ENGINEERING APPENDIX

Mississippi

Coastal

Improvement

Projects

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History of Tropical Cyclones on the Mississippi Coast

Introduction

Tropical cyclones are commonly recurring hazards in coastal Mississippi. Climatologically, the central Gulf coast region has one of the highest rates of occurrence in the United States. The Atlantic tropical cyclone database since 1886 indicates significant tropical storm impacts on the region occurring about every 2-3 years, and at least category 1 hurricane impact about every 8-9 years. However, the record since 1886 has severe limitations in assessing a longer temporal perspective on tropical cyclone activity. Historical records enable reconstruction of tropical cyclones that extend back to the eighteenth century. Meteorological records afford a detailed and continuous reconstruction at yearly resolution back to the mid 1800`s.

Historical Data

All available historical data has been utilized in the present study. First, tropical cyclone occurrences were compiled for each year from the HURDAT database from 1851-2005, counting each storm believed to be of hurricane intensity when it was centered within 75 miles of the Mississippi Coast. Similarly, a compilation of early nineteenth century hurricanes (1800-1850) was utilized (Bossak, 2003). This database relied primarily upon the landmark work of Ludlum (1963). All storms prior to 1800 were compiled from Ludlum (1963). For the period 1800-1870, only minor adjustments were made from a detailed examination of early instrumental records, diaries, and newspapers.

Results

A chronological listing of all known Hurricanes to affect Mississippi from 1711 to 2005 is given in Table 1. The resultant time series is shown in Figure 1. For the period of record, 66 tropical cyclones were identified as being of hurricane intensity Examination of the series reveals an obvious discontinuity in storm frequency circa 1840. This is simply a statistical artifact, as many tropical cyclone events prior to this time must have been unreported due to sparse population and lack of communication. Not until daily Meteorological observations were initiated by U.S. Army Post Surgeons at New Orleans in 1838, and near Mobile in 1840, can we be certain that all hurricanes were accounted for.

Temporal analysis of the tropical cyclone record, smoothed by 9-year running frequencies, indicate decadal variability in the historical past exceeding that of modern times. In particular, the 1850-1880 period was extraordinarily active. It was followed by another active period from 1910-1930. Much of the twentieth century…1930-1990…was conspicuous for relative inactivity. Indeed, it was this era that is the most anomalous period in the entire record.

The most active hurricane years were 1860 and 2005, with three hurricanes each. Since 1800, major Hurricane impact (category 3 or greater) is clearly evident in 1812, 1819, 1852, 1855, 1860, 1893, 1906, 1909, 1915, 1916, 1947, 1969, 1985, and 2005.

The small but extremely intense Bay St. Louis Hurricane of July 27-28, 1819…and the nearly identical Category 5 Hurricane Camille of August 17-18, 1969 were the most intense storms of record. Hurricanes Camille (1969) and Katrina (2005) produced the largest known tidal surge.

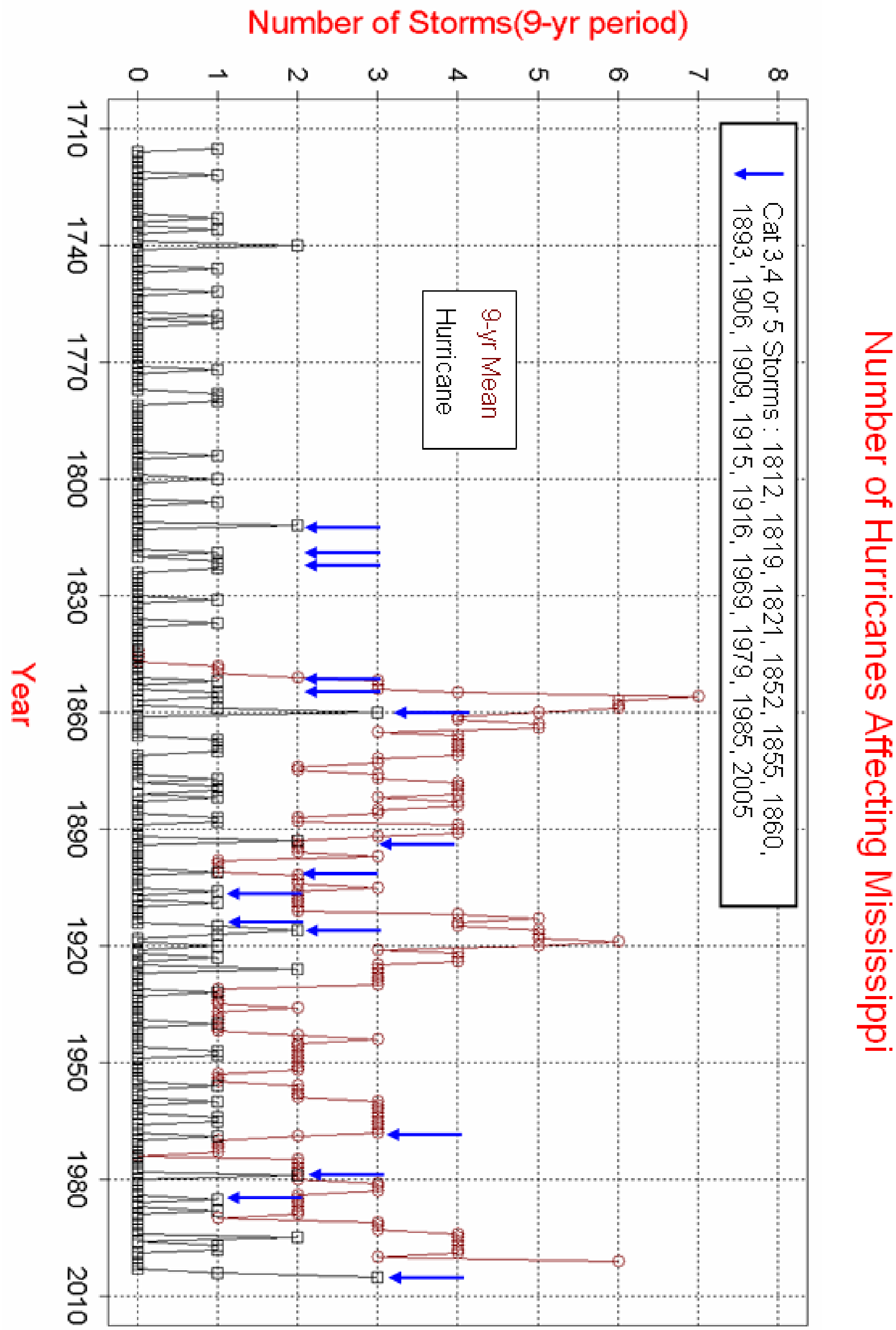
Number of Hurricanes Affecting Mississippi
Number of Storms(9-yr period)
Year
Cat 3,4 or 5 Storms : 1812, 1819, 1821, 1852, 1855, 1860, 1893, 1906, 1909, 1915, 1916, 1969, 1979, 1985, 2005
9-yr Mean
Hurricane
0
1
2
3
4
5
6
7
8
1710
1740
1770
1800
1830
1860
1890
1920
1950
1980
2010

Figure 1.

Table 1.
Hurricanes Affecting Mississippi Coast (1715-2005)

Year	Landfall	Estimated Storm Category at Landfall
1715 n.d.	Dauphin Island	(1)/Unknown
1722 Sept. 22-23	New Orleans	(1)
1733	Mobile	(1)
1736	Pensacola	(1)
1740 Sept. 22	Mobile	(1) The Twin Mobile Hurricanes of 1740
1740 Sept. 29	Mobile	(1) Second Mobile Hurricane
1746 n.d.	Ala.-Miss.-La.	(1)
1752 Nov. 3	Pensacola	(1)
1758 n.d.	N.W. Florida	(1)
1760 Aug. 12	Pensacola	(1)
1772 Aug. 30-Sept. 3	Fla.-La.	(1)
1778 Oct. 7-10	Fla.-La.	(1)
1779 Aug. 18	New Orleans	(1)
1780 Aug. 24	New Orleans	(1)
1794 Aug. 31?	Louisiana	(1)
1800 Aug	New Orleans	1
1806 Sept. 17	New Orleans	1
1812 June 11-12	Louisiana	1
1812 Aug 19	New Orleans	3
1819 July 27-28	Bay St. Louis	3/4
1821 Sept. 15-17	Bay St. Louis	3
1822 July 7-8	Biloxi	1
1823 Sept. 12-14	La.-Ala.	1
1831 Aug. 17-18	New Orleans	3/4
1837 Oct. 3-7	La.-Fla.	2
1852 Aug. 25	Pascagoula	3
1855 Sept. 15-16	Bay St. Louis	3
1856 Aug. 10-11	New Orleans	4
1859 Sept. 15	Mobile	1
1860 Aug. 11	Biloxi	3
1860 Sept. 14-15	Biloxi	2
1860 Oct. 2-3	Houma, La.	2
1867 Oct. 4-5	La.-Fla.	2
1868 Oct. 3-4	La.-Fla.	1
1869 Sept. 5	New Orleans	1
1870 July 30	Mobile	1
1877 Sept. 21	La.-Fla	1
1879 Aug. 31-Sept.1	New Orleans	2/3
1880 Aug. 26-30	Pensacola	1
1882 Sept. 10	Pensacola	3
1887 Oct. 19	Port Eads, La.	1
1888 Aug. 19-20	New Orleans	1/2
1893 Sept. 7-8	Grand Isle, La	1/2
1893 Oct. 2	Pascagoula	3

Table 1.
Hurricanes Affecting Mississippi Coast (1715-2005) (cont.)

Year	Landfall	Estimated Storm Category at Landfall
1901 Aug. 15	Gulfport	1
1906 Sept. 27	Pascagoula	3
1909 Sept. 20	New Orleans	3
1915 Sept. 29	New Orleans	2/3
1916 July 5	Pascagoula	3
1916 Oct. 18	Perdido Key	3
1917 Sept. 28	Pensacola	2
1920 Sept. 21	Houma, La.	2
1923 Oct. 15	Houma, La	1/2
1926 Aug, 26	Houma, La	2
1926 Sept. 21	Perdido Key	1/2
1932 Sept. 1	Mobile	1
1940 Aug.6	La.-Tx.	1
1947 Sept. 19	New Orleans	2
1948 Sept. 4	New Orleans	1
1956 Sept. 24	Port Eads/ Ft. Walton	1
1960 Sept. 15	Gulfport	1
1964 Oct. 3	Franklin, La	1
1965 Sept. 10	New Orleans	3
1969 Aug. 17	Bay St. Louis	5
1979 July 5	Grand Isle	1
1979 Sept. 12	Mobile/Pascagoula	3
1985 Sept. 2	Biloxi	3
1988 Sept. 9	New Orleans	1
1995 Aug. 3	Pensacola	3
1995 Oct. 4	Navaree, Fla.	3
1997 July 19	Mobile	1
1998 Sept. 28	Biloxi	2
2004 Sept. 16	Pensacola	3
2005 July 6	Grand Isle, La.	1
2005 July 10	Navarre, Fla.	2
2005 Aug. 29	Bay St. Louis	3

Conclusion

Tropical cyclones affecting coastal Mississippi appear to have been somewhat more frequent in the historical past than during the present human lifetime. Only during the last decade have we seen a significant upswing in the frequency of occurrence. Six major hurricanes struck the Mississippi coast during the 1800`s…with seven major storms in the 1900`s. Only hurricane Katrina of 2005 has made landfall as a major hurricane during the 21st Century. Thus, there is no evidence that land falling hurricanes in Mississippi are becoming more intense.

References

Bossak, B. H., 2003: Early 19th Century U.S. Hurricanes: A GIS Tool and Climate Analysis, Florida State University Department of Meteorology.

Ludlum, D.M., 1963: Early American Hurricanes, American Meteorological Society, Boston, MA.

MOBILE DISTRICT TIDE GAGE FREQUENCY ANALYSIS

Background

The US Army Corps of Engineers Mobile District (CESAM) maintains a network of tide gages along the Gulf Coast from Gulfport, MS eastward to Carrabelle, FL. Hurricane Katrina made landfall at the Louisiana-Mississippi State line August 29, 2005 and generated record storm surge along the Mississippi and Alabama coast. Preliminary high water mark (HWM) data values from FEMA indicate surge ranging from 28 ft at Bay St. Louis to 11.5 ft at Mobile, AL. The following are Mobile District tide gages along the Mississippi and Alabama coast with long term records; Gulfport, MS (42 years), Biloxi, MS (123 years) Pascagoula, MS (65 years), Dauphin Island (42 years) and State Docks (65years). A Graphical Frequency analysis was performed on the observed historical annual peak water (tide) levels to estimate the still water storm surge return interval. This analysis is limited to the historical water levels only, no meteorologically information or probability statistic such as storm frequency included in the analysis.

"Water level during a storm is the sum effect of wind speed, direction, and atmospheric pressure, in addition to the timing and strength of the tide when the storm reaches its peak strength. It is important to keep in mind that the timing of the maximum observed water level is dependent upon the interaction of the tide and the storm. However, the maximum storm surge is dependent upon the timing of the observed water level with the predicted water level, and does not necessarily coincide with the occurrence of the maximum observed water level."1 This analysis uses the maximum water level as the representative storm surge. Water levels recorded at the gage sites are collected in a stilling well to eliminate the impacts from wave height and wave run-up. In cases where the tide gage was destroyed or malfunctioned, the maximum water level represented by a high water mark measured in a nearby enclosed structured.

Each tide gage is installed to support our navigation coastal dredging program. Consequently the gages are installed near the navigation projects such as harbors, ports, federal docks, and shipping channels. The gages are operated and maintained by the Mobile District Engineering Division, Hydraulics & Hydrology Branch. Mobile District archives the data for legal reasons and makes it available to the public upon request. Monthly and annual reports of the tide levels are generated, archived and made available upon request. The gages were not installed to provide data for modeling requiring accuracy of less than 0.1 foot. We see no problem using the data to develop correlations between the gage sites and making trend estimates. There is limited quality control of the tide data.

Figure 1. Mobile District Tide Gage Network

When there is a great possibility that a hurricane is going to strike the Gulf Coast CESAM personnel are dispatched to remove recorded data from coastal gages and insure that the gages are working properly to record the hurricane surge. All equipment is removed from gage sites in areas of forecasted direct storm path 1-3 days before landfall. Therefore, removing the proper gage is a function of the forecast accuracy. Two gages were removed in Mississippi and one in Alabama on 28 August 2005, one day before the projected landfall. Water levels along the Gulf Coast for the time period during the storm are available at 16 gages and partial record from 5 gages. A total of 9 gages were destroyed and 2 gages were damaged by the hurricane.

