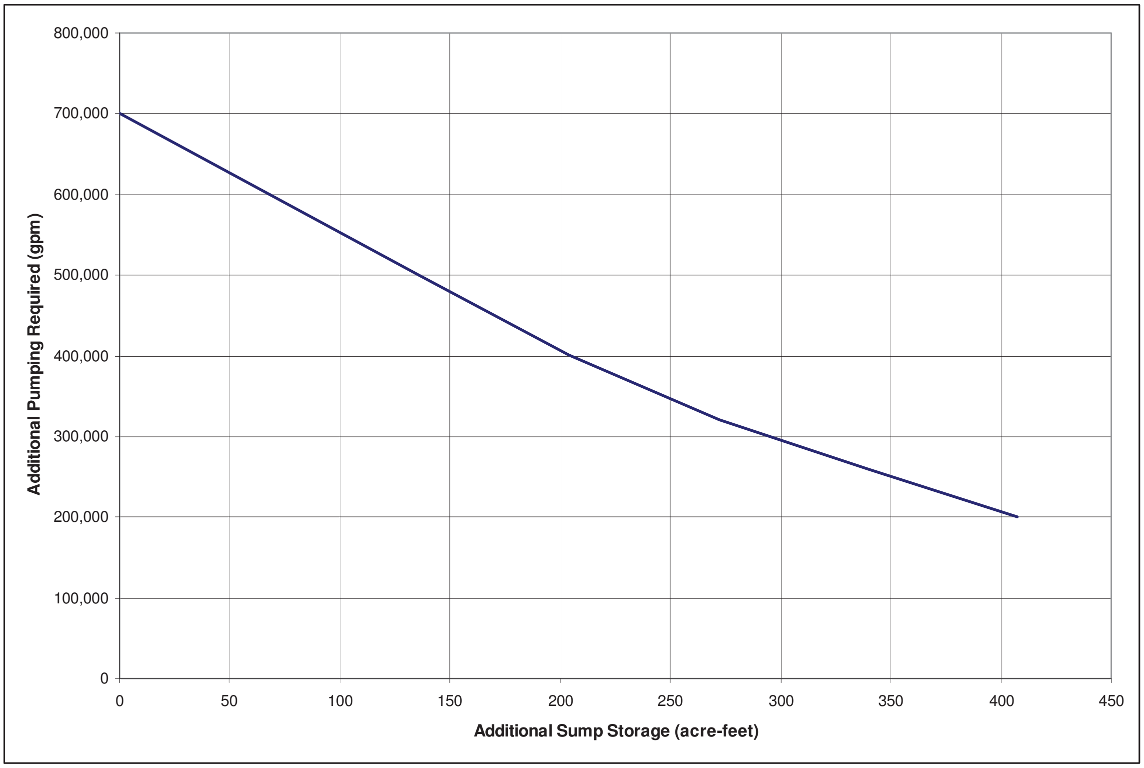

Figure 4.2 - Additional Pumping vs. Additional Sump Storage for Hampton-Oak Lawn Sump

Seven different alternates were formulated to evaluate the probable cost of additional sump storage versus additional required pumping. The alternates are summarized in Table 4.3. The probable costs listed in Table 4.3 do not include the cost of the required outfall extension to avoid conflicts with the proposed Trinity Parkway, which is constant for all alternates.

Table 4.3 - Summary of Alternates for Hampton-Oak Lawn Sump/Baker Pumping Plant

Alternate	Volume of Additional Sump Storage (ac-ft)	Probable Cost of Additional Sump Storage (millions)	Additional Pumping Capacity Required (gpm)	Probable Cost of Additional Pumping (millions)	Total Probable Cost (millions)
HOL-1	0	$0	700,000	$35.8	$35.8
HOL-2	68	$8.6	600,000	$33.5	$42.1
HOL-3	136	$17.2	500,000	$31.3	$48.5
HOL-4	204	$25.8	400,000	$29.0	$54.8
HOL-5	272	$34.4	320,000	$27.2	$61.6
HOL-6	340	$43.0	260,000	$25.9	$68.9
HOL-7	407	$51.4	200,000	$21.0	$72.4

The probable costs of these alternates are shown graphically in Figure 4.3. Figure 4.3 shows that the probable cost of additional sump storage increases at a faster rate than the probable cost of additional pumping decreases. Therefore, the lowest total probable cost is achieved with the combination of zero additional sump storage and a new pump station with 700,000 gpm capacity (Alternate HOL-1). The probable cost of Alternate HOL-1 is approximately $35.8 million for the new pump station plus a probable cost of approximately $900,000 to extend the outfall beyond the proposed Trinity Parkway. Therefore, the total probable cost of Alternate HOL-1 is approximately $36.7 million.

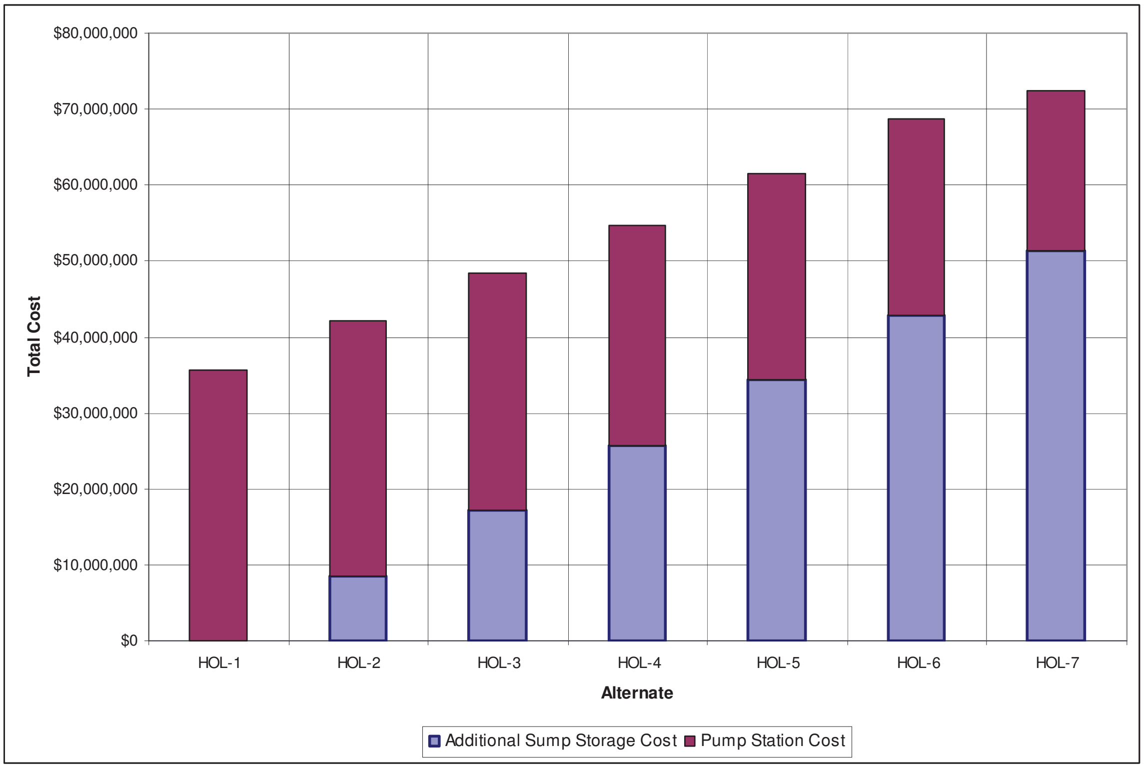

Figure 4.3 - Additional Sump Storage Cost vs. Pump Station Cost for Hampton-Oak Lawn Sump

4.3.2 Pressure Sewers

Based on a thorough analysis of the Hampton-Oak Lawn Sump drainage basin, a potential location for a new pressure sewer was identified on Cedar Springs Branch. The proposed alignment of the Cedar Springs Branch Pressure Sewer runs along Wycliff Avenue and is shown on Exhibit 25. The length of the proposed pressure sewer is approximately 2 miles, and the proposed section is a 15-ft diameter circular conduit. The capacity of the proposed pressure sewer is approximately 3,500 cfs with a tailwater elevation in the Dallas Floodway computed as described in Section 2.3.4. At a preliminary probable tunneling cost of $6,480/ft, the total probable cost of the Cedar Springs Branch Pressure Sewer is in excess of $68 million. This far exceeds the probable cost of the proposed new Baker pump station described in Section 4.3.1. Therefore, construction of the Cedar Springs Branch Pressure Sewer would be economically infeasible.

4.3.3 Floodwalls

As discussed in Sections 1.2.2.3 and 3.1.3, at the design sump elevation for Record Crossing Sump, overflow will occur from Record Crossing Sump into Hampton-Oak Lawn Sump along the left bank of Knights Branch near the intersection of Inwood Road and Irving Boulevard. The 1971 Interior Drainage Study recommended the construction of a floodwall along the left bank of Knights Branch from the Record Crossing Sump channel to the TRE embankment. However, based on the 2001 LiDAR 2-ft contours, this floodwall would not prevent the overflow from Record Crossing Sump unless Irving Boulevard and the southbound Stemmons Freeway access road were also raised. It appears unlikely that the grades of these streets could be altered, particularly since improvements to Inwood Road have already been designed. Therefore, the Knights Branch floodwall is not recommended at this time. All recommendations for Baker Pumping Plant and Hampton-Oak Lawn Sump presented in this report assume that the overflow from Record Crossing Sump into Hampton-Oak Lawn Sump will occur for the 100-year, 24-hour storm event.

4.3.4 Recommendations

As shown in Table 4.3 and Figure 4.3, the least expensive alternate for Hampton-Oak Lawn Sump is Alternate HOL-1, at a probable cost of approximately $35.8 million, plus an additional $900,000 for the required outfall extension. Therefore, Alternate HOL-1 is the recommended solution for Hampton-Oak Lawn Sump. The rehabilitation of the existing New Baker pump station is also recommended, at a probable cost of approximately $676,000 plus $300,000 for the required outfall extension. The total probable cost for these improvements is approximately $37.7 million. Detailed preliminary opinions of probable costs for these recommended improvements are included in Appendix A.

4.4 NOBLES BRANCH SUMP

Nobles Branch Sump is drained by a 60-inch gated conduit structure at Empire Central Drive. This gated culvert is the only means of releasing water from Nobles Branch Sump. The alternatives investigated for Nobles Branch Sump include increasing the sump storage capacity and increasing the flow out of the sump by adding more gated culverts at Empire Central Drive. The goal of the alternatives was to reduce the 100-year, 24-hour predicted peak sump stage to the design elevation of 408.1 ft. No feasible pressure sewer location was found in the Nobles Branch Sump drainage basin.

Because Nobles Branch Sump releases water to Record Crossing Sump, the selected alternate for Nobles Branch Sump affects the alternates for Record Crossing Sump. Therefore, the alternatives for both Nobles Branch Sump and Record Crossing Sump had to be optimized simultaneously to minimize the total probable costs for both sumps.

Three alternatives were formulated for Nobles Branch Sump, as summarized in Table 4.4.

Table 4.4 - Summary of Alternates for Nobles Branch Sump

Alternate	Volume of Additional Sump Storage (ac-ft)	Probable Cost of Additional Sump Storage (millions)	Number of Additional Gated Culverts Required (gpm)	Probable Cost of Additional Gated Culverts (millions)	Total Probable Cost (millions)
NB-1	212	$26.8	0	$0	$26.8
NB-2	98	$12.4	1	$0.3	$12.7
NB-3	0	$0	3	$1.0	$1.0

For Nobles Branch Sump, it was found that it was far less expensive to construct additional gated culvert structures than to provide additional sump storage. Because providing additional gated culverts increased the releases from Nobles Branch Sump into Record Crossing Sump, this increased the pumping and/or sump storage required for Record Crossing Sump. However, the total probable cost for both sumps was found to beminimized for Nobles Branch Sump Alternate NB-3. Therefore, the recommended alternate for Nobles Branch Sump is NB-3, at a total probable cost of $1.0 million.

4.5 HAMPTON PUMPING PLANT AND RECORD CROSSING SUMP

The HEC-HMS model of the combined East Levee watershed was used to evaluate potential improvements for the Record Crossing Sump area and Hampton Pumping Plant. The alternates investigated for the Record Crossing Sump area included increased pumping capacity, increased sump storage, pressure sewers, and a floodwall. The goal of the alternatives was to decrease the predicted 100-year, 24-hour maximum sump stage to the City of Dallas design sump elevation of 405.0 ft.

4.5.1 Pump Stations

No alternative evaluated for Record Crossing Sump eliminated the need for an additional pump station to be built at Hampton Pumping Plant. The proposed new Hampton pump station will include five equally-sized vertical turbine pumps, one of which is a backup. The pump station will include all the necessary mechanical and electrical equipment to operate the pump station efficiently and with minimum maintenance. This will include equipment such as automated self cleaning bar screens with a trash conveyor system, CCTV cameras, and SCADA equipment for remote operation and monitoring.

The existing Hampton pump stations were evaluated by an electrical engineer and architect on the project team and the following rehabilitation recommendations were made:

Old Hampton Station (OHX)

- Replace four existing pumps with new 50,000 gpm pumps
- Replace roof and gravel stop
- Replace wall louvers: fixed and operable
- Paint interior (walls, floor, and ceiling)
- Replace roof hatch hardware
- Replace double door, frame, and hardware
- Repair vehicular gates
- Replace Transformers and panelboards
- New 480V motor control center
- New conduit and wire
- New Lighting
- Reconnect controls and SCADA system

The probable cost for rehabilitation of the existing Old Hampton Pump Station (OHX) is approximately $5.0 million. A detailed preliminary opinion of probable costs for the rehabilitation of OHX is included in Appendix A. In addition, the OHX outfall will need to be extended approximately 300 feet to avoid conflicts with the proposed Trinity Parkway at a probable cost of approximately $300,000.