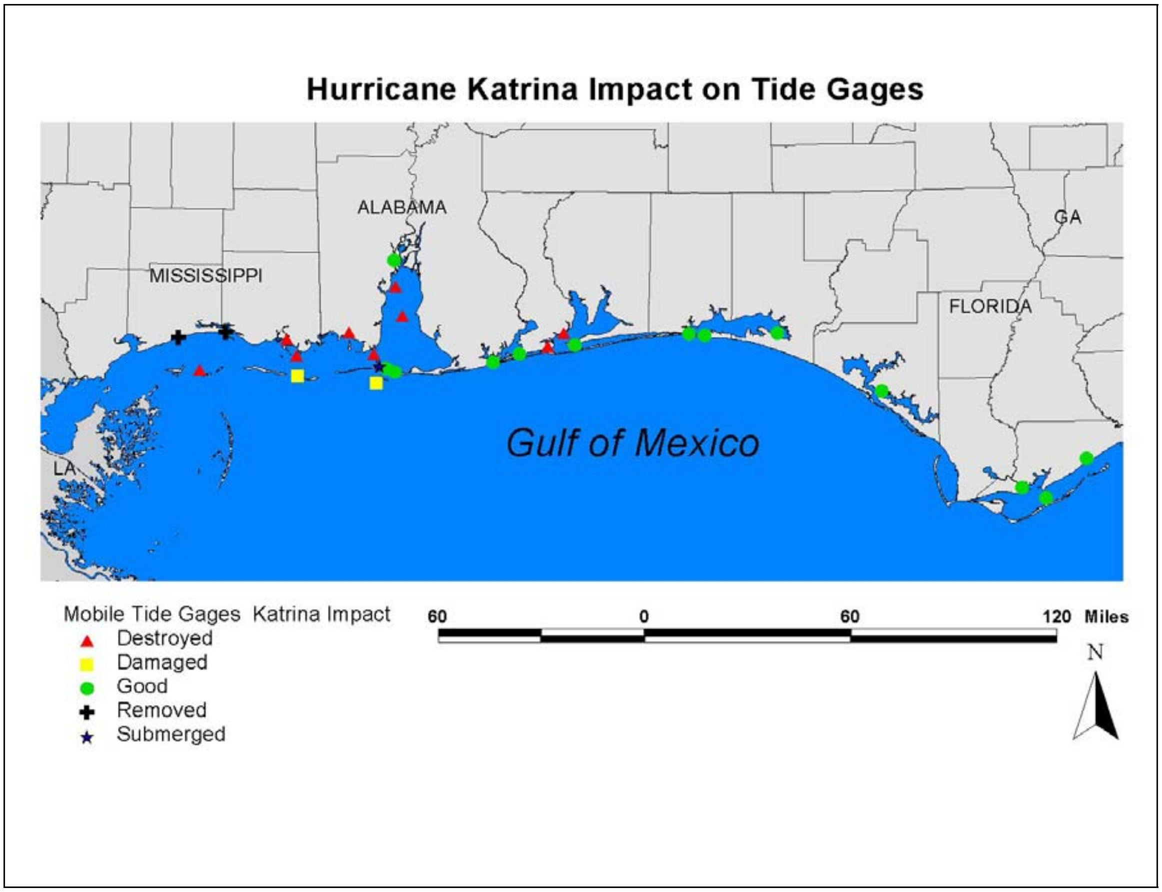

Figure 2. Impact on Tide Gages

There are 7 active CESAM tide gages along the Mississippi Coast gages. The map below depicts the location of those gages.

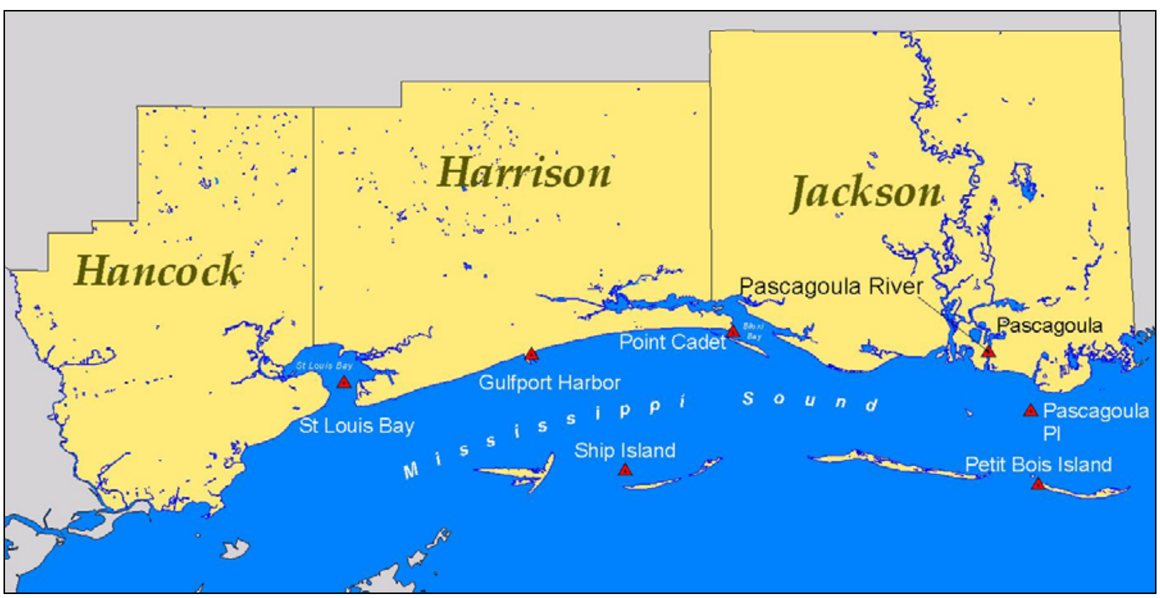

Figure 3. CESAM Mississippi Coast Tide Gages

Only two gages Gulfport and Pascagoula are currently operational. The remaining gages were destroyed or heavy damaged by hurricane Katrina. The Mississippi gages selected for graphical analysis are Gulfport, Biloxi and Pascagoula.

History of Tide Gages
Gulfport, MS (02481341)
Location: The gage is located at latitude 30 21' 50", longitude 89 05'21", Gulfport South MS Quad, Harrison County, on the east wooden dock on the north east end of boat house Coast Guard pier near US Coast Guard facility, Gulfport Harbor in Gulfport, MS. The station number is 02481341.

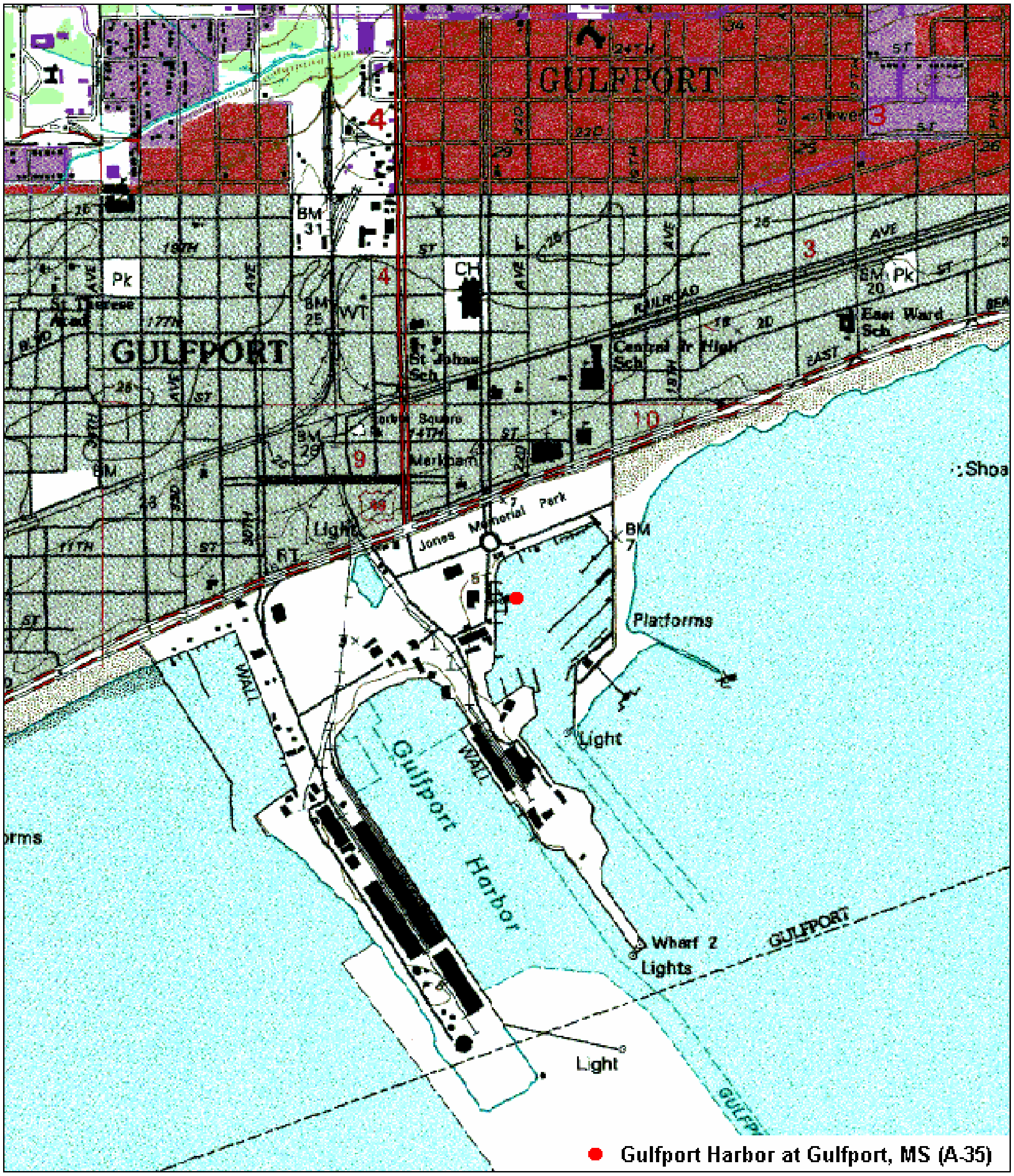

Figure 4. Gulfport , MS Tide Gage

Period of Record: CESAM established the continuous recording gage on 8 May 1963. Continuous record is available since that date except for periods of equipment malfunction or destruction by storm. The gage has been relocated on 2 occasions within close proximity of the initial installment. The gage was removed before hurricane Katrina made landfall and re-installed on the Coast Guard property in December 2005. A permanent location will be determined after the destroyed Coast Guard building is rebuilt.

Storm Surges: Above the average high tide levels caused by tropical disturbances has been documented for 37 tropical storms. Historic accounts of Gulfport, MS storm surge values prior to establishing the gage are available for the storms listed below.

Table 1.
Gulfport, MS Gage Historic Storm Surge

Storm	*Elevation in Feet above Mean Sea Level
September 29, 1915	9.0
September 21, 1926	6.0
September 19, 1947	14
September 4, 1948	6.0
August 26, 1955	6.0
Flossy (September 24, 1956)	4.0
Tropical Storm Ester (September 18, 1957)	6.5
Ethel (September 15, 1960)	5.0

*Elevations from "Report on Hurricane Survey of Mississippi Coast" 1965

The top three storm surge values are listed below

Table 2.
Gulfport, MS Top Three Storm Surge Values

Hurricane	Still Water Storm Surge Feet above NGVD
Katrina (2005)	24.17
Camille (1969)	19.68
Sep 14, 1947	14.00

Biloxi, MS (02480350)
Location: The gage is located at latitude 30° 23′ 22″, longitude 88° 51′ 26″, Harrison County, Ocean Springs MS Quad, located approximately 400 feet south of the southwest end of U.S. Hwy 90 bridge over Biloxi Bay, behind the Marine Education Center on the boat dock. The station number is 02480350.

Figure 5. Point Cadet, MS Tide Gage

Period of Record: New Orleans District established the gage 1881. The gage was transferred to Mobile District September 30, 1983. Continuous record is available since the original installation except for 1886-1894 and periods of equipment malfunction or destruction by storm. The gage has been relocated on 3 occasions. From 1881 to 1998 the gage was located on US Hwy 90 Bridge over Biloxi Bay near center span. On 25 June 1998 a second gage located on land 1.3 mile southwest at Point Cadet. The 2 gages existed concurrently for about 6 months until the gage located on US Hwy 90 was removed. The Point Cadet gage was removed before hurricane Katrina made landfall and will be re-installed after boat dock reconstruction. Plans are underway to raise the gage above major hurricane storm surge.

Storm Surge: Above average high tide levels caused by tropical disturbances has been documented for 65 tropical storms. The top three surge values are listed below

Table 3.
Point Cadet, MS Top Three Storm Surge Values

Hurricane	Still Water Storm Surge Feet above NGVD
Katrina (2005)	23.8
Camille (1969)	15.6
Sep 14, 1947	10.8

Pascagoula, MS (02480301)
Location: The gage is located at latitude 30° 23′ 01″, longitude 88° 33′ 48″, Jackson County, Pascagoula South MS Quad, located on the west side of NOAA's facility station on the Pascagoula River, 0.85 miles above the mouth of the Pascagoula River. The station number is 02480301.

Figure 6. Pascagoula, MS Tide Gage

Period of Record: CESAM established the continuous recording gage on 18 July 1940 at Ingalls shipyard. Continuous record is available since that date except for periods of equipment malfunction or destruction by storm. The gage has been relocated on 6 occasions within close proximity of the initial installment ranging from the mouth of the Pascagoula River to 0.85 above the mouth. The gage was submerged by Katrina's storm surge. Data from the gage was successfully transmitted via satellite up until that time. A highwater mark at the gage site documents the still water storm surge. CESAM reinstalled the gage at the same location approximately 3 months after the storm. Plans are underway to raise the gage above major hurricane storm surge.

Storm Surge: Above average high tide levels caused by tropical disturbances has been documented for 51 tropical storms. The top three surge values are listed below

Table 4.
Pascagoula, MS Top Three Storm Surge Values

Hurricane	Still Water Storm Surge Feet above NGVD
Katrina (2005)	16.6
Camille (1969)	11.2
Georges (1998)	8.4

No reliable historic accounts of Pascagoula, MS storm surge values prior to establishing the gage we sited from our available reports.

Hurricane Katrina's record storm surge extended beyond the Mississippi coast line. The State Docks Mobile, AL tide gage site has official records back to 1772 and is located about 100 miles east of Hurricane Katrina's storm landfall at Louisiana-Mississippi state line. The recorded surge of 11.45 feet above NGVD is the highest recorded. Below is State Dock's hurricane Katrina storm stage hydrograph

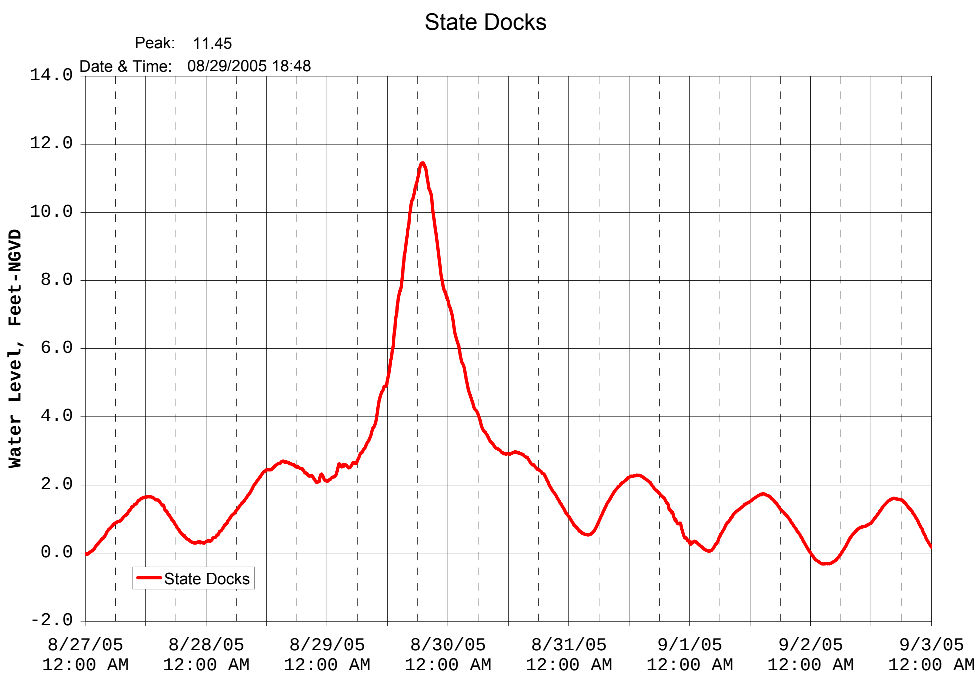

Figure 7. Mobile, AL State Docks Katrina Hydrograph

Methodology

EM 1110-2-1415 recommends using graphical analysis for stage (elevation) frequency computation. The Corps of Engineers computer program Flood Frequency Analysis (FFA) was selected to compute the graphical plotting positions. Historical data was incorporated into the graphical analysis using the procedures outlined in Bulletin 17B. The median plotting position formula was selected to derive the re-occurrence intervals because it corrects for the bias caused by small sample sizes.

Care was taken to select a uniform data set for the frequency analysis. Each event represents the peak water level for each January-December calendar year. There are a few years with less than 12 month of recorded data, partial record. In most cases this is due to a gage malfunction or damage from a storm event. No significant changes in the hydrologic conditions have taken place at the 3 gaging sites during the period of record. However, the data set is not complete uniformly because of local subsidence and rise in sea level. For this analysis no attempts have been made to adjust the data to account for these factors. Future analysis by this office will research the necessary adjustments. Each of the 3 gages has been relocated within the period of record. No adjustments were required because of the close proximity of relocations. In cases where the gage was destroyed by a severe storm, a still water high water at or near the gage used to represent the peak elevation for that storm event.

Historic data is information before the collection of systemic record. The account is often described in newspaper article, personal accounts from a witness or an investigation by some agency or entity. Historic data is very useful for locations with relative short period of record and use to extend the period of systemic record. The use of historic record can improve the frequency estimate.

The population includes annual peaks that result from storm surge and normal tidal fluctuations. There are years were multiple storms caused storm surge above normal high tide. Only the maximum recorded for each year used in the analysis. Partial duration frequency analysis was eliminated because of limited available daily data for the full period of record.

Gulfport has 43 year, 1963-2005, on continuous systematic record. Well document historic values for the years 1915, 1926, 1947, 1948, 1955-1957, and 1960 are included in the analysis.

Biloxi has 111 years, 1882-1885 and 1896-2005, of continuous systematic record.

Pascagoula has 66 years, 1940-2005, of continuous systematic record.

A best fit curve was drawn through the median plotting positions for each gage site. The reoccurrence intervals selected for the 2, 5, 10, 20, 50 and 100 years are tabulated in the table below.

Table 5.
Results from Graphical Frequency Analysis

Exceedence Frequency	Exceedence Interval in Years	Pascagoula	Biloxi	Gulfport
50	2	3.2	3.6	4.2
20	5	4.0	4.4	6.0
10	10	6.0	5.6	6.8
5	20	7.8	7.5	9.3
2	50	12.5	12.5	14.0
1	100	17.0	19.0	23.0
0.5	200			
	Period of Record	1916-2005	1882-2005	1941-2005

Presentation of Data

Table 6.
Mississippi Coast Recorded Storm Surge at Mobile District Tide Gages

Storm	Date	Gulfport (1963)			Pascagoula (1940)			Biloxi (1882)		
		G.H.	**NGVD**		**G.H.**	**NGVD**		**G.H.**	**NGVD**	
Sep 1882	9/10/1882								2.29	
27Sep1906	1906-Sep-27								5.92	
20Sep1909	1909-Sep-20							10.43	4.35	
12Aug1911	1911-Aug-12								4.36	
14Sep1912	1912-Sep-14								3.38	
29Sep1915	1915-Sep-29		9.00	1					8.92	
05Jul1916	1916-Jul-05								4.07	
28Sep1917	1917-Sep-28							8.61	2.53	
21Sep1920	1920-Sep-21								5.44	
15Oct1923	1923-Oct-15							11.96	5.88	7
21Sep1926	1926-Sep-21		6.00	1					3.82	
Sep 1932	1932-Sep							9.16	3.08	
Oct 1932	1932-Oct							9.33	3.25	
July 1933	1933-Jul							9.16	3.08	
Sep 1933	1933-Sep							9.74	3.66	
Jun 1934	1934-Jun							8.98	2.90	
T.S. Jun 1939	1939-Jun							9.05	2.97	
26Sep1939	1939-Sep-29							9.5	3.42	
	1940-Aug-06					3.63		10.4	4.32	
12Sep1941	1941-Sep-12					3.30		9.52	3.44	
06Sep1945	1945-Sep-06						5	9.1	3.02	
	1947-Sep-08					2.60				6
19Sep1947	1947-Sep-19		14.00	1		7.40	2,6	16.88	10.80	2,6
04Sep1948	1948-Sep-04		6.00	1		4.00			5.60	
	1949-Sep-04					3.90			4.46	
Baker	1950-Aug-30					3.65			3.53	
Barbara	1954-Jul-29					2.35		9.1	3.02	
Brenda	1955-Aug-01					3.10			3.87	
26Aug1955	1955-Aug-26		6.00	1		2.75			3.54	
	1956-Jun-13					3.40		10.78	4.70	
Flossy	1956-Sep-24		4.00	1		3.10		9.39	3.31	
Audrey	1957-Jun-27					3.28			3.62	
T.S Ester	1957-Sep-18		6.50	1		2.55			4.64	
Ethel	1960-Sep-15		5.00	1		4.50			5.12	
Helda	1964-Oct-04	5.14	4.14			4.05			4.63	
Betsy	1965-Sep-09		10.70	2,7		6.40		14.64	8.56	
Debbie	1965-Sep-29	6.8	3.80			2.84				6
Camille	1969-Aug-17		19.68	2		11.25	2		15.56	2
Felice	1970-Sep-15	3.01	3.01		2.43	2.31		8.94	2.86	
Fern	1971-Sep-05	2.68	2.41		2.37	2.25				
Edith	1971-Sep-16	3.35	3.08		2.08	1.96			3.50	
Carmen	1974-Sep-08	4.95	4.68		3.98	3.86			4.47	
Babe	1977-Sep-06	3.9	3.63				5			5
Bob	1979-Jul-11		6.00			4.55			5.62	
Frederic	1979-Sep-12		3.30			5.78			3.90	
Elena	1985-Sep-02		5.43			5.50			6.03	
Juan	1985-Oct-28		6.50			5.31			5.83	

Table 6.
Mississippi Coast Recorded Storm Surge at Mobile District Tide Gages (continued)

		Gulfport (1963)			Pascagoula (1940)			Biloxi (1882)		
Storm	**Date**	**G.H.**	**NGVD**		**G.H.**	**NGVD**		**G.H.**	**NGVD**	
Bonnie	1986-Jun-23		2.60			2.37			2.70	
Gilbert	1988-Sep-08		4.77			3.02			3.93	
Flourence	1988-Sep-10		4.54			3.03			6.26	
Chantal	1989-Jul-31		3.00			2.23			3.35	
Andrew	1992-Aug-26		3.89			3.10			3.77	
TS Dean	1995-Jul-28		3.57			2.75			3.39	
Erin	1995-Aug-04		2.55			2.76			2.91	
Opal	1995-Oct-04		2.92			2.57				3
Josephine	1996-Oct-05		3.34			2.66			3.34	
Danny	1997-Jul-19		4.12			2.90			3.74	
Earl	1998-Sep-02		3.17			3.08		3.52	2.87	
Georges	1998-Sep-28		7.05			8.36	2		8.05	
T.S. Helen	2000-Nov-24		3.62			3.00			3.35	
T.S. Allison	2001-Jun-11		4.43			3.90				5
T.D. Edward	2002-Sep-06		4.00		4.09	3.37			3.44	
T.S. Hanna	2002-Sep-14	5.14	4.52		4.64	3.92			4.03	
Isidore	2002-Sep-26	8.26	7.64			5.75			6.86	
Lili	2002-Oct-04	3.79	3.17			3.88			4.75	
T.S. Bill	2003-Jul-10	4.6	3.98			3.33			3.99	
Ivan	2004-Sep-16	5.28	4.66			6.72	4		4.23	
T.S. Matthew	2004-Oct-10	4.88	4.26		3.66	2.94		4.32	3.67	
T.S. Cindy	2005-Jul-06	6.16	5.54			5.75			5.84	
Dennis	2005-Jul-10	3.63	3.01			3.25			2.86	
Katrina	2005-Aug-29		24.17	4		16.60	2		23.80	4
Storm Count			45			51			65	

1 Report on Hurricane Survey
2 High Water Mark at Gage Site
3 No Record gage vandelized
4 Gage Removed before landfall, HWM at gage site
5 No Record Gage Malfunctioned
6 No Record gage destroyed
7 Partial Record, gage malfunction

Results

Table 7.
Guflport, MS Annual Peaks

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	24.17	1	1.09	0.77	Katrina (2005)
1969	19.68	2	2.17	1.86	Camile (1969)
1947	14.00	3	3.26	2.95	Sep 19, 1947
1965	10.70	4	4.35	4.05	Betsy (1965)
1915	9.00	5	5.43	5.14	Sep 29, 1915
2002	7.61	6	6.99	6.71	Isidore (2002)
1998	7.05	7	9.03	8.76	Georges (1998)
1957	6.50	8	11.06	10.8	TS Ester (1957)
1985	6.50	9	13.09	12.85	Juan (1985)
1926	6.01	10	15.12	14.89	Sep 21, 1926
1948	6.00	12	19.19	18.98	
1979	6.00	11	17.16	16.94	Bob (1979)
1955	5.99	13	21.22	21.03	
1973	5.33	14	23.25	23.08	
1960	5.00	15	25.28	25.12	Ethel (1960)
1988	4.77	16	27.32	27.17	Gilbert (1988)
1970	4.72	17	29.35	29.21	
1984	4.70	18	31.38	31.26	
1974	4.68	19	33.41	33.3	Carmen (1974)
1986	4.65	20	35.44	35.35	
2004	4.63	21	37.48	37.39	Ivan (2004)
2001	4.43	22	39.51	39.44	TS Allison (2001)
1971	4.23	23	41.54	41.49	
1972	4.23	24	43.57	43.53	
1964	4.14	25	45.6	45.58	Helda (1964)
1997	4.12	26	47.64	47.62	Danny (1997)
1983	4.05	27	49.67	49.67	
1999	4.05	28	51.7	51.71	
1990	4.01	29	53.73	53.76	
1956	4.00	30	55.77	55.8	Flossy (1956)
1991	4.00	31	57.8	57.85	
2003	3.98	32	59.83	59.89	TS Bill
1992	3.89	33	61.86	61.94	Andrew (1992)
1980	3.80	34	63.89	63.99	
1967	3.74	35	65.93	66.03	
1987	3.70	36	67.96	68.08	
1977	3.63	37	69.99	70.12	
2000	3.62	38	72.02	72.17	TS Helen
1976	3.58	39	74.05	74.21	
1995	3.57	40	76.09	76.26	TS Dean (1995)
1993	3.49	41	78.12	78.3	
1994	3.36	42	80.15	80.35	
1996	3.34	43	82.18	82.39	Josephine (1996)

Table 7.
Guflport, MS Annual Peaks (continued)

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1975	3.23	44	84.22	84.44	
1966	3.22	45	86.25	86.49	
1981	3.10	46	88.28	88.53	
1982	3.07	47	90.31	90.58	
1989	3.00	48	92.34	92.62	
1978	2.93	49	94.38	94.67	
1968	2.83	50	96.41	96.71	
1963	2.62	51	98.44	98.76	

Table 8.
Biloxi, MS Annual Peaks

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	23.80	1	0.89	0.63	Katrina (2005)
1969	15.56	2	1.79	1.53	Camile (1969)
1947	10.80	3	2.68	2.42	Sep 19, 1947
1915	8.92	4	3.57	3.32	Sep 29, 1915
1965	8.56	5	4.46	4.22	Betsy (1965)
1998	8.05	6	5.36	5.12	Georges (1998)
2002	6.86	7	6.25	6.01	Isidore (2002)
1988	6.26	8	7.14	6.91	Florence (1988)
1985	6.03	9	8.04	7.81	Elena (1985)
1906	5.92	10	8.93	8.71	Sep 27, 1906
1923	5.88	11	9.82	9.61	Oct 15, 1923
1973	5.72	12	10.71	10.50	
1979	5.62	13	11.61	11.40	Bob (1979)
1948	5.60	14	12.50	12.30	Sep 4, 1948
1920	5.44	15	13.39	13.20	Sep 21, 1920
1960	5.12	16	14.29	14.09	Ethel (1960)
1972	4.99	17	15.18	14.99	
1956	4.70	18	16.07	15.89	Jun 13, 1956
1957	4.64	19	16.96	16.79	TS Ester (1957)
1964	4.63	20	17.86	17.68	Helda (1964)
1919	4.51	21	18.75	18.58	
1974	4.47	22	19.64	19.48	Carmen (1974)
1949	4.46	23	20.54	20.38	Sep 4, 1949
1934	4.44	24	21.43	21.27	
1984	4.43	25	22.32	22.17	
1983	4.40	26	23.21	23.07	
1911	4.36	27	24.11	23.97	Aug 21, 1911
1909	4.35	28	25.00	24.87	Sep 9, 1909
1940	4.32	29	25.89	25.76	Aug 6, 1940
1992	4.32	30	26.79	26.66	
1999	4.25	31	27.68	27.56	

Table 8.
Biloxi, MS Annual Peaks (continued)