New Hampton Station (NHX)

- Replace roof
- Paint and repair sliding door
- Replace damaged louvers
- Clean and paint exterior concrete
- Paint Interior (walls, floor, and ceiling)
- Grind slope in concrete at door (to prevent water from running under door during rain)
- Replace 5 KV switchgear
- Replace transformers and panelboards
- New 480V motor control center
- New conduit and wire
- New lighting
- Reconnect controls and SCADA system

The probable cost for the rehabilitation of the existing New Hampton Pump Station (NHX) is approximately $597,000. A detailed preliminary opinion of probable costs for the renovation of NHX is included in Appendix A. In addition, the NHX outfall will need to be extended approximately 300 feet to avoid conflicts with the proposed Trinity Parkway at a probable cost of approximate $300,000

The curve of required additional pumping capacity versus addition sump volume for Hampton Pumping Plant and Record Crossing Sump is shown in Figure 4.4. From Figure 4.4 it can be seen that if no additional sump storage is provided, the required additional pumping capacity is 500,000 gpm; if 639 acre-feet of additional sump storage is provided, no additional pumping capacity is required.

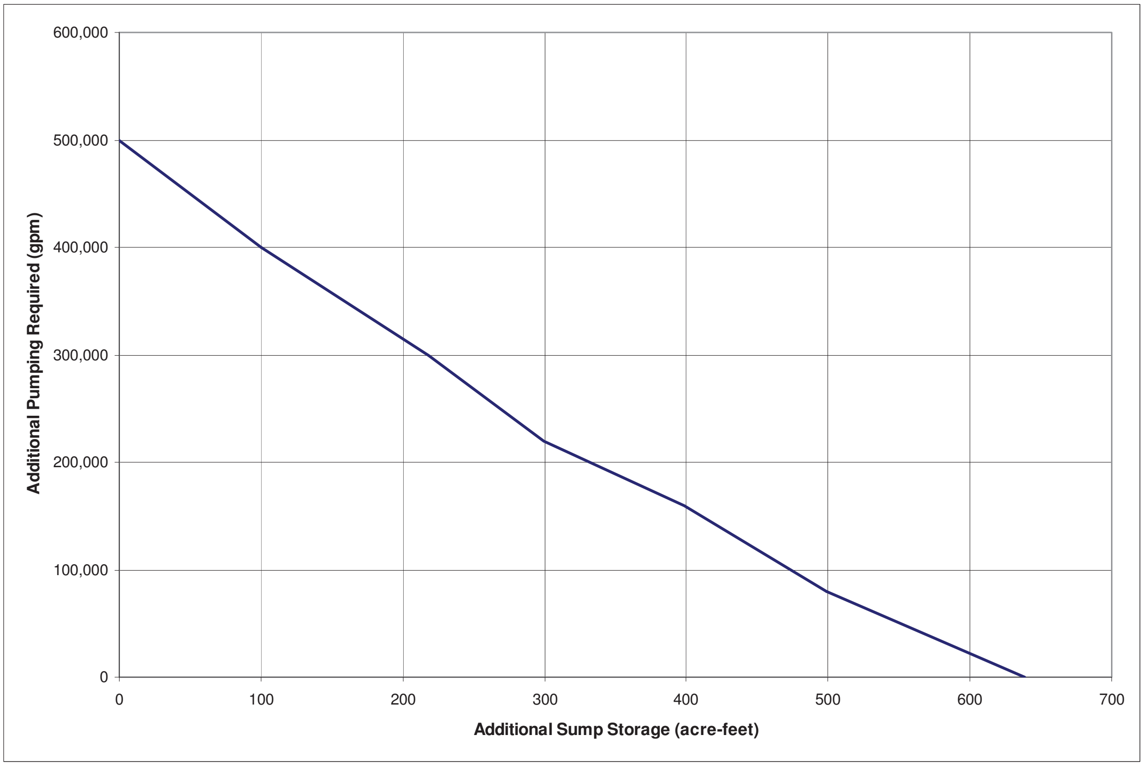

Figure 4.4 - Additional Pumping vs. Additional Sump Storage for Record Crossing Sump

Seven different alternates were formulated to evaluate the cost of additional sump storage versus additional required pumping. The alternates are summarized in Table 4.5. The estimated pump station costs in Table 4.5 do not include the probable cost of extending the pump station outfall to avoid conflicts with the proposed Trinity Parkway, which is constant for all alternates which include a new pump station.

Table 4.5 - Summary of Alternates for Record Crossing Sump/Hampton Pumping Plant

Alternate	Volume of Additional Sump Storage (ac-ft)	Probable Cost of Additional Sump Storage (millions)	Additional Pumping Capacity Required (gpm)	Probable Cost of Additional Pumping (millions)	Total Probable Cost (millions)
RC-1	0	$0	500,000	$31.3	$31.3
RC-2	100	$12.6	400,000	$29.0	$41.6
RC-3	218	$27.5	300,000	$26.8	$54.3
RC-4	299	$37.8	220,000	$25.0	$62.8
RC-5	399	$50.4	160,000	$23.6	$74.0
RC-6	499	$63.0	80,000	$21.8	$84.8
RC-7	639	$80.7	0	$0	$80.7

The probable costs of these alternatives are shown graphically in Figure 4.5. Figure 4.5 shows that the probable cost of additional sump storage increases at a faster rate than the probable cost of additional pumping decreases. The alternate with the lowest estimated cost is RC-1 at approximately $31.3 million, plus an additional $900,000 required to construct an extended outfall to avoid conflicts with the proposed Trinity Parkway. Therefore, the total probable cost of alternate RC-1 is approximately $32.2 million.