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2004	4.23	32	28.57	28.46	Ivan (2004)
1961	4.21	33	29.46	29.35	
1945	4.13	34	30.36	30.25	
1916	4.07	35	31.25	31.15	Jul 05, 1916
2003	3.99	36	32.14	32.05	TS Bill (2003)
1987	3.97	37	33.04	32.94	
1933	3.92	38	33.93	33.84	
1971	3.90	39	34.82	34.74	
1950	3.87	40	35.71	35.64	Baker (1950)
1966	3.83	41	36.61	36.54	
1905	3.82	42	37.50	37.43	
1926	3.82	43	38.39	38.33	Sep 21, 1926
1993	3.80	44	39.29	39.23	
1997	3.74	45	40.18	40.13	Danny (1997)
1932	3.67	46	41.07	41.02	
1990	3.67	47	41.96	41.92	
1991	3.63	48	42.86	42.82	
1970	3.59	49	43.75	43.72	
1955	3.54	50	44.64	44.61	TS 26Aug1955
1996	3.53	51	45.54	45.51	
1927	3.52	52	46.43	46.41	
1952	3.48	53	47.32	47.31	
1941	3.45	54	48.21	48.20	
1935	3.43	55	49.11	49.10	
2001	3.43	56	50.00	50.00	
1939	3.42	57	50.89	50.90	Sep 26, 1939
1928	3.39	58	51.79	51.80	
1995	3.39	59	52.68	52.69	TS Dean (1995)
1912	3.38	61	54.46	54.49	Sep 14, 1912
1967	3.38	60	53.57	53.59	
1918	3.37	62	55.36	55.39	
1989	3.35	63	56.25	56.28	
2000	3.35	64	57.14	57.18	TS Helen (2000)
1953	3.34	65	58.04	58.08	Florence (1953)
1986	3.34	66	58.93	58.98	
1914	3.32	67	59.82	59.87	
1994	3.31	68	60.71	60.77	
1898	3.29	70	62.50	62.57	
1900	3.29	71	63.39	63.46	
1931	3.29	69	61.61	61.67	
1946	3.27	72	64.29	64.36	
1980	3.25	73	65.18	65.26	
1951	3.24	74	66.07	66.16	
1938	3.20	75	66.96	67.06	
1954	3.15	76	67.86	67.95	

Table 8.
Biloxi, MS Annual Peaks (continued)

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1897	3.10	77	68.75	68.85	
1908	3.04	78	69.64	69.75	
1930	3.03	79	70.54	70.65	
1944	3.02	80	71.43	71.54	Sep 10, 1944
1929	2.94	81	72.32	72.44	
1937	2.94	82	73.21	73.34	
1942	2.94	83	74.11	74.24	
1943	2.92	84	75.00	75.13	
1982	2.92	85	75.89	76.03	
1921	2.89	88	78.57	78.73	
1958	2.89	86	76.79	76.93	
1975	2.89	87	77.68	77.83	
1922	2.83	89	79.46	79.62	
1959	2.82	90	80.36	80.52	TS Irene (1959)
1936	2.74	91	81.25	81.42	
1963	2.73	92	82.14	82.32	
1976	2.72	93	83.04	83.21	
1981	2.70	94	83.93	84.11	
1924	2.66	95	84.82	85.01	
1907	2.64	96	85.71	85.91	
1913	2.62	97	86.61	86.80	
1904	2.57	98	87.50	87.70	
1896	2.53	99	88.39	88.60	
1917	2.53	100	89.29	89.50	Sep 28, 1917
1903	2.46	101	90.18	90.39	
1968	2.41	102	91.07	91.29	
1910	2.37	103	91.96	92.19	
1899	2.35	104	92.86	93.09	
1882	2.29	105	93.75	93.99	Sep 10, 1882
1884	2.27	106	94.64	94.88	
1925	2.22	107	95.54	95.78	
1962	2.21	108	96.43	96.68	
1902	2.17	109	97.32	97.58	
1885	1.94	110	98.21	98.47	
1901	1.94	111	99.11	99.37	

Table 9.
Pascagoula, MS Annual Peaks

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	16.60	1	1.49	1.05	Katrina (2005)
1969	11.24	2	2.99	2.56	Camile (1969)
1998	8.36	3	4.48	4.07	Georges (1998)
1947	7.68	4	5.97	5.57	Sep 19, 1947
2004	6.72	5	7.46	7.08	Ivan (2004)
1965	6.40	6	8.96	8.58	Betsy (1965)
1979	5.78	7	10.45	10.09	Frederic (1979)
2002	5.75	8	11.94	11.60	Isidore (2002)
1985	5.50	9	13.43	13.10	Elena (1985)
1972	5.26	10	14.93	14.61	
1960	4.50	11	16.42	16.11	Ethel (1960)
1964	4.05	12	17.91	17.62	Helda (1964)
1948	4.00	13	19.40	19.13	Sep 4, 1948
1949	3.90	14	20.90	20.63	
2001	3.90	15	22.39	22.14	TS Allison (2001)
1974	3.86	16	23.88	23.64	Carmen (1974)
1970	3.81	17	25.37	25.15	
1961	3.80	18	26.87	26.66	
1984	3.71	19	28.36	28.16	
1983	3.68	20	29.85	29.67	
1950	3.65	21	31.34	31.17	Baker (1950)
1940	3.63	22	32.84	32.68	Aug 6, 1940
1980	3.53	23	34.33	34.19	
1987	3.53	24	35.82	35.69	
1993	3.45	25	37.31	37.20	
1956	3.40	26	38.81	38.70	
1945	3.37	27	40.30	40.21	
1971	3.35	28	41.79	41.72	
1967	3.33	29	43.28	43.22	
2003	3.33	30	44.78	44.73	TS Bill (2003)
1941	3.30	31	46.27	46.23	Sep 12, 1941
1957	3.28	32	47.76	47.74	Audrey (1957)
1992	3.28	33	49.25	49.25	Andrew(1992)
1996	3.28	34	50.75	50.75	
1986	3.24	35	52.24	52.26	
1952	3.15	36	53.73	53.77	
1955	3.10	37	55.22	55.27	Brenda (1955)
1953	3.05	38	56.72	56.78	
1988	3.03	39	58.21	58.28	Flourence (1988)
1991	3.03	40	59.70	59.79	
2000	3.00	41	61.19	61.30	TS Helen(2000)
1978	2.92	42	62.69	62.80	
1990	2.88	43	64.18	64.31	
1989	2.87	44	65.67	65.81	
1973	2.86	46	68.66	68.83	

Table 9.
Pascagoula, MS Annual Peaks (continued)

Year	Gage Height	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1951	2.85	47	70.15	70.33	
1966	2.84	48	71.64	71.84	
1994	2.84	49	73.13	73.34	
1975	2.81	50	74.63	74.85	
1958	2.80	51	76.12	76.36	
1959	2.80	52	77.61	77.86	
1963	2.76	53	79.10	79.37	
1982	2.75	54	80.60	80.87	
1995	2.75	55	82.09	82.38	TS Dean (1995)
1946	2.68	56	83.58	83.89	
1999	2.68	57	85.07	85.39	
1954	2.65	58	86.57	86.90	
1976	2.57	59	88.06	88.40	
1981	2.46	60	89.55	89.91	
1944	2.38	61	91.04	91.42	
1977	2.38	62	92.54	92.92	
1942	2.35	63	94.03	94.43	
1943	2.35	64	95.52	95.93	
1968	2.19	65	97.01	97.44	
1962	2.13	66	98.51	98.95	