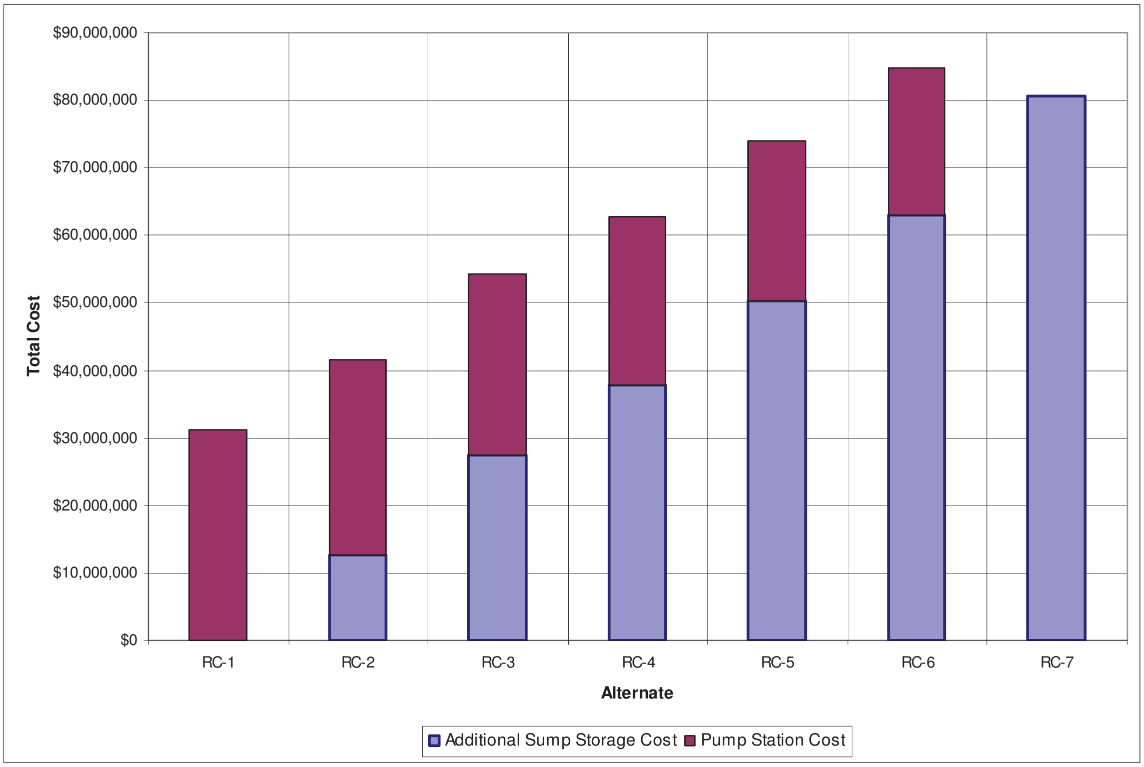

Figure 4.5 - Additional Sump Storage Cost vs. Pump Station Cost for Record Crossing Sump

4.5.2 Pressure Sewers

Based on a thorough analysis of the Record Crossing Sump drainage basin, a potential location for a new pressure sewer was identified on Knights Branch. The proposed alignment of the Knights Branch Pressure Sewer runs along Inwood Road and is shown on Exhibit 25. The length of the proposed pressure sewer is approximately 2 miles, and the proposed section is a 15-ft diameter circular conduit. The capacity of the proposed pressure sewer is approximately 3,500 cfs with a tailwater elevation in the Dallas Floodway computed as described in Section 2.3.4. At a preliminary probable tunneling cost of $6,480/ft, the total probable cost of the Knights Branch Pressure Sewer is in excess of $72 million. This far exceeds the probable cost of the proposed new Baker pump station described in Section 4.5.1. Therefore, it would be economically infeasible to construct the Knights Branch Pressure Sewer.

4.5.3 Flood Walls

The floodwall alternative to prevent overflow from Record Crossing Sump in Hampton-Oak Lawn Sump is discussed in Section 4.3.3. No floodwall construction is recommended at this time.

4.5.4 Recommendations

As shown in Table 4.5 and Figure 4.5, the least expensive alternate for Record Crossing Sump is Alternate RC-1, at a probable cost of approximately $32.2 million. Therefore, Alternate RC-1 is the recommended solution for Hampton-Oak Lawn Sump. The rehabilitation of the existing Hampton pump stations are also recommended, at a total probable cost of approximately $6.1 million including the required outfall extensions. The total probable cost for these improvements is approximately $38.3 million. Detailed preliminary opinions of probable costs for the recommended improvements are included in Appendix A.

4.6 EXISTING PRESSURE SEWERS

To prevent conflicts with the proposed Balanced Vision Plan for the Trinity River Corridor, the existing East Levee pressure sewer outfalls will have to be extended. Based on conceptual drawings of the Balanced Vision Plan, the Woodall Rodgers and Dallas Branch pressure sewer outfalls will have to be extended approximately 1,100 ft to pass under proposed man-made lakes in the Dallas Floodway. The Turtle Creek and Belleview pressure sewer outfalls will have to be extended approximately 300 ft to pass under the proposed Trinity Parkway embankment. The probable costs associated with these outfall extensions are summarized in Table 4.6.

Table 4.6 - Summary of Estimated Costs for Existing Pressure Sewer Outfall Extensions

Item	Probable Cost
Extend Turtle Creek Pressure Sewer 300 ft	$0.9 million
Extend Woodall Rodgers Pressure Sewer 1,100 ft	$3.3 million
Extend Dallas Branch Pressure Sewer 1,100 ft	$3.3 million
Extend Belleview Pressure 300 ft	$0.9 million
Total	$8.4 million

4.7 SUMMARY OF RECOMMENDATIONS

Table 4.7 contains a summary of all recommendations for the East Levee interior drainage system, including present-day probable costs and future probable costs escalated at 5% to a midpoint of construction 5 years in the future. In early 2006, these recommendations were used to develop the City of Dallas Public Works and Transportation Department recommendations for the City's November 2006 bond package.

Table 4.7 - Summary of Recommendations for the East Levee Interior Drainage System

ITEM DESCRIPTION	Current Probable Cost (millions)	Escalated 5% per Year for 5 Years (millions)
multiplier =	*1.000*	*1.276*
ABLE PUMPING PLANT		
New 400,000 gpm Pump Station	$29.0	$37.0
Extend New Pump Station outfall (1,100 ft)	$3.3	$4.2
Replace Existing Culverts	$3.0	$3.8
Rehab existing Pump Stations	$6.5	$8.3
Extend existing SAX outfall (1,100 ft)	$1.1	$1.4
Extend existing LAX outfall (1,100 ft)	$1.1	$1.4
SUBTOTAL	$44.0	$56.1
BAKER PUMPING PLANT		
New 700,000 gpm Pump Station	$35.8	$45.7
Extend New Pump Station outfall (300 ft)	$0.9	$1.1
Rehab existing Pump Station (NBX only)	$0.7	$0.9
Extend existing NBX outfall (300 ft)	$0.3	$0.4
SUBTOTAL	$37.7	$48.1
HAMPTON PUMPING PLANT		
New 500,000 gpm Pump Station	$31.3	$39.9
Extend New Pump Station outfall (300 ft)	$0.9	$1.1
Rehab existing Pump Stations	$5.5	$7.0
Extend existing NHX outfall (300 ft)	$0.3	$0.4
Extend existing OHX outfall (300 ft)	$0.3	$0.4
SUBTOTAL	$38.3	$48.9
NOBLES BRANCH SUMP		
Add 3 new 60-inch gated culverts	$1.0	$1.3
SUBTOTAL	$1.0	$1.3
PRESSURE SEWERS		
Extend existing WOODALL RODGERS (1,100 ft)	$3.3	$4.2
Extend existing DALLAS BRANCH (1,100 ft)	$3.3	$4.2
Extend existing TURTLE CREEK (300 ft)	$0.9	$1.1
Extend existing BELLEVIEW (300 ft)	$0.9	$1.1
SUBTOTAL	$8.4	$10.7
TOTAL PROBABLE COST	**$129**	**$165**

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Rehabilitation of Existing Small Able Pump Station

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$0	$0	$5,000	$5,000	$5,000
Subtotal for Division 1				$0		$5,000	$5,000
Division 2 - Site Work							
Repair vehicular gates	1	LS	$500	$500	$200	$200	$700
Subtotal for Division 2				$500		$200	$700
Division 3 - Concrete							
Subtotal Division 3				$0		$0	$0
Division 4 - Masonry							
Re-brick closed openings to provide weather tight enclosure	1	LS	$800	$800	$200	$200	$1,000
Povide masonry to close window currently blocked with steel plate	1	LS	$200	$200	$200	$200	$400
Subtotal for Division 4				$1,000		$400	$1,400
Division 5 - Metals							
Subtotal for Division 5				$0		$0	$0
Division 6 - Carpentry							
Subtotal for Division 6				$0		$0	$0
Division 7 - Thermal and Moisture Protection							
Replace and repair tile coping and roof	1	LS	$17,000	$17,000	$2,550	$2,550	$19,550
Replace roof hatch hardware	1	LS	$2,000	$2,000	$500	$500	$2,500
Subtotal for Division 7				$19,000		$3,050	$22,050
Division 8 - Doors and Windows							
Replace steel sliding door and hardware	1	LS	$3,000	$3,000	$500	$500	$3,500
Replace wall louvers: fixed and operable	1	LS	$1,200	$1,200	$600	$500	$1,700
Subtotal for Division 8				$4,200		$1,000	$5,200
Division 9- Finishes							
Paint interior (walls, floors, & ceiling)	1	LS	$1,900	$1,900	$500	$500	$2,400
Glaze and paint steel casement windows	1	LS	$300	$300	$150	$150	$450
Subtotal for Division 9				$2,200		$650	$2,850
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Replace 2 - 40,000 gpm pumps (installed in 1960's)	1	LS	$1,170,000	$1,170,000	$130,000	$130,000	$1,300,000
Subtotal for Division 11				$1,170,000		$130,000	$1,300,000
Division 12 - Furnishings							
Subtotal for Division 12				$0		$0	$0
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
Subtotal for Division 14				$0		$0	$0
Division 15 - Mechanical							
HVAC	1	LS	$55,000	$55,000	$5,000	$5,000	$60,000
Subtotal for Division 15				$55,000		$5,000	$60,000
Division 16 - Electrical							
Replace transformers and panelboards	1	LS	$19,250	$19,250	$2,888	$2,888	$22,138
New 480V motor control center	1	LS	$57,500	$57,500	$8,625	$8,625	$66,125
New conduit & wire	1	LS	$16,450	$16,450	$2,468	$2,468	$18,918
New lighting	1	LS	$1,920	$1,920	$288	$288	$2,208
Reconnect controls and SCADA system	1	LS	$15,000	$15,000	$2,250	$2,250	$17,250
Subtotal for Division 16				$110,120		$16,518	$126,638
Division 17 - Instrumentation & Control							
Subtotal for Division 17				$0		$0	$0
Subtotal				$1,362,020		$161,818	$1,523,838
Contingencies (25%)							$380,960
Subtotal							$1,904,798
Contractor's Profit on Material (10%)							$170,253
Materials Testing (2%)							$38,096
Construction Inspection (5%)							$95,240
Engineering (10%)							$190,480
Const. Work Effort							$2,398,866
Escallation to Midpoint of Construction @ 5% for 5 yrs							$662,807
Total Base Bid Estimated Cost							$3,061,672

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Rehabilitation of Existing Large Able Pump Station

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$0	$0	$5,000	$5,000	$5,000
Subtotal for Division 1				$0		$5,000	$5,000
Division 2 - Site Work							
Repair vehicular gates	1	LS	$500	$500	$200	$200	$700
Subtotal for Division 2				$500		$200	$700
Division 3 - Concrete							
Subtotal Division 3				$0		$0	$0
Division 4 - Masonry							
Clean brick and graffiti	1	LS	$500	$500	$100	$100	$600
Subtotal for Division 4				$500		$100	$600
Division 5 - Metals							
Subtotal for Division 5				$0		$0	$0
Division 6 - Carpentry							
Subtotal for Division 6				$0		$0	$0
Division 7 - Thermal and Moisture Protection							
Replace roof and gravel stop	1	LS	$21,000	$21,000	$2,550	$2,500	$23,500
Subtotal for Division 7				$21,000		$2,500	$23,500
Division 8 - Doors and Windows							
Replace double doors, frame, and hardware	1	LS	$1,500	$1,500	$500	$500	$2,000
Replace single door, frame and hardware	1	LS	$800	$800	$500	$500	$1,300
Replace wall louvers: fixed and operable	1	LS	$1,400	$1,400	$600	$600	$2,000
Replace roof hatch hardware	1	LS	$2,000	$2,000	$500	$500	$2,500
Subtotal for Division 8				$5,700		$2,100	$7,800
Division 9- Finishes							
Paint interior (walls, floors, & ceiling)	1	LS	$2,800	$2,800	$500	$500	$3,300
Subtotal for Division 9				$2,800		$500	$3,300
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Replace 3 - 47,000 gpm pumps (installed 1950's)	1	LS	$2,160,000	$2,160,000	$240,000	$240,000	$2,400,000
Subtotal for Division 11				$2,160,000		$240,000	$2,400,000
Division 12 - Furnishings							
Subtotal for Division 12				$0		$0	$0
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
Subtotal for Division 14				$0		$0	$0
Division 15 - Mechanical							
HVAC	1	LS	$55,000	$55,000	$5,000	$5,000	$60,000
Subtotal for Division 15				$55,000		$5,000	$60,000
Division 16 - Electrical							
Replace transformers and panelboards	1	LS	$33,750	$33,750	$5,063	$5,063	$38,813
New 480V motor control center	1	LS	$47,500	$47,500	$7,125	$7,125	$54,625
New conduit & wire	1	LS	$18,250.00	$18,250	$2,738	$2,738	$20,988
New lighting	1	LS	$1,780	$1,780	$267	$267	$2,047
Reconnect controls and SCADA system	1	LS	$15,000	$15,000	$2,250	$2,250	$17,250
Subtotal for Division 16				$116,280		$17,442	$133,722
Division 17 - Instrumentation & Control							
Subtotal for Division 17				$0		$0	$0
Subtotal				$2,361,780		$270,642	$2,634,622
Contingencies (25%)							$658,656
Subtotal							$3,293,278
Contractor's Profit on Material (10%)							$295,223
Materials Testing (2%)							$65,866
Construction Inspection (5%)							$164,664
Engineering (10%)							$329,328
Const. Work Effort							$4,148,357
Escalation to Midpoint of Construction @ 5% for 5 yrs							$1,146,191
Total Base Bid Estimated Cost							$5,294,548

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
New 400,000 gpm Storm Water Pump Station @ Able Pumping Plant