Graphs

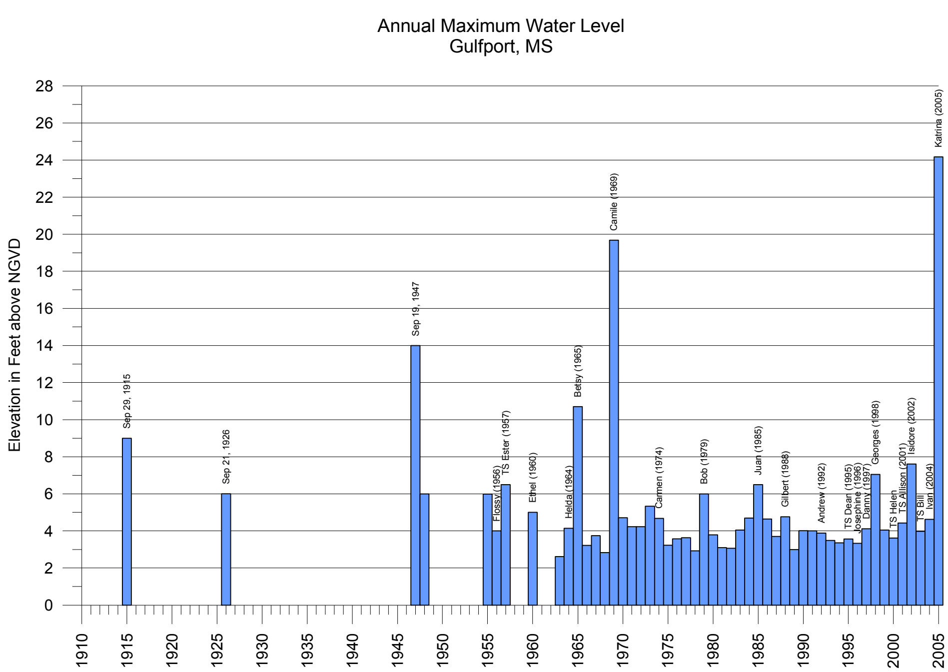

Figure 8. Gulfport, MS Annual Maximum Water Level

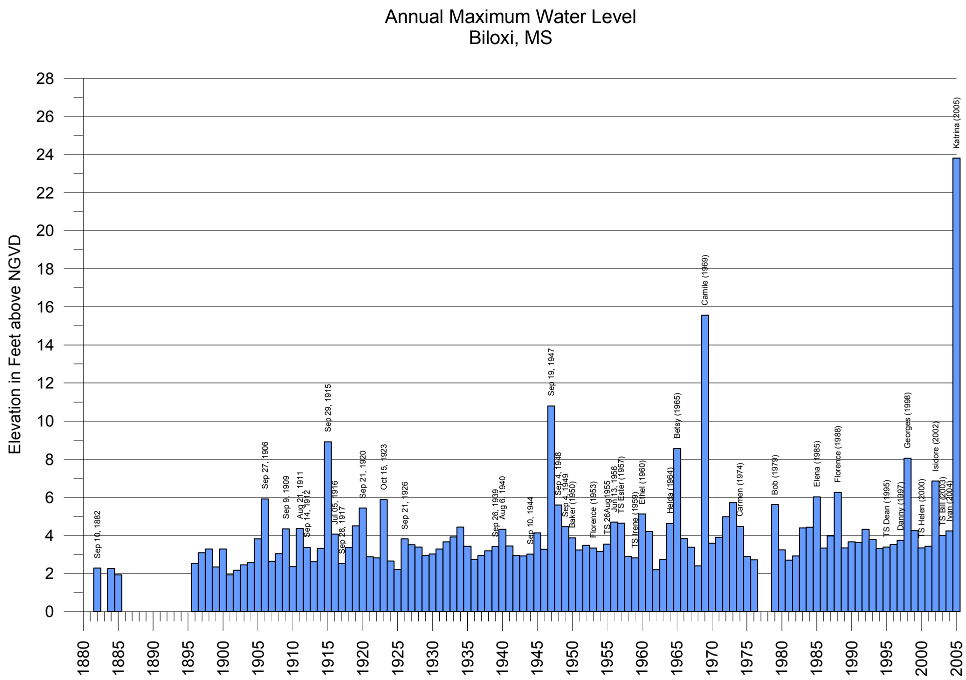

Figure 9. Biloxi, MS Annual Maximum Water Level

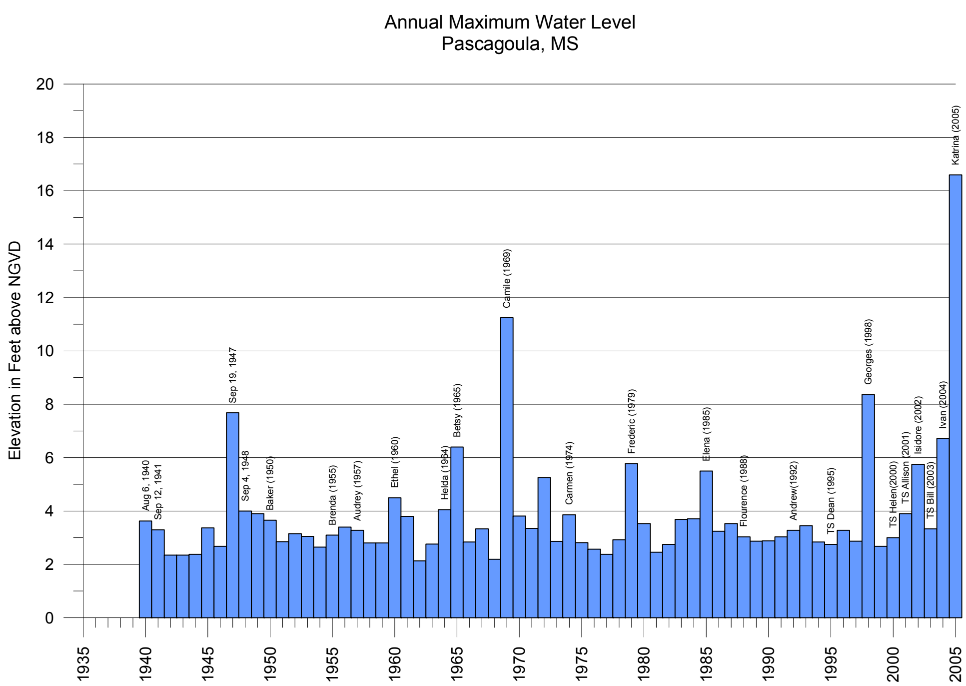

Figure 10. Pascagoula, MS Annual Maximum Water Level

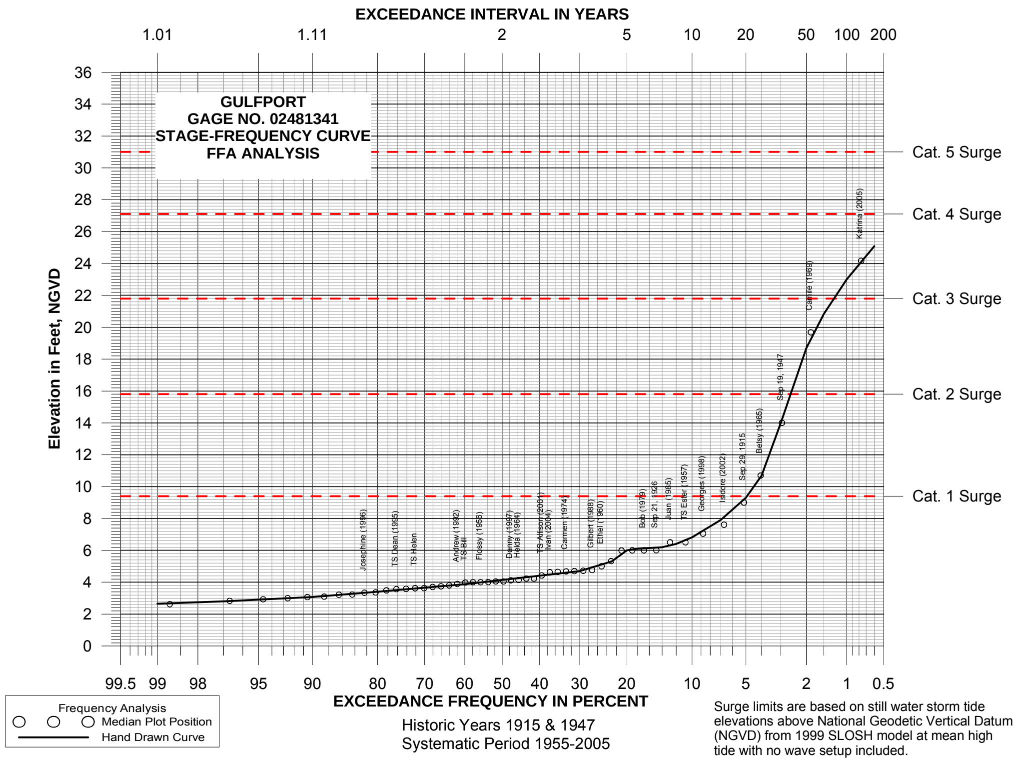

Figure 11. Gulfport, MS Frequency Curve

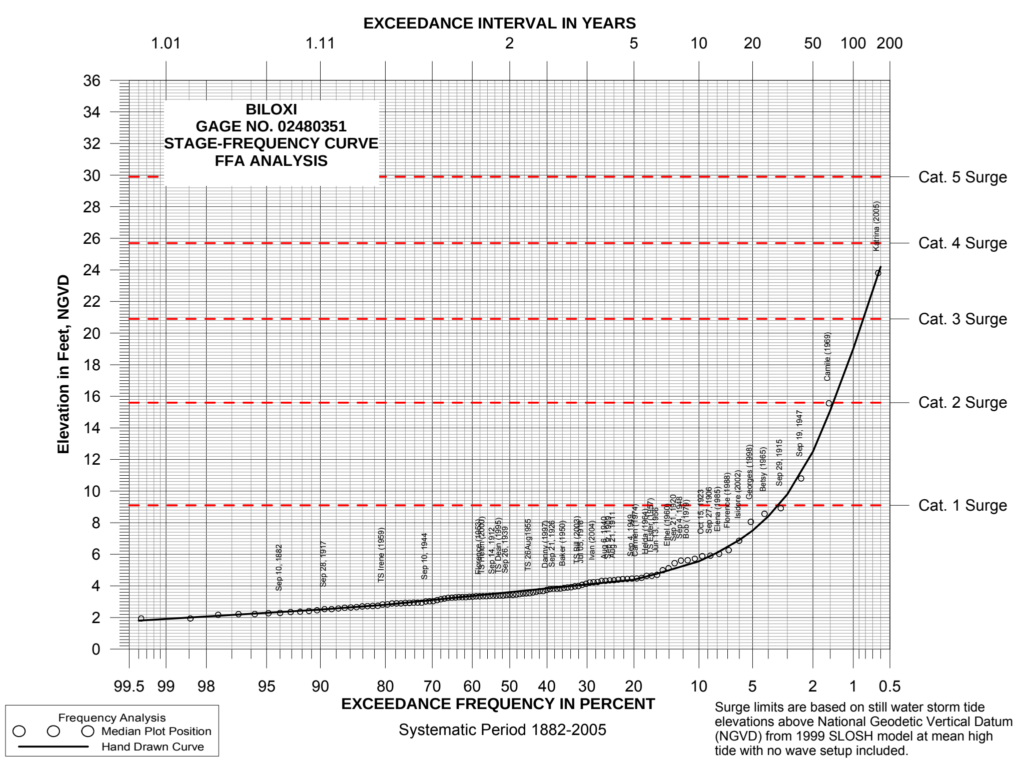

Figure 12. Biloxi, MS Frequency Curver

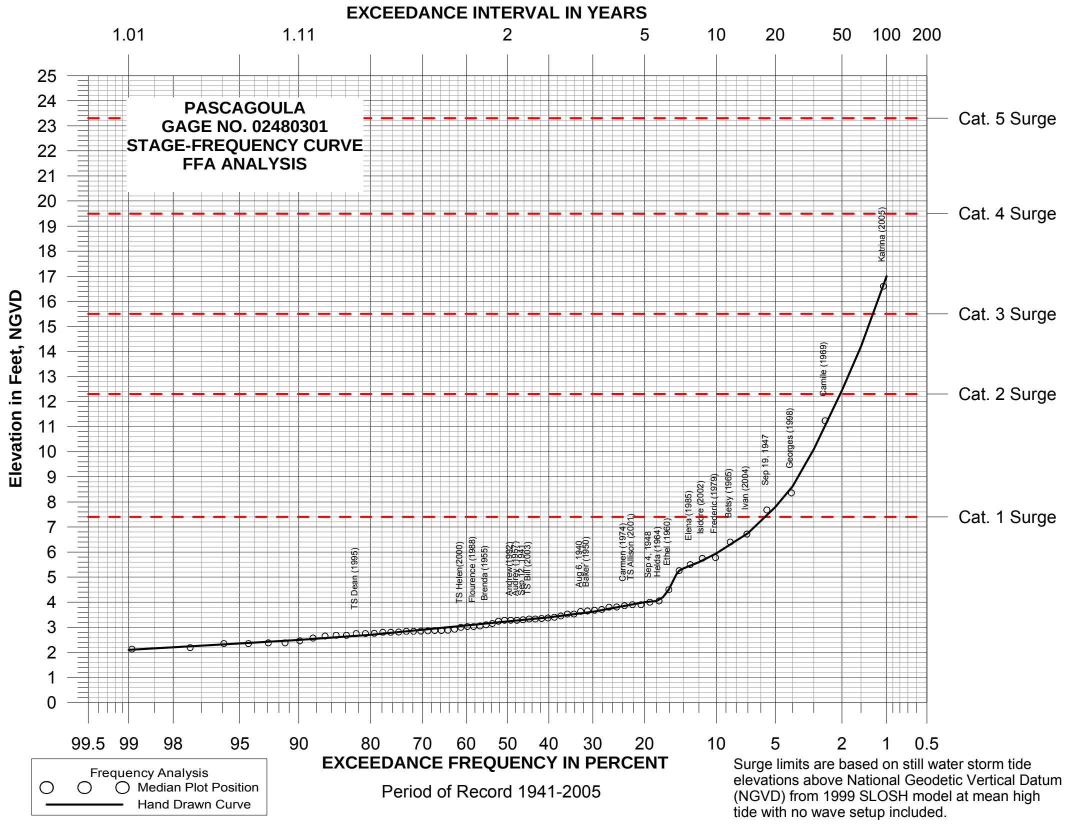

Figure 13. Pascagoula, MS Frequency Curve

FFA Results

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************************************ ************************************
 * FFA * * *
 * FLOOD FREQUENCY ANALYSIS * * U.S. ARMY CORPS OF ENGINEERS *
 * PROGRAM DATE: MAY 1992 * * THE HYDROLOGIC ENGINEERING CENTER *
 * VERSION: 3.0 * * 609 SECOND STREET *
 * RUN DATE AND TIME: * * DAVIS, CALIFORNIA 95616 *
 * 06 APR 06 20:26:07 * * (916) 756-1104 *
 * * * *
 ************************************ ************************************



 INPUT FILE NAME: TIDEANUL.DAT
 OUTPUT FILE NAME: TIDEANUL.OUT
 DSS FILE NAME: TIDEANUL.DSS



 -----DSS---ZOPEN: Existing File Opened, File: TIDEANUL.DSS
 Unit: 71; DSS Version: 6-GX

 **TITLE RECORD(S)**
 TT GULFPORT, MS TIDE ELEVATION FREQUENCY ANALYSIS
 TT GRAPHICAL ANALYSIS OF MAXIMUM TIDE ELEVATION
 TT GRAPHICAL ANALYSIS USING MEDIAN PLOTTING POSITIONS
 TT HISTORICAL OBSERVATIONS INCLUDED
 TT 43 YEARS 1963-2005, HISTORICAL YEARS 1915, 1926, 1947, 1948, 1955-1957, 1960

 **JOB RECORD(S)**
 IPPC ISKFX IPROUT IFMT IWYR IUNIT ISMRY IPNCH IREG
 J1 2 0 0 1 0 3 1 0 0

 **SPECIFIED VARIABLE AND UNITS**
 FU TIDE FEET

 **STATION IDENTIFICATION**
 ID 02481341 GULFPORT, MS 1

 **SPECIAL STATION INFORMATION**
 IYRA IYRL HITHRS LOTHRS LOGT NDEC NSIG
 SI 0 0 0. 0. 0 2 0

 **HP PLOT **
 HP PLOT FILE IHPCV KLIMIT IPER BAREA
 HP GPORT.PCL 0 0 0

 SELECTED CURVES ON HPPLOT
 EXPECTED PROBABILITY CURVE
 CONFIDENCE LIMITS

 HP GULFPORT, MISSISSIPPI

 **HISTORIC EVENTS**
 QH 0 0 1915 9.

 **SYSTEMATIC EVENTS**
 50 EVENTS TO BE ANALYZED

 **END OF INPUT DATA**
 ED ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
 ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
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************************************************************
CAUTION FROM SUBROUTINE WTSKEW
***** NO GENERALIZED SKEW PROVIDED
ADOPTED SKEW SET TO COMPUTED SKEW



ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄPRELIMINARY RESULTS ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

-PLOTTING POSITIONS- 02481341 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º EVENTS ANALYZED ³ ORDERED EVENTS º
º TIDE ³ WATER TIDE MEDIAN º
º MON DAY YEAR FEET ³ RANK YEAR FEET PLOT POS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 0 0 1926 6.01 ³ 1 2005 24.17 1.39 º
º 0 0 1947 14.00 ³ 2 1969 19.68 3.37 º
º 0 0 1948 6.00 ³ 3 1947 14.00 5.36 º
º 0 0 1955 5.99 ³ 4 1965 10.70 7.34 º
º 0 0 1956 4.00 ³ 5 2002 7.61 9.33 º
º 0 0 1957 6.50 ³ 6 1998 7.05 11.31 º
º 0 0 1960 5.00 ³ 7 1985 6.50 13.29 º
º 0 0 1963 2.62 ³ 8 1957 6.50 15.28 º
º 0 0 1964 4.14 ³ 9 1926 6.01 17.26 º
º 0 0 1965 10.70 ³ 10 1948 6.00 19.25 º
º 0 0 1966 3.22 ³ 11 1979 6.00 21.23 º
º 0 0 1967 3.74 ³ 12 1955 5.99 23.21 º
º 0 0 1968 2.83 ³ 13 1973 5.33 25.20 º
º 0 0 1969 19.68 ³ 14 1960 5.00 27.18 º
º 0 0 1970 4.72 ³ 15 1988 4.77 29.17 º
º 0 0 1971 4.23 ³ 16 1970 4.72 31.15 º
º 0 0 1972 4.23 ³ 17 1984 4.70 33.13 º
º 0 0 1973 5.33 ³ 18 1974 4.68 35.12 º
º 0 0 1974 4.68 ³ 19 1986 4.65 37.10 º
º 0 0 1975 3.23 ³ 20 2004 4.63 39.09 º
º 0 0 1976 3.58 ³ 21 2001 4.43 41.07 º
º 0 0 1977 3.63 ³ 22 1972 4.23 43.06 º
º 0 0 1978 2.93 ³ 23 1971 4.23 45.04 º
º 0 0 1979 6.00 ³ 24 1964 4.14 47.02 º
º 0 0 1980 3.80 ³ 25 1997 4.12 49.01 º
º 0 0 1981 3.10 ³ 26 1983 4.05 50.99 º
º 0 0 1982 3.07 ³ 27 1999 4.05 52.98 º
º 0 0 1983 4.05 ³ 28 1990 4.01 54.96 º
º 0 0 1984 4.70 ³ 29 1991 4.00 56.94 º
º 0 0 1985 6.50 ³ 30 1956 4.00 58.93 º
º 0 0 1986 4.65 ³ 31 2003 3.98 60.91 º
º 0 0 1987 3.70 ³ 32 1992 3.89 62.90 º
º 0 0 1988 4.77 ³ 33 1980 3.80 64.88 º
º 0 0 1989 3.00 ³ 34 1967 3.74 66.87 º
º 0 0 1990 4.01 ³ 35 1987 3.70 68.85 º
º 0 0 1991 4.00 ³ 36 1977 3.63 70.83 º
º 0 0 1992 3.89 ³ 37 2000 3.62 72.82 º
º 0 0 1993 3.49 ³ 38 1976 3.58 74.80 º
º 0 0 1994 3.36 ³ 39 1995 3.57 76.79 º
º 0 0 1995 3.57 ³ 40 1993 3.49 78.77 º
º 0 0 1996 3.34 ³ 41 1994 3.36 80.75 º
º 0 0 1997 4.12 ³ 42 1996 3.34 82.74 º
º 0 0 1998 7.05 ³ 43 1975 3.23 84.72 º
º 0 0 1999 4.05 ³ 44 1966 3.22 86.71 º
º 0 0 2000 3.62 ³ 45 1981 3.10 88.69 º
º 0 0 2001 4.43 ³ 46 1982 3.07 90.67 º
```

```
º 0 0 2002 7.61 ³ 47 1989 3.00 92.66 º
º 0 0 2003 3.98 ³ 48 1978 2.93 94.64 º
º 0 0 2004 4.63 ³ 49 1968 2.83 96.63 º
º 0 0 2005 24.17 ³ 50 1963 2.62 98.61 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼



-SKEW WEIGHTING -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
BASED ON 50 EVENTS, MEAN-SQUARE ERROR OF STATION SKEW =-99.000
DEFAULT OR INPUT MEAN-SQUARE ERROR OF GENERALIZED SKEW = .302
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


PRELIMINARY RESULTS

-FREQUENCY CURVE- 02481341 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º COMPUTED EXPECTED ³ PERCENT ³ CONFIDENCE LIMITS º
º CURVE PROBABILITY ³ CHANCE ³ .05 .95 º
º TIDE IN FEET ³ EXCEEDANCE ³ TIDE IN FEET º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 50.00 64.60 ³ .2 ³ 81.30 35.20 º
º 33.00 39.50 ³ .5 ³ 49.50 24.50 º
º 24.10 27.40 ³ 1.0 ³ 34.00 18.70 º
º 17.50 19.20 ³ 2.0 ³ 23.40 14.20 º
º 11.60 12.10 ³ 5.0 ³ 14.30 9.83 º
º 8.44 8.67 ³ 10.0 ³ 9.95 7.41 º
º 6.16 6.23 ³ 20.0 ³ 6.98 5.52 º
º 4.06 4.06 ³ 50.0 ³ 4.52 3.62 º
º 3.28 3.27 ³ 80.0 ³ 3.67 2.87 º
º 3.11 3.10 ³ 90.0 ³ 3.49 2.70 º
º 3.03 3.03 ³ 95.0 ³ 3.41 2.63 º
º 2.98 2.97 ³ 99.0 ³ 3.35 2.58 º
ÌÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¹
º SYSTEMATIC STATISTICS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º LOG TRANSFORM: TIDE, FEET ³ NUMBER OF EVENTS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º MEAN .6693 ³ HISTORIC EVENTS 0 º
º STANDARD DEV .1974 ³ HIGH OUTLIERS 0 º
º COMPUTED SKEW 1.9575 ³ LOW OUTLIERS 0 º
º REGIONAL SKEW -99.0000 ³ ZERO OR MISSING 0 º
º ADOPTED SKEW 2.0000 ³ SYSTEMATIC EVENTS 50 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼