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$262,620	$262,620	$29,180	$29,180	$291,800
Subtotal for Division 1				$262,620		$29,180	$291,800
Division 2 - Site Work							
Site work	1	LS	$2,848,410	$2,848,410	$316,490	$316,490	$3,164,900
Subtotal for Division 2				$2,848,410		$316,490	$3,164,900
Division 3 - Concrete							
Concrete (2500 CY)	1	LS	$718,573	$718,573	$79,841	$79,841	$798,414
Subtotal Division 3				$718,573		$79,841	$798,414
Division 4 - Masonry							
Brick	10,000	SF	$10.00	$100,000	$0	$0	$100,000
Subtotal for Division 4				$100,000		$0	$100,000
Division 5 - Metals							
Metals	1	LS	$79,290	$79,290	$8,810	$8,810	$88,100
Subtotal for Division 5				$79,290		$8,810	$88,100
Division 6 - Carpentry							
Misc.	1	LS	$25,000	$25,000	$50,000	$50,000	$75,000
Subtotal for Division 6				$25,000		$50,000	$75,000
Division 7 - Thermal and Moisture Protection							
Thermal and moisture proctection	1	LS	$307,195	$307,195	$34,133	$34,133	$341,328
Subtotal for Division 7				$307,195		$34,133	$341,328
Division 8 - Doors and Windows							
Doors and windows	1	LS	$6,853	$6,853	$761	$761	$7,614
Subtotal for Division 8				$6,853		$761	$7,614
Division 9- Finishes							
Finishes	1	LS	$156,474	$156,474	$17,386	$17,386	$173,860
Subtotal for Division 9				$156,474		$17,386	$173,860
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Equipment (includes 4-100,00 gpm pumps + 1 spare)	1	LS	$7,866,000	$7,866,000	$874,000	$874,000	$8,740,000
Subtotal for Division 11				$7,866,000		$874,000	$8,740,000
Division 12 - Furnishings							
Furnishings	1	LS	$7,667	$7,667	$852	$852	$8,519
Subtotal for Division 12				$7,667		$852	$8,519
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
30 ton bridge crane	1	LS	$157,500	$157,500	$17,500	$17,500	$175,000
Subtotal for Division 14				$157,500		$17,500	$175,000
Division 15 - Mechanical							
Mechanical	1	LS	$157,500	$157,500	$17,500	$17,500	$175,000
Subtotal for Division 15				$157,500		$17,500	$175,000
Division 16 - Electrical							
Electrical	1	LS	$2,756,925	$2,756,925	$306,325	$306,325	$3,063,250
Subtotal for Division 16				$2,756,925		$306,325	$3,063,250
Division 17 - Instrumentation & Control							
Instrumentation & control	1	LS	$668,700	$668,700	$74,300	$74,300	$743,000
Subtotal for Division 17				$668,700		$74,300	$743,000
Subtotal				$16,118,707		$1,827,079	$17,945,785
Contingencies (25%)							4,486,446
Subtotal							$22,432,231
Contractor's Profit on Material (10%)							$2,014,838
Prime Profit on Subcontractors							$800,000
Materials Testing (2%)							$448,645
Construction Inspection (5%)							$1,121,612
Engineering (10%)							$2,243,223
Const. Work Effort							$29,050,549
Escallation to Midpoint of Construction @ 5% for 5 yrs							$8,029,430
Total Base Bid Estimated Cost							$37,089,979

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Rehabilitation of Existing New Baker Pump Station

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$0	$0	$5,000	$5,000	$5,000
Subtotal for Division 1				$0		$5,000	$5,000
Division 2 - Site Work							
Repair vehicular gates	1	LS	$500	$500	$200	$200	$700
Subtotal for Division 2				$500		$200	$700
Division 3 - Concrete							
Subtotal Division 3				$0		$0	$0
Division 4 - Masonry							
Subtotal for Division 4				$0		$0	$0
Division 5 - Metals							
Subtotal for Division 5				$0		$0	$0
Division 6 - Carpentry							
Subtotal for Division 6				$0		$0	$0
Division 7 - Thermal and Moisture Protection							
Replace roof	1	LS	$25,500	$25,500	$2,550	$2,500	$28,000
Subtotal for Division 7				$25,500		$2,500	$28,000
Division 8 - Doors and Windows							
Paint and repair sliding door	1	LS	$500	$500	$200	$200	$700
Fix damaged sceen on louvers	1	LS	$300	$300	$150	$150	$450
Replace roof hatch hardware	1	LS	$2,000	$2,000	$500	$500	$2,500
Grind slope in concrete at door	1	LS	$1,400	$1,400	$500	$500	$1,900
Subtotal for Division 8				$4,200		$1,350	$5,550
Division 9- Finishes							
Paint interior (walls, floors, & ceiling)	1	LS	$3,900	$3,900	$500	$500	$4,400
Clean and paint exterior concrete	1	LS	$1,500	$1,500	$500	$500	$2,000
Subtotal for Division 9				$5,400		$1,000	$6,400
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Subtotal for Division 11				$0		$0	$0
Division 12 - Furnishings							
Subtotal for Division 12				$0		$0	$0
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
Subtotal for Division 14				$0		$0	$0
Division 15 - Mechanical							
HVAC	1	LS	$55,000	$55,000	$5,000	$5,000	$60,000
Subtotal for Division 15				$55,000		$5,000	$60,000
Division 16 - Electrical							
Replace 5 kV switchgear	1	LS	$151,000	$151,000	$22,650	$22,650	$173,650
Replace transformers and panelboards	1	LS	$33,750	$33,750	$5,063	$5,063	$38,813
New 480V motor control center	1	LS	$47,500	$47,500	$7,125	$7,125	$54,625
New conduit & wire	1	LS	$23,700	$23,700	$3,555	$3,555	$27,255
New lighting	1	LS	$11,155	$11,155	$1,673	$1,673	$12,828
Reconnect controls and SCADA system	1	LS	$15,000	$15,000	$2,250	$2,250	$17,250
Subtotal for Division 16				$282,105		$42,316	$324,421
Division 17 - Instrumentation & Control							
Subtotal for Division 17				$0		$0	$0
Subtotal				$372,705		$57,366	$430,071
Contingencies (25%)							$107,518
Subtotal							$537,588
Contractor's Profit on Material (10%)							$46,588
Materials Testing (2%)							$10,752
Construction Inspection (5%)							$26,879
Engineering (10%)							$53,759
Const. Work Effort							$675,547
Escalation to Midpoint of Construction @ 5% for 5 yrs							$186,659
Total Base Bid Estimated Cost							$862,226

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
New 700,000 gpm Storm Water Pump Station @ Baker Pumping Plant