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CAUTION FROM SUBROUTINE WTSKEW
***** NO GENERALIZED SKEW PROVIDED
ADOPTED SKEW SET TO COMPUTED SKEW
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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ FINAL RESULTS ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

-PLOTTING POSITIONS- 02481341 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º EVENTS ANALYZED ³ ORDERED EVENTS º
º TIDE ³ WATER TIDE MEDIAN º
º MON DAY YEAR FEET ³ RANK YEAR FEET PLOT POS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 0 0 1915 9.00 ³ 1 2005 24.17 .77 º
º 0 0 1926 6.01 ³ 2 1969 19.68 1.86 º
º 0 0 1947 14.00 ³ 3 1947 14.00 2.95 º
º 0 0 1948 6.00 ³ 4 1965 10.70 4.05 º
º 0 0 1955 5.99 ³ 5 1915 9.00 5.14 º
º 0 0 1956 4.00 ³ 6 2002 7.61 6.71 º
º 0 0 1957 6.50 ³ 7 1998 7.05 8.76 º
º 0 0 1960 5.00 ³ 8 1957 6.50 10.80 º
º 0 0 1963 2.62 ³ 9 1985 6.50 12.85 º
º 0 0 1964 4.14 ³ 10 1926 6.01 14.89 º
º 0 0 1965 10.70 ³ 11 1979 6.00 16.94 º
º 0 0 1966 3.22 ³ 12 1948 6.00 18.98 º
º 0 0 1967 3.74 ³ 13 1955 5.99 21.03 º
º 0 0 1968 2.83 ³ 14 1973 5.33 23.08 º
º 0 0 1969 19.68 ³ 15 1960 5.00 25.12 º
º 0 0 1970 4.72 ³ 16 1988 4.77 27.17 º
º 0 0 1971 4.23 ³ 17 1970 4.72 29.21 º
º 0 0 1972 4.23 ³ 18 1984 4.70 31.26 º
º 0 0 1973 5.33 ³ 19 1974 4.68 33.30 º
º 0 0 1974 4.68 ³ 20 1986 4.65 35.35 º
º 0 0 1975 3.23 ³ 21 2004 4.63 37.39 º
º 0 0 1976 3.58 ³ 22 2001 4.43 39.44 º
º 0 0 1977 3.63 ³ 23 1971 4.23 41.49 º
º 0 0 1978 2.93 ³ 24 1972 4.23 43.53 º
º 0 0 1979 6.00 ³ 25 1964 4.14 45.58 º
º 0 0 1980 3.80 ³ 26 1997 4.12 47.62 º
º 0 0 1981 3.10 ³ 27 1999 4.05 49.67 º
º 0 0 1982 3.07 ³ 28 1983 4.05 51.71 º
º 0 0 1983 4.05 ³ 29 1990 4.01 53.76 º
º 0 0 1984 4.70 ³ 30 1956 4.00 55.80 º
º 0 0 1985 6.50 ³ 31 1991 4.00 57.85 º
º 0 0 1986 4.65 ³ 32 2003 3.98 59.89 º
º 0 0 1987 3.70 ³ 33 1992 3.89 61.94 º
º 0 0 1988 4.77 ³ 34 1980 3.80 63.99 º
º 0 0 1989 3.00 ³ 35 1967 3.74 66.03 º
º 0 0 1990 4.01 ³ 36 1987 3.70 68.08 º
º 0 0 1991 4.00 ³ 37 1977 3.63 70.12 º
º 0 0 1992 3.89 ³ 38 2000 3.62 72.17 º
º 0 0 1993 3.49 ³ 39 1976 3.58 74.21 º
º 0 0 1994 3.36 ³ 40 1995 3.57 76.26 º
º 0 0 1995 3.57 ³ 41 1993 3.49 78.30 º
º 0 0 1996 3.34 ³ 42 1994 3.36 80.35 º
º 0 0 1997 4.12 ³ 43 1996 3.34 82.39 º
º 0 0 1998 7.05 ³ 44 1975 3.23 84.44 º
º 0 0 1999 4.05 ³ 45 1966 3.22 86.49 º
º 0 0 2000 3.62 ³ 46 1981 3.10 88.53 º
º 0 0 2001 4.43 ³ 47 1982 3.07 90.58 º
º 0 0 2002 7.61 ³ 48 1989 3.00 92.62 º
º 0 0 2003 3.98 ³ 49 1978 2.93 94.67 º
º 0 0 2004 4.63 ³ 50 1968 2.83 96.71 º
º 0 0 2005 24.17 ³ 51 1963 2.62 98.76 º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÁÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º NOTE- PLOTTING POSITIONS BASED ON-HISTORIC PERIOD (H) = 91 º
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º NUMBER OF HISTORIC EVENTS PLUS HIGH OUTLIERS(Z) = 5 º
º WEIGHTING FACTOR FOR SYSTEMATIC EVENTS (W) = 1.8696 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼


-OUTLIER TESTS -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
HIGH OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 50 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 2.768

4 HIGH OUTLIER(S) IDENTIFIED ABOVE TEST VALUE OF 16.
OR INPUT BASE OF 9.

NOTE - INPUT BASE NOT SPECIFIED
THEREFORE BASE SET TO DEFAULT (LOWEST HISTORIC PEAK)

NOTE - COLLECTION OF HISTORICAL INFORMATION AND COMPARISONS
WITH SIMILAR DATA SETS SHOULD BE EXPLORED IF NOT
INCORPORATED IN THIS ANALYSIS.

STATISTICS AND FREQUENCY CURVE ADJUSTED FOR 4 HIGH OUTLIER(S)
AND 1 HISTORIC EVENT(S)


ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
LOW OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 91 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 2.984

0 LOW OUTLIER(S) IDENTIFIED BELOW TEST VALUE OF 1.4
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


-SKEW WEIGHTING -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
BASED ON 91 EVENTS, MEAN-SQUARE ERROR OF STATION SKEW =-99.000
DEFAULT OR INPUT MEAN-SQUARE ERROR OF GENERALIZED SKEW = .302
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ



FINAL RESULTS

-FREQUENCY CURVE- 02481341 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º COMPUTED EXPECTED ³ PERCENT ³ CONFIDENCE LIMITS º
º CURVE PROBABILITY ³ CHANCE ³ .05 .95 º
º TIDE IN FEET ³ EXCEEDANCE ³ TIDE IN FEET º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 33.60 41.80 ³ .2 ³ 50.80 24.90 º
º 23.60 27.50 ³ .5 ³ 33.30 18.30 º
º 18.00 20.10 ³ 1.0 ³ 24.20 14.50 º
º 13.80 14.90 ³ 2.0 ³ 17.60 11.50 º
º 9.69 10.10 ³ 5.0 ³ 11.60 8.44 º
º 7.41 7.58 ³ 10.0 ³ 8.52 6.64 º
º 5.67 5.73 ³ 20.0 ³ 6.31 5.17 º
º 3.98 3.98 ³ 50.0 ³ 4.36 3.62 º
º 3.32 3.32 ³ 80.0 ³ 3.66 2.97 º
```

```
 º 3.17 3.17 ³ 90.0 ³ 3.50 2.82 º
 º 3.11 3.10 ³ 95.0 ³ 3.44 2.75 º
 º 3.06 3.06 ³ 99.0 ³ 3.39 2.71 º
 ÌÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¹
 º ADJUSTED STATISTICS º
 ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
 º LOG TRANSFORM: TIDE, FEET ³ NUMBER OF EVENTS º
 ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
 º MEAN .6517 ³ HISTORIC EVENTS 1 º
 º STANDARD DEV .1677 ³ HIGH OUTLIERS 4 º
 º COMPUTED SKEW 1.9740 ³ LOW OUTLIERS 0 º
 º REGIONAL SKEW -99.0000 ³ ZERO OR MISSING 0 º
 º ADOPTED SKEW 2.0000 ³ SYSTEMATIC EVENTS 50 º
 º ³ HISTORIC PERIOD 91 º
 ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼


 TABLE 1. SUMMARY OF STATISTICS -- FINAL RESULTS
--------------------------------------------------------------------------------------
-------------------------------------
 STATION STATION NAME AND LOCATION AREA ....YEARS..... MEAN STD .......SKEW........
HIST OUTLIER ZERO/
 NUMBER .............................................. SQ MI RECD SYST HIST LOG DEV
ADOPT COMP GENRL EVENT HI LO MSNG
--------------------------------------------------------------------------------------
-------------------------------------
 02481341 GULFPORT, MS 50 50 91 .652 .168 2.00 1.974 -99.00 1 4 0 0
--------------------------------------------------------------------------------------
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 TABLE 2. SUMMARY OF FREQUENCY CURVE ORDINATES -- FINAL RESULTS
--------------------------------------------------------------------------------------
-------------------------------------
 STATION STATION NAME AND LOCATION AREA ....YEARS..... ............PERCENT CHANCE
EXCEEDANCE...........
 NUMBER .............................................. SQ MI RECD SYST HIST 10. 5.
2. 1. .5 .2
--------------------------------------------------------------------------------------
-------------------------------------
 02481341 GULFPORT, MS 50 50 91 7 9 13 18 23 33
--------------------------------------------------------------------------------------
-------------------------------------


 +++++++++++++++++++++++++
 + END OF RUN +
 + NORMAL STOP IN FFA +
 +++++++++++++++++++++++++
```

```
************************************ ************************************
 * FFA * * *
 * FLOOD FREQUENCY ANALYSIS * * U.S. ARMY CORPS OF ENGINEERS *
 * PROGRAM DATE: MAY 1992 * * THE HYDROLOGIC ENGINEERING CENTER *
 * VERSION: 3.0 * * 609 SECOND STREET *
 * RUN DATE AND TIME: * * DAVIS, CALIFORNIA 95616 *
 * 06 APR 06 16:17:17 * * (916) 756-1104 *
 * * * *
 ************************************ ************************************



 INPUT FILE NAME: TIDEANUL.DAT
 OUTPUT FILE NAME: TIDEANUL.OUT
 DSS FILE NAME: TIDEANUL.DSS



 -----DSS---ZOPEN: Existing File Opened, File: TIDEANUL.DSS
 Unit: 71; DSS Version: 6-GX

 **TITLE RECORD(S)**
 TT BILOXI, MS TIDE ELEVATION FREQUENCY ANALYSIS
 TT GRAPHICAL ANALYSIS OF MAXIMUM TIDE ELEVATION
 TT GRAPHICAL ANALYSIS USING MEDIAN PLOTTING POSITIONS
 TT HISTORICAL OBSERVATIONS INCLUDED
 TT 111 YEARS 1882-2005, MISSING YEARS 1883,1886-1895, 1977-1978
 TT DATA SET ANALYZED AS A SINGLE RECORD

 **JOB RECORD(S)**
 IPPC ISKFX IPROUT IFMT IWYR IUNIT ISMRY IPNCH IREG
 J1 2 0 0 1 0 3 1 0 0

 **SPECIFIED VARIABLE AND UNITS**
 FU TIDE FEET

 **STATION IDENTIFICATION**
 ID 02480351 GULFPORT, MS 1

 **SPECIAL STATION INFORMATION**
 IYRA IYRL HITHRS LOTHRS LOGT NDEC NSIG
 SI 0 0 0. 0. 0 2 0

 **HP PLOT **
 HP PLOT FILE IHPCV KLIMIT IPER BAREA
 HP BILXI.PCL 0 0 0

 SELECTED CURVES ON HPPLOT
 EXPECTED PROBABILITY CURVE
 CONFIDENCE LIMITS

 HP BILOXI, MISSISSIPPI

 **SYSTEMATIC EVENTS**
 111 EVENTS TO BE ANALYZED

 **END OF INPUT DATA**
 ED ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
 ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++


 ************************************************************
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```
CAUTION FROM SUBROUTINE WTSKEW
***** NO GENERALIZED SKEW PROVIDED
ADOPTED SKEW SET TO COMPUTED SKEW



ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ FINAL RESULTS ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

-PLOTTING POSITIONS- 02480351 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º EVENTS ANALYZED ³ ORDERED EVENTS º
º TIDE ³ WATER TIDE MEDIAN º
º MON DAY YEAR FEET ³ RANK YEAR FEET PLOT POS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 0 0 1882 2.29 ³ 1 2005 23.80 .63 º
º 0 0 1884 2.27 ³ 2 1969 15.56 1.53 º
º 0 0 1885 1.94 ³ 3 1947 10.80 2.42 º
º 0 0 1896 2.53 ³ 4 1915 8.92 3.32 º
º 0 0 1897 3.10 ³ 5 1965 8.56 4.22 º
º 0 0 1898 3.29 ³ 6 1998 8.05 5.12 º
º 0 0 1899 2.35 ³ 7 2002 6.86 6.01 º
º 0 0 1900 3.29 ³ 8 1988 6.26 6.91 º
º 0 0 1901 1.94 ³ 9 1985 6.03 7.81 º
º 0 0 1902 2.17 ³ 10 1906 5.92 8.71 º
º 0 0 1903 2.46 ³ 11 1923 5.88 9.61 º
º 0 0 1904 2.57 ³ 12 1973 5.72 10.50 º
º 0 0 1905 3.82 ³ 13 1979 5.62 11.40 º
º 0 0 1906 5.92 ³ 14 1948 5.60 12.30 º
º 0 0 1907 2.64 ³ 15 1920 5.44 13.20 º
º 0 0 1908 3.04 ³ 16 1960 5.12 14.09 º
º 0 0 1909 4.35 ³ 17 1972 4.99 14.99 º
º 0 0 1910 2.37 ³ 18 1956 4.70 15.89 º
º 0 0 1911 4.36 ³ 19 1957 4.64 16.79 º
º 0 0 1912 3.38 ³ 20 1964 4.63 17.68 º
º 0 0 1913 2.62 ³ 21 1919 4.51 18.58 º
º 0 0 1914 3.32 ³ 22 1974 4.47 19.48 º
º 0 0 1915 8.92 ³ 23 1949 4.46 20.38 º
º 0 0 1916 4.07 ³ 24 1934 4.44 21.27 º
º 0 0 1917 2.53 ³ 25 1984 4.43 22.17 º
º 0 0 1918 3.37 ³ 26 1983 4.40 23.07 º
º 0 0 1919 4.51 ³ 27 1911 4.36 23.97 º
º 0 0 1920 5.44 ³ 28 1909 4.35 24.87 º
º 0 0 1921 2.89 ³ 29 1940 4.32 25.76 º
º 0 0 1922 2.83 ³ 30 1992 4.32 26.66 º
º 0 0 1923 5.88 ³ 31 1999 4.25 27.56 º
º 0 0 1924 2.66 ³ 32 2004 4.23 28.46 º
º 0 0 1925 2.22 ³ 33 1961 4.21 29.35 º
º 0 0 1926 3.82 ³ 34 1945 4.13 30.25 º
º 0 0 1927 3.52 ³ 35 1916 4.07 31.15 º
º 0 0 1928 3.39 ³ 36 2003 3.99 32.05 º
º 0 0 1929 2.94 ³ 37 1987 3.97 32.94 º
º 0 0 1930 3.03 ³ 38 1933 3.92 33.84 º
º 0 0 1931 3.29 ³ 39 1971 3.90 34.74 º
º 0 0 1932 3.67 ³ 40 1950 3.87 35.64 º
º 0 0 1933 3.92 ³ 41 1966 3.83 36.54 º
º 0 0 1934 4.44 ³ 42 1926 3.82 37.43 º
º 0 0 1935 3.43 ³ 43 1905 3.82 38.33 º
º 0 0 1936 2.74 ³ 44 1993 3.80 39.23 º
º 0 0 1937 2.94 ³ 45 1997 3.74 40.13 º
º 0 0 1938 3.20 ³ 46 1932 3.67 41.02 º
º 0 0 1939 3.42 ³ 47 1990 3.67 41.92 º
º 0 0 1940 4.32 ³ 48 1991 3.63 42.82 º
º 0 0 1941 3.45 ³ 49 1970 3.59 43.72 º
```