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
General conditions	1	LS	$262,620	$262,620	$29,180	$29,180	$291,800
Subtotal for Division 1				**$262,620**		**$29,180**	**$291,800**
Division 2 - Site Work							
Site work	1	LS	$2,891,880	$2,891,880	$321,320	$321,320	$3,213,200
Subtotal for Division 2				**$2,891,880**		**$321,320**	**$3,213,200**
Division 3 - Concrete							
Concrete (4550 CY)	1	LS	$1,304,483	$1,304,483	$144,943	$144,943	$1,449,426
Subtotal Division 3				**$1,304,483**		**$144,943**	**$1,449,426**
Division 4 - Masonary							
Brick	13,500	SF	$10	$135,000	$0	$0	$135,000
Subtotal for Division 4				**$135,000**		**$0**	**$135,000**
Division 5 - Metals							
Metals	1	LS	$89,478	$89,478	$9,942	$9,942	$99,420
Subtotal for Division 5				**$89,478**		**$9,942**	**$99,420**
Division 6 - Carpentry							
Misc.	1	LS	$25,000	$25,000	$50,000	$50,000	$75,000
Subtotal for Division 6				**$25,000**		**$50,000**	**$75,000**
Division 7 - Thermal and Moisture Protection							
Thermal and moisture protection	1	LS	$315,457	$315,457	$35,051	$35,051	$350,508
Subtotal for Division 7				**$315,457**		**$35,051**	**$350,508**
Division 8 - Doors and Windows							
Doors and windows	1	LS	$6,853	$6,853	$761	$761	$7,614
Subtotal for Division 8				**$6,853**		**$761**	**$7,614**
Division 9- Finishes							
Finishes	1	LS	$158,058	$158,058	$17,562	$17,562	$175,620
Subtotal for Division 9				**$158,058**		**$17,562**	**$175,620**
Division 10 - Specialities							
Subtotal for Division 10				**$0**		**$0**	**$0**
Division 11 - Equipment							
Equipment (includes 4-175,00 gpm pumps + 1 spare)	1	LS	$11,070,000	$11,070,000	$1,230,000	$1,230,000	$12,300,000
Subtotal for Division 11				**$11,070,000**		**$1,230,000**	**$12,300,000**
Division 12 - Furnishings							
Furnishings	1	LS	$7,667	$7,667	$852	$852	$8,519
Subtotal for Division 12				**$7,667**		**$852**	**$8,519**
Division 13 - Special Construction							
Subtotal for Division 13				**$0**		**$0**	**$0**
Division 14 - Conveying Systems							
30 ton bridge crane	1	Ea	$150,000	$150,000	$25,000	$25,000	$175,000
Subtotal for Division 14				**$150,000**		**$25,000**	**$175,000**
Division 15 - Mechanical							
HVAC - 135,000 cfm	1	LS	$150,000	$150,000	$25,000	$25,000	$175,000
Subtotal for Division 15				**$150,000**		**$25,000**	**$175,000**
Division 16 - Electrical							
Electrical	1	LS	$2,756,925	$2,756,925	$306,325	$306,325	$3,063,250
Subtotal for Division 16				**$2,756,925**		**$306,325**	**$3,063,250**
Division 17 - Instrumentation & Control							
Instrumentation & control	1	LS	$668,700	$668,700	$74,300	$74,300	$743,000
Subtotal for Division 17				**$668,700**		**$74,300**	**$743,000**
Subtotal				**$19,992,121**		**$2,270,236**	**$22,262,357**
Contingencies (25%)							$5,565,589
Subtotal							**$27,827,946**
Contractor's Profit on Material (10%)							$2,499,015
Prime Profit on Subcontractors							$800,000
Materials Testing (2%)							$556,559
Construction Inspection (5%)							$1,391,397
Engineering (10%)							$2,782,795
Const. Work Effort							**$35,857,712**
Escallation to Midpoint of Construction @ 5% for 5 yrs							$9,907,486
Total Base Bid Estimated Cost							**$45,765,198**

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Rehabilitation of Existing Old Hampton Pump Station

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$0	$0	$5,000	$5,000	$5,000
Subtotal for Division 1				$0		$5,000	$5,000
Division 2 - Site Work							
Repair vehicular gates	1	LS	$500	$500	$200	$200	$700
Subtotal for Division 2				$500		$200	$700
Division 3 - Concrete							
Subtotal Division 3				$0		$0	$0
Division 4 - Masonry							
Subtotal for Division 4				$0		$0	$0
Division 5 - Metals							
Subtotal for Division 5				$0		$0	$0
Division 6 - Carpentry							
Subtotal for Division 6				$0		$0	$0
Division 7 - Thermal and Moisture Protection							
Replace roof and gravel stop	1	LS	$21,000	$21,000	$2,550	$2,550	$23,550
Subtotal for Division 7				$21,000		$2,550	$23,550
Division 8 - Doors and Windows							
Replace wall louvers: fixed and operable	1	LS	$1,400	$1,400	$200	$200	$1,600
Replace roof hatch hardware	1	LS	$2,000	$2,000	$500	$500	$2,500
Replace double door, frame, and hardware	1	LS	$1,500	$1,500	$500	$500	$2,000
Subtotal for Division 8				$4,900		$1,200	$6,100
Division 9- Finishes							
Paint interior (walls, floors, & ceiling)	1	LS	$3,900	$3,900	$500	$500	$4,400
Clean and paint exterior concrete	1	LS	$1,500	$1,500	$500	$500	$2,000
Subtotal for Division 9				$1,500		$500	$2,000
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Replace 4 - 50,000 gpm pumps (installed in 1950's)	1	LS	$2,700,000	$2,700,000	$300,000	$300,000	$3,000,000
Subtotal for Division 11				$2,700,000		$300,000	$3,000,000
Division 12 - Furnishings							
Subtotal for Division 12				$0		$0	$0
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
Subtotal for Division 14				$0		$0	$0
Division 15 - Mechanical							
HVAC	1	LS	$55,000	$55,000	$5,000	$5,000	$60,000
Subtotal for Division 15				$55,000		$5,000	$60,000
Division 16 - Electrical							
Replace transformers and panelboards	1	LS	$33,750	$33,750	$5,063	$5,063	$38,813
New 480V motor control center	1	LS	$27,500	$27,500	$4,125	$4,125	$31,625
New conduit & wire	1	LS	$22,200	$22,200	$3,330	$3,330	$25,530
New lighting	1	LS	$2,060	$2,060	$309	$309	$2,369
Reconnect controls and SCADA system	1	LS	$15,000	$15,000	$2,250	$2,250	$17,250
Subtotal for Division 16				$100,510		$15,077	$115,587
Division 17 - Instrumentation & Control							
Subtotal for Division 17				$0		$0	$0
Subtotal				$2,883,410		$329,527	$3,212,937
Contingencies (25%)							$803,234
Subtotal							$4,016,171
Contractor's Profit on Material (10%)							$360,426
Materials Testing (2%)							$80,323
Construction Inspection (5%)							$200,809
Engineering (10 %)							$401,617
Const. Work Effort							$5,059,346
Escallation to Midpoint of Construction @ 5% for 5 yrs							$1,397,897
Total Base Bid Estimated Cost							$6,457,243

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Rehabilitation of Existing New Hampton Pump Station