```
º 0 0 1942 2.94 ³ 50 1955 3.54 44.61 º
º 0 0 1943 2.92 ³ 51 1996 3.53 45.51 º
º 0 0 1944 3.02 ³ 52 1927 3.52 46.41 º
º 0 0 1945 4.13 ³ 53 1952 3.48 47.31 º
º 0 0 1946 3.27 ³ 54 1941 3.45 48.20 º
º 0 0 1947 10.80 ³ 55 1935 3.43 49.10 º
º 0 0 1948 5.60 ³ 56 2001 3.43 50.00 º
º 0 0 1949 4.46 ³ 57 1939 3.42 50.90 º
º 0 0 1950 3.87 ³ 58 1995 3.39 51.80 º
º 0 0 1951 3.24 ³ 59 1928 3.39 52.69 º
º 0 0 1952 3.48 ³ 60 1912 3.38 53.59 º
º 0 0 1953 3.34 ³ 61 1967 3.38 54.49 º
º 0 0 1954 3.15 ³ 62 1918 3.37 55.39 º
º 0 0 1955 3.54 ³ 63 1989 3.35 56.28 º
º 0 0 1956 4.70 ³ 64 2000 3.35 57.18 º
º 0 0 1957 4.64 ³ 65 1953 3.34 58.08 º
º 0 0 1958 2.89 ³ 66 1986 3.34 58.98 º
º 0 0 1959 2.82 ³ 67 1914 3.32 59.87 º
º 0 0 1960 5.12 ³ 68 1994 3.31 60.77 º
º 0 0 1961 4.21 ³ 69 1931 3.29 61.67 º
º 0 0 1962 2.21 ³ 70 1900 3.29 62.57 º
º 0 0 1963 2.73 ³ 71 1898 3.29 63.46 º
º 0 0 1964 4.63 ³ 72 1946 3.27 64.36 º
º 0 0 1965 8.56 ³ 73 1980 3.25 65.26 º
º 0 0 1966 3.83 ³ 74 1951 3.24 66.16 º
º 0 0 1967 3.38 ³ 75 1938 3.20 67.06 º
º 0 0 1968 2.41 ³ 76 1954 3.15 67.95 º
º 0 0 1969 15.56 ³ 77 1897 3.10 68.85 º
º 0 0 1970 3.59 ³ 78 1908 3.04 69.75 º
º 0 0 1971 3.90 ³ 79 1930 3.03 70.65 º
º 0 0 1972 4.99 ³ 80 1944 3.02 71.54 º
º 0 0 1973 5.72 ³ 81 1929 2.94 72.44 º
º 0 0 1974 4.47 ³ 82 1942 2.94 73.34 º
º 0 0 1975 2.89 ³ 83 1937 2.94 74.24 º
º 0 0 1976 2.72 ³ 84 1982 2.92 75.13 º
º 0 0 1979 5.62 ³ 85 1943 2.92 76.03 º
º 0 0 1980 3.25 ³ 86 1958 2.89 76.93 º
º 0 0 1981 2.70 ³ 87 1921 2.89 77.83 º
º 0 0 1982 2.92 ³ 88 1975 2.89 78.73 º
º 0 0 1983 4.40 ³ 89 1922 2.83 79.62 º
º 0 0 1984 4.43 ³ 90 1959 2.82 80.52 º
º 0 0 1985 6.03 ³ 91 1936 2.74 81.42 º
º 0 0 1986 3.34 ³ 92 1963 2.73 82.32 º
º 0 0 1987 3.97 ³ 93 1976 2.72 83.21 º
º 0 0 1988 6.26 ³ 94 1981 2.70 84.11 º
º 0 0 1989 3.35 ³ 95 1924 2.66 85.01 º
º 0 0 1990 3.67 ³ 96 1907 2.64 85.91 º
º 0 0 1991 3.63 ³ 97 1913 2.62 86.80 º
º 0 0 1992 4.32 ³ 98 1904 2.57 87.70 º
º 0 0 1993 3.80 ³ 99 1896 2.53 88.60 º
º 0 0 1994 3.31 ³ 100 1917 2.53 89.50 º
º 0 0 1995 3.39 ³ 101 1903 2.46 90.39 º
º 0 0 1996 3.53 ³ 102 1968 2.41 91.29 º
º 0 0 1997 3.74 ³ 103 1910 2.37 92.19 º
º 0 0 1998 8.05 ³ 104 1899 2.35 93.09 º
º 0 0 1999 4.25 ³ 105 1882 2.29 93.99 º
º 0 0 2000 3.35 ³ 106 1884 2.27 94.88 º
º 0 0 2001 3.43 ³ 107 1925 2.22 95.78 º
º 0 0 2002 6.86 ³ 108 1962 2.21 96.68 º
º 0 0 2003 3.99 ³ 109 1902 2.17 97.58 º
º 0 0 2004 4.23 ³ 110 1901 1.94 98.47 º
º 0 0 2005 23.80 ³ 111 1885 1.94 99.37 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼
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-OUTLIER TESTS -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
HIGH OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 111 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 3.052

2 HIGH OUTLIER(S) IDENTIFIED ABOVE TEST VALUE OF 12.

NOTE - COLLECTION OF HISTORICAL INFORMATION AND COMPARISONS
WITH SIMILAR DATA SETS SHOULD BE EXPLORED IF NOT
INCORPORATED IN THIS ANALYSIS.


ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
LOW OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 111 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 3.052

0 LOW OUTLIER(S) IDENTIFIED BELOW TEST VALUE OF 1.1
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


-SKEW WEIGHTING -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
BASED ON 111 EVENTS, MEAN-SQUARE ERROR OF STATION SKEW =-99.000
DEFAULT OR INPUT MEAN-SQUARE ERROR OF GENERALIZED SKEW = .302
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ



FINAL RESULTS

-FREQUENCY CURVE- 02480351 GULFPORT, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º COMPUTED EXPECTED ³ PERCENT ³ CONFIDENCE LIMITS º
º CURVE PROBABILITY ³ CHANCE ³ .05 .95 º
º TIDE IN FEET ³ EXCEEDANCE ³ TIDE IN FEET º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 25.80 28.20 ³ .2 ³ 33.20 21.10 º
º 18.60 19.80 ³ .5 ³ 22.90 15.60 º
º 14.40 15.10 ³ 1.0 ³ 17.30 12.50 º
º 11.20 11.60 ³ 2.0 ³ 13.10 9.90 º
º 8.01 8.14 ³ 5.0 ³ 8.99 7.27 º
º 6.19 6.25 ³ 10.0 ³ 6.77 5.73 º
º 4.76 4.78 ³ 20.0 ³ 5.11 4.47 º
º 3.33 3.33 ³ 50.0 ³ 3.54 3.12 º
º 2.72 2.72 ³ 80.0 ³ 2.91 2.53 º
º 2.57 2.57 ³ 90.0 ³ 2.76 2.38 º
º 2.50 2.50 ³ 95.0 ³ 2.68 2.31 º
º 2.43 2.43 ³ 99.0 ³ 2.62 2.24 º
ÌÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¹
º SYSTEMATIC STATISTICS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º LOG TRANSFORM: TIDE, FEET ³ NUMBER OF EVENTS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º MEAN .5696 ³ HISTORIC EVENTS 0 º
º STANDARD DEV .1685 ³ HIGH OUTLIERS 0 º
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 º COMPUTED SKEW 1.7824 ³ LOW OUTLIERS 0 º
 º REGIONAL SKEW -99.0000 ³ ZERO OR MISSING 0 º
 º ADOPTED SKEW 1.8000 ³ SYSTEMATIC EVENTS 111 º
 ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼


 TABLE 1. SUMMARY OF STATISTICS -- FINAL RESULTS
--------------------------------------------------------------------------------------
-------------------------------------
 STATION STATION NAME AND LOCATION AREA ....YEARS..... MEAN STD .......SKEW........
HIST OUTLIER ZERO/
 NUMBER ................................................ SQ MI RECD SYST HIST LOG DEV
ADOPT COMP GENRL EVENT HI LO MSNG
--------------------------------------------------------------------------------------
-------------------------------------
 02480351 GULFPORT, MS 111 111 0 .570 .169 1.80 1.782 -99.00 0 0 0 0
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 TABLE 2. SUMMARY OF FREQUENCY CURVE ORDINATES -- FINAL RESULTS
--------------------------------------------------------------------------------------
-------------------------------------
 STATION STATION NAME AND LOCATION AREA ....YEARS..... ............PERCENT CHANCE
EXCEEDANCE...........
NUMBER ................................................ SQ MI RECD SYST HIST 10. 5. 2.
1. .5 .2
--------------------------------------------------------------------------------------
-------------------------------------
 02480351 GULFPORT, MS 111 111 0 6 8 11 14 18 25
--------------------------------------------------------------------------------------
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 +++++++++++++++++++++++++
 + END OF RUN +
 + NORMAL STOP IN FFA +
 +++++++++++++++++++++++++
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************************************ ************************************
 * FFA * * *
 * FLOOD FREQUENCY ANALYSIS * * U.S. ARMY CORPS OF ENGINEERS *
 * PROGRAM DATE: MAY 1992 * * THE HYDROLOGIC ENGINEERING CENTER *
 * VERSION: 3.0 * * 609 SECOND STREET *
 * RUN DATE AND TIME: * * DAVIS, CALIFORNIA 95616 *
 * 06 APR 06 11:55:07 * * (916) 756-1104 *
 * * * *
 ************************************ ************************************

 INPUT FILE NAME: TIDEANUL.DAT
 OUTPUT FILE NAME: TIDEANUL.OUT
 DSS FILE NAME: TIDEANUL.DSS

 -----DSS---ZOPEN: Existing File Opened, File: TIDEANUL.DSS
 Unit: 71; DSS Version: 6-GX

 **TITLE RECORD(S)**
 TT PASCAGOULA, MS TIDE ELEVATION FREQUENCY ANALYSIS
 TT GRAPHICAL ANALYSIS OF MAXIMUM TIDE ELEVATION
 TT GRAPHICAL ANALYSIS USING MEDIAN PLOTTING POSITIONS
 TT HISTORICAL OBSERVATIONS INCLUDED
 TT 66 YEARS 1940-2005

 **JOB RECORD(S)**
 IPPC ISKFX IPROUT IFMT IWYR IUNIT ISMRY IPNCH IREG
 J1 2 0 0 1 0 3 1 0 0

 **SPECIFIED VARIABLE AND UNITS**
 FU TIDE FEET

 **STATION IDENTIFICATION**
 ID 02480301 PASCAGOULA, MS 1

 **SPECIAL STATION INFORMATION**
 IYRA IYRL HITHRS LOTHRS LOGT NDEC NSIG
 SI 0 0 0. 0. 0 2 0

 **HP PLOT **
 HP PLOT FILE IHPCV KLIMIT IPER BAREA
 HP PASCA.PCL 0 0 0

 SELECTED CURVES ON HPPLOT
 EXPECTED PROBABILITY CURVE
 CONFIDENCE LIMITS

 HP PASCAGOULA, MISSISSIPPI

 **SYSTEMATIC EVENTS**
 66 EVENTS TO BE ANALYZED

 **END OF INPUT DATA**
 ED ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
 ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++

 ************************************************************
 CAUTION FROM SUBROUTINE WTSKEW
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***** NO GENERALIZED SKEW PROVIDED
ADOPTED SKEW SET TO COMPUTED SKEW



ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ FINAL RESULTS ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