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$0	$0	$5,000	$5,000	$5,000
Subtotal for Division 1				$0		$5,000	$5,000
Division 2 - Site Work							
Subtotal for Division 2				$0		$0	$0
Division 3 - Concrete							
Subtotal Division 3				$0		$0	$0
Division 4 - Masonry							
Subtotal for Division 4				$0		$0	$0
Division 5 - Metals							
Subtotal for Division 5				$0		$0	$0
Division 6 - Carpentry							
Subtotal for Division 6				$0		$0	$0
Division 7 - Thermal and Moisture Protection							
Replace roof	1	LS	$25,000	$25,000	$2,550	$2,550	$27,550
Subtotal for Division 7				$25,000		$2,550	$27,550
Division 8 - Doors and Windows							
Paint and repair sliding door	1	LS	$500	$500	$200	$200	$700
Replace damaged louvers	1	LS	$7,500	$7,500	$500	$1,000	$8,500
Grind slope in concrete at door	1	LS	$1,400	$1,400	$500	$1,000	$2,400
Subtotal for Division 8				$9,400		$2,200	$11,600
Division 9- Finishes							
Paint interior (walls, floors, & ceiling)	1	LS	$3,900	$3,900	$500	$500	$4,400
Clean and paint exterior concrete	1	LS	$1,500	$1,500	$500	$500	$2,000
Subtotal for Division 9				$5,400		$1,000	$6,400
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Subtotal for Division 11				$0		$0	$0
Division 12 - Furnishings							
Subtotal for Division 12				$0		$0	$0
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
Subtotal for Division 14				$0		$0	$0
Division 15 - Mechanical							
HVAC	1	LS	$55,000	$55,000	$5,000	$5,000	$60,000
Subtotal for Division 15				$55,000		$5,000	$60,000
Division 16 - Electrical							
Replace 5kV switchgear	1	LS	$113,500	$113,500	$17,025	$17,025	$130,525
Replace transformers and panelboards	1	LS	$33,750	$33,750	$5,063	$5,063	$38,813
New 480V motor control center	1	LS	$37,500	$37,500	$5,625	$5,625	$43,125
New conduit & wire	1	LS	$23,700	$23,700	$3,555	$3,555	$27,255
New lighting	1	LS	$10,780	$10,780	$1,617	$1,617	$12,397
Reconnect controls and SCADA system	1	LS	$15,000	$15,000	$2,250	$2,250	$17,250
Subtotal for Division 16				$234,230		$35,135	$269,365
Division 17 - Instrumentation & Control							
Subtotal for Division 17				$0		$0	$0
Subtotal				$329,030		$50,885	$379,915
Contingencies (25%)							$94,979
Subtotal							$474,893
Contractor's Profit on Material (10%)							$41,129
Materials Testing (2%)							$9,498
Construction Inspection (5%)							$23,745
Engineering (10%)							$47,489
Const. Work Effort							$596,75[illegible]
Escalation to Midpoint of Construction @ 5% for 5 yrs							$164,883
Total Base Bid Estimated Cost							[illegible]81,637

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
New 500,000 gpm Storm Water Pump Station @ Hampton Pumping Plant

	QTY	UNIT	UNIT MAT'L	TOTAL MAT'L	UNIT LABOR	TOTAL LABOR	TOTAL COST
Division 1 - General Conditions							
Mobilization	1	LS	$262,620	$262,620	$29,180	$29,180	$291,800
Subtotal for Division 1				$262,620		$29,180	$291,800
Division 2 - Site Work							
Site work	1	LS	$2,891,880	$2,891,880	$321,320	$321,320	$3,213,200
Subtotal for Division 2				$2,891,880		$321,320	$3,213,200
Division 3 - Concrete							
Concrete (3000 CY)	1	LS	$864,000	$864,000	$96,000	$96,000	$960,000
Subtotal Division 3				$864,000		$96,000	$960,000
Division 4 - Masonary							
Brick	13,500	SF	$10	$135,000	$0	$0	$135,000
Subtotal for Division 4				$135,000		$0	$135,000
Division 5 - Metals							
Metals	1	LS	$89,478	$89,478	$9,942	$9,942	$99,420
Subtotal for Division 5				$89,478		$9,942	$99,420
Division 6 - Carpentry							
Misc.	1	LS	$25,000	$25,000	$50,000	$50,000	$75,000
Subtotal for Division 6				$25,000		$50,000	$75,000
Division 7 - Thermal and Moisture Protection							
Thermal and moisture protection	1	LS	$315,457	$315,457	$35,051	$35,051	$350,508
Subtotal for Division 7				$315,457		$35,051	$350,508
Division 8 - Doors and Windows							
Doors and windows	1	LS	$6,853	$6,853	$761	$761	$7,614
Subtotal for Division 8				$6,853		$761	$7,614
Division 9- Finishes							
Finishes	1	LS	$158,058	$158,058	$17,562	$17,562	$175,620
Subtotal for Division 9				$158,058		$17,562	$175,620
Division 10 - Specialities							
Subtotal for Division 10				$0		$0	$0
Division 11 - Equipment							
Equipment (includes 4-125,000 gpm pumps + 1 spare)	1	LS	$8,910,000	$8,910,000	$990,000	$990,000	$9,900,000
Subtotal for Division 11				$8,910,000		$990,000	$9,900,000
Division 12 - Furnishings							
Furnishings	1	LS	$7,667	$7,667	$852	$852	$8,519
Subtotal for Division 12				$7,667		$852	$8,519
Division 13 - Special Construction							
Subtotal for Division 13				$0		$0	$0
Division 14 - Conveying Systems							
30 ton bridge crane	1	Ea	$150,000	$150,000	$25,000	$25,000	$175,000
Subtotal for Division 14				$150,000		$25,000	$175,000
Division 15 - Mechanical							
HVAC - 135,000 cfm	1	LS	$150,000	$150,000	$25,000	$25,000	$175,000
Subtotal for Division 15				$150,000		$25,000	$175,000
Division 16 - Electrical							
Electrical	1	LS	$2,756,925	$2,756,925	$306,325	$306,325	$3,063,250
Subtotal for Division 16				$2,756,925		$306,325	$3,063,250
Division 17 - Instrumentation & Control							
Instrumentation & control	1	LS	$668,700	$668,700	$74,300	$74,300	$743,000
Subtotal for Division 17				$668,700		$74,300	$743,000
Subtotal				$17,391,638		$1,981,293	$19,372,931
Contingencies (25%)							$4,843,233
Subtotal							$24,216,164
Contractor's Profit on Material (10%)							$2,173,955
Prime Profit on Subcontractors							$800,000
Materials Testing (2%)							$484,323
Construction Inspection (5%)							$1,210,808
Engineering (10%)							$2,421,616
Const. Work Effort							$31,306,866
Escallation to Midpoint of Construction @ 5% & 5 yrs.							$8,650,087
Total Base Bid Estimated Cost							$39,956,953

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Replacement of Able Sump Culverts

	QTY	UNIT	UNIT COST	TOTAL COST
10' x 10' RCBC	1,950	LF	$600	$1,170,000
10' x 8' RCBC	510	LF	$500	$255,000
10' x 6' RCBC	510	LF	$450	$229,500
Headwalls and wingwalls	1	LS	$250,000	$250,000
Subtotal				**$1,904,500**
Contingencies (25%)				476,125
Subtotal				**$2,380,625**
Contractor's Profit on Material (10%)				$238,063
Materials Testing (2%)				$47,613
Construction Inspection (5%)				$119,031
Engineering (10%)				$238,063
Const. Work Effort				**$3,023,394**
Escallation to Midpoint of Construction @ 5% for 5 yrs				$835,364
Total Base Bid Estimated Cost				**$3,858,757**

INTERIOR LEVEE DRAINAGE STUDY, PHASE 1
Preliminary Opinion of Probable Cost
Addition to Grauwyler Gated Conduit Structure

	QTY	UNIT	UNIT COST	TOTAL COST
60" RCP	480	LF	$200	$96,000
Motor-operated rising stem sluice gates	3	Ea.	$150,000	$450,000
Concrete headwall and gate structure	1	LS	$100,000	$100,000
Subtotal				**$646,000**
Contingencies (25%)				161,500
Subtotal				**$807,500**
Contractor's Profit on Material (10%)				$80,750
Materials Testing (2%)				$16,150
Construction Inspection (5%)				$40,375
Engineering (10%)				$80,750
Const. Work Effort				**$1,025,525**
Escallation to Midpoint of Construction @ 5% for 5 yrs				$283,353
Total Base Bid Estimated Cost				**$1,308,878**