-PLOTTING POSITIONS- 02480301 PASCAGOULA, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º EVENTS ANALYZED ³ ORDERED EVENTS º
º TIDE ³ WATER TIDE MEDIAN º
º MON DAY YEAR FEET ³ RANK YEAR FEET PLOT POS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 0 0 1940 3.63 ³ 1 2005 15.78 1.05 º
º 0 0 1941 3.30 ³ 2 1969 11.24 2.56 º
º 0 0 1942 2.35 ³ 3 1998 8.36 4.07 º
º 0 0 1943 2.35 ³ 4 1947 7.68 5.57 º
º 0 0 1944 2.38 ³ 5 2004 6.72 7.08 º
º 0 0 1945 3.37 ³ 6 1965 6.40 8.58 º
º 0 0 1946 2.68 ³ 7 1979 5.78 10.09 º
º 0 0 1947 7.68 ³ 8 2002 5.75 11.60 º
º 0 0 1948 4.00 ³ 9 1985 5.50 13.10 º
º 0 0 1949 3.90 ³ 10 1972 5.26 14.61 º
º 0 0 1950 3.65 ³ 11 1960 4.50 16.11 º
º 0 0 1951 2.85 ³ 12 1964 4.05 17.62 º
º 0 0 1952 3.15 ³ 13 1948 4.00 19.13 º
º 0 0 1953 3.05 ³ 14 1949 3.90 20.63 º
º 0 0 1954 2.65 ³ 15 2001 3.90 22.14 º
º 0 0 1955 3.10 ³ 16 1974 3.86 23.64 º
º 0 0 1956 3.40 ³ 17 1970 3.81 25.15 º
º 0 0 1957 3.28 ³ 18 1961 3.80 26.66 º
º 0 0 1958 2.80 ³ 19 1984 3.71 28.16 º
º 0 0 1959 2.80 ³ 20 1983 3.68 29.67 º
º 0 0 1960 4.50 ³ 21 1950 3.65 31.17 º
º 0 0 1961 3.80 ³ 22 1940 3.63 32.68 º
º 0 0 1962 2.13 ³ 23 1980 3.53 34.19 º
º 0 0 1963 2.76 ³ 24 1987 3.53 35.69 º
º 0 0 1964 4.05 ³ 25 1993 3.45 37.20 º
º 0 0 1965 6.40 ³ 26 1956 3.40 38.70 º
º 0 0 1966 2.84 ³ 27 1945 3.37 40.21 º
º 0 0 1967 3.33 ³ 28 1971 3.35 41.72 º
º 0 0 1968 2.19 ³ 29 1967 3.33 43.22 º
º 0 0 1969 11.24 ³ 30 2003 3.33 44.73 º
º 0 0 1970 3.81 ³ 31 1941 3.30 46.23 º
º 0 0 1971 3.35 ³ 32 1992 3.28 47.74 º
º 0 0 1972 5.26 ³ 33 1996 3.28 49.25 º
º 0 0 1973 2.86 ³ 34 1957 3.28 50.75 º
º 0 0 1974 3.86 ³ 35 1986 3.24 52.26 º
º 0 0 1975 2.81 ³ 36 1952 3.15 53.77 º
º 0 0 1976 2.57 ³ 37 1955 3.10 55.27 º
º 0 0 1977 2.38 ³ 38 1953 3.05 56.78 º
º 0 0 1978 2.92 ³ 39 1988 3.03 58.28 º
º 0 0 1979 5.78 ³ 40 1991 3.03 59.79 º
º 0 0 1980 3.53 ³ 41 2000 3.00 61.30 º
º 0 0 1981 2.46 ³ 42 1978 2.92 62.80 º
º 0 0 1982 2.75 ³ 43 1990 2.88 64.31 º
º 0 0 1983 3.68 ³ 44 1989 2.87 65.81 º
º 0 0 1984 3.71 ³ 45 1997 2.87 67.32 º
º 0 0 1985 5.50 ³ 46 1973 2.86 68.83 º
º 0 0 1986 3.24 ³ 47 1951 2.85 70.33 º
º 0 0 1987 3.53 ³ 48 1994 2.84 71.84 º
º 0 0 1988 3.03 ³ 49 1966 2.84 73.34 º
º 0 0 1989 2.87 ³ 50 1975 2.81 74.85 º
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º 0 0 1990 2.88 ³ 51 1958 2.80 76.36 º
º 0 0 1991 3.03 ³ 52 1959 2.80 77.86 º
º 0 0 1992 3.28 ³ 53 1963 2.76 79.37 º
º 0 0 1993 3.45 ³ 54 1995 2.75 80.87 º
º 0 0 1994 2.84 ³ 55 1982 2.75 82.38 º
º 0 0 1995 2.75 ³ 56 1999 2.68 83.89 º
º 0 0 1996 3.28 ³ 57 1946 2.68 85.39 º
º 0 0 1997 2.87 ³ 58 1954 2.65 86.90 º
º 0 0 1998 8.36 ³ 59 1976 2.57 88.40 º
º 0 0 1999 2.68 ³ 60 1981 2.46 89.91 º
º 0 0 2000 3.00 ³ 61 1944 2.38 91.42 º
º 0 0 2001 3.90 ³ 62 1977 2.38 92.92 º
º 0 0 2002 5.75 ³ 63 1943 2.35 94.43 º
º 0 0 2003 3.33 ³ 64 1942 2.35 95.93 º
º 0 0 2004 6.72 ³ 65 1968 2.19 97.44 º
º 0 0 2005 15.78 ³ 66 1962 2.13 98.95 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼


-OUTLIER TESTS -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
HIGH OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 66 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 2.871

2 HIGH OUTLIER(S) IDENTIFIED ABOVE TEST VALUE OF 10.

NOTE - COLLECTION OF HISTORICAL INFORMATION AND COMPARISONS
WITH SIMILAR DATA SETS SHOULD BE EXPLORED IF NOT
INCORPORATED IN THIS ANALYSIS.


ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
LOW OUTLIER TEST
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


BASED ON 66 EVENTS, 10 PERCENT OUTLIER TEST VALUE K(N) = 2.871

0 LOW OUTLIER(S) IDENTIFIED BELOW TEST VALUE OF 1.2
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ


-SKEW WEIGHTING -
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
BASED ON 66 EVENTS, MEAN-SQUARE ERROR OF STATION SKEW =-99.000
DEFAULT OR INPUT MEAN-SQUARE ERROR OF GENERALIZED SKEW = .302
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ



FINAL RESULTS

-FREQUENCY CURVE- 02480301 PASCAGOULA, MS
ÉÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÑÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ»
º COMPUTED EXPECTED ³ PERCENT ³ CONFIDENCE LIMITS º
º CURVE PROBABILITY ³ CHANCE ³ .05 .95 º
º TIDE IN FEET ³ EXCEEDANCE ³ TIDE IN FEET º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º 23.80 27.70 ³ .2 ³ 33.20 18.50 º
º 17.10 19.00 ³ .5 ³ 22.60 13.80 º
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º 13.30 14.40 ³ 1.0 ³ 16.90 11.10 º
º 10.30 10.90 ³ 2.0 ³ 12.60 8.87 º
º 7.41 7.62 ³ 5.0 ³ 8.60 6.58 º
º 5.75 5.84 ³ 10.0 ³ 6.45 5.22 º
º 4.45 4.48 ³ 20.0 ³ 4.86 4.11 º
º 3.15 3.15 ³ 50.0 ³ 3.40 2.90 º
º 2.62 2.61 ³ 80.0 ³ 2.84 2.38 º
º 2.49 2.49 ³ 90.0 ³ 2.71 2.26 º
º 2.43 2.43 ³ 95.0 ³ 2.65 2.20 º
º 2.38 2.38 ³ 99.0 ³ 2.60 2.15 º
ÌÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¹
º SYSTEMATIC STATISTICS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÂÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º LOG TRANSFORM: TIDE, FEET ³ NUMBER OF EVENTS º
ÇÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÅÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ¶
º MEAN .5461 ³ HISTORIC EVENTS 0 º
º STANDARD DEV .1627 ³ HIGH OUTLIERS 0 º
º COMPUTED SKEW 1.8707 ³ LOW OUTLIERS 0 º
º REGIONAL SKEW -99.0000 ³ ZERO OR MISSING 0 º
º ADOPTED SKEW 1.9000 ³ SYSTEMATIC EVENTS 66 º
ÈÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÏÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍÍ¼


TABLE 1. SUMMARY OF STATISTICS -- FINAL RESULTS
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-------------------------------------
STATION STATION NAME AND LOCATION AREA ....YEARS..... MEAN STD .......SKEW........
HIST OUTLIER ZERO/
NUMBER ................................................ SQ MI RECD SYST HIST LOG DEV
ADOPT COMP GENRL EVENT HI LO MSNG
--------------------------------------------------------------------------------------
-------------------------------------
02480301 PASCAGOULA, MS 66 66 0 .546 .163 1.90 1.871 -99.00 0 0 0 0
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TABLE 2. SUMMARY OF FREQUENCY CURVE ORDINATES -- FINAL RESULTS
--------------------------------------------------------------------------------------
-------------------------------------
STATION STATION NAME AND LOCATION AREA ....YEARS..... ............PERCENT CHANCE
EXCEEDANCE...........
NUMBER ................................................ SQ MI RECD SYST HIST 10. 5.
2. 1. .5 .2
--------------------------------------------------------------------------------------
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02480301 PASCAGOULA, MS 66 66 0 5 7 10 13 17 23
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+ END OF RUN +
+ NORMAL STOP IN FFA +
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Reference

"STORM SURGE MEASURED AT WATER LEVEL STATIONS DURING HURRICANES CHARLEY, FRANCES, IVAN & JEANNE", Gerald T. Hovis, U.S. Department of Commerce, NOAA National Ocean Service

HISTORIC AND EXISTING WIND, WAVE, WATER LEVEL, AND SEDIMENT TRANSPORT CONDITIONS

The Mississippi Sound extends from Mobile Bay, Alabama, to the east to Lake Bornge, Louisiana, to the west. The Sound is a mostly unstratified brackish water body approximately 81miles long, 6.8 to 15 miles wide, and 820 square miles in area, which averages approximately 10 ft in depth. xico coast from The Sound extends about nine miles north to south from the Mississippi mainland coastline to a series of offshore barrier islands which separate the Mississippi Sound from the Gulf of Mexico. The Sound has a mean depth of 10 ft Mean Low Water (MLW) and more than 99% of the system is shallower then 20 ft MLW.

Winds

Prevailing winds for the Mississippi coast are produced by two pressure ridges which dominate weather conditions: the Bermuda High, centered over the Bermuda-Azores area of the Atlantic and the Mexican Heat Low centered over Texas during warm months. Prevailing winds are predominately from the east and south east during spring and summer months, and from the east and north east during fall and winter months. The strongest winds are recorded in February and March with the exception of storm and hurricane conditions.

Waves

Wave intensity of the Mississippi Sound is generally low to moderate. Fetch and depth limited waves within the sound average less than 1 ft in height. Breaking wave heights along the shoreline of the barrier islands average about 3 ft. However, hurricane and storm conditions, and strong winter cold fronts can produce significant surges and much larger wave conditions at the coast and barrier islands.

Tides

The tidal variation in the Mississippi Sound and adjacent waters is diurnal with an average tide cycle of 24.8 hours. Tides within the Sound range up to 2.5 ft with a mean tidal range of 1.77 ft. Although the tidal range caused by astronomical forces is relatively small, winds can induce larger variations. Strong winds blowing from the north can force water out of the sound and result in current velocities of several knots in the passes. The reverse occurs with winds blowing from the southeast, which forces water shoreward toward the Mississippi coastline.

Currents

The general circulation patterns in the Mississippi Sound are primarily induced by tides and winds, with freshwater inflows having secondary influences. The currents caused by the tide diverge and split the Mississippi Sound into two distinct areas. Horn Island Pass and the area north of the pass is the natural dividing point for tidal currents. Currents from this area to Lake Borgne generally flow into the Sound through the Barrier Island Passes and flow westward on the flood tide. During ebb tide, the flow is eastward and out of the Sound. from Horn Island Pass to Mobile Bay, currents flow in through the Barrier Island Passes and eastward on the flood tide, and reverse westward and out of the sound during ebb tide. Strong winds blowing from the north can force water out of the sound and result in current velocities of several knots in the passes. The reverse occurs with winds blowing from the southeast, which forces water shoreward toward the Mississippi coastline. Typical tidal currents range between 0.5 to 1.0 ft/s.

Sediment Transport

The Mississippi coast is a wave-dominated coastline. Because prevailing wind in the Mississippi barrier island and mainland areas is from the eastern quadrants, most waves approach the shoreline at an angle and induce longshore currents that move sediment to the west. The islands migrate west due to littoral drift at approximately 50 ft/yr. There are a variety of structures along the Mississippi mainland coastline with divide the shoreline into closed littoral cells. For annual average wave conditions, the beaches may shift due to specific storm event but remain largely in equilibrium. For higher wave conditions there appears to be a tendency for sand to bypass the structures. Small shoreline structures such as outfall pipes produce minor localized perturbations in the coastline with accretion on the east sides of the structures indicating a westward littoral drift, however, longshore processes have minimal influence on the beaches in comparison to the cross-shore processes that exert primary control on shoreline response. The Mississippi River and several rivers along the northern boarder direct silt and clay into the sound. Salinity-induced flocculation of these very find sediments induces settling and results in the continuous infilling of the sound. The high sediment load also produced elevated turbidity levels, giving the water of the Mississippi Sound their characteristically brownish appearance.

GEOLOGIC SETTING AND GENERAL GEOPHYSICAL INVESTIGATIONS

Geologic Setting

The coastal area of Mississippi is part of the Gulf Coastal Plain that extends from Florida westward to Texas. Coastal plains are generally characterized by gently sloping sedimentary formations that dip towards the coast line. The Gulf Coastal Plain is also affected by the Mississippi Embayment which is a trough that underlies the Mississippi River delta. This trough extends inward from the coast and is gradually subsiding near the coast under the sediment load that is being transported by the Mississippi River and deposited at the mouth of the river. Subsidence along this trough has changed the dip of formations that make up the coastal plain of Miocene and older age to a somewhat southwesterly direction. Of interest to this study are the three counties that front the Mississippi Sound. The Sound is a narrow, east-west; shallow body of water that separates the mainland from barrier islands that lie 10 to 15 miles offshore and the Gulf of Mexico southward of the islands. These counties, east to west, are Jackson, Harrison, and Hancock.

The Geologic Map of Mississippi (Moore, 1976), published by the Mississippi Geological Survey identifies three strata or formations that underlie the three subject counties. These include the alluvial/coastal deposits of Holocene age, the Citronelle formation of Pliocene/Pleistocene age, and the Pascagoula/Hattiesburg formation of Miocene age. Later and more detailed work (Otvos, 1986, 1992 and 2005) has further defined the various formations and provided information as to their depositional environment. This work also provides information concerning the barrier islands which lie off the coast of Mississippi. Some of this later work also addressed the presence of or lack of sand and other sediments along the coast, in the Mississippi Sound and near the barrier islands.

Within the Mississippi Sound, Holocene deposits form thin, muddy, strata that cover the older Pleistocene formations. These include alluvial, estuarine, and lagoonal-bay deposits. Sampling studies have shown the strata to contain particle sizes from colloidal to sand size depending on the energy associated with its depositional environment (Upshaw, Creath and Brooks, 1966).

Closer to the coast, late Pleistocene glacial action has caused a transgressive-regressive sequence that reworked sand along the coast. The last glacial period created a coastline near the edge of the continental shelf. As the ice began to melt, the associated sea level rise and wave action began to form the exposed sand into barrier islands. A predominant wave action from the southeast has created a westward littoral drift that replenishes the sand to the beaches and inlands as well as causing a westward drift to some of the islands. This has resulted in three formations that correlate from the alluvium along the coast to the barrier islands. These formations are the Prairie, Biloxi, and Gulfport formations. The Gulfport and Prairie formations are generally very sandy and have some economic value because of the sand. A generalized geologic map of the Mississippi coast based on these studies is shown in Figure 1, (after Otvos, 1997). The Prairie formation is found just landward of the coast in all three counties and the Gulfport formation is found along the beaches and barrier islands.

The Plio/Pleistocene Citronelle formation outcrops northward of the late Pleistocene formations. Utilizing outcrop, boring and fossil data from numerous locations, the Citronelle formation has been characterized as upland, alluvial/fluvial deposit that covers much of the study area. It consists predominantly of silt and sand with some gravelly deposits. The source of the sand came from rivers that drained to the Gulf coast. Where paleo-streams and rivers have been incised into the underlying

Miocene formation, Citronelle has formed thicker sequences than its general sedimentary deposits that cover much of the three counties.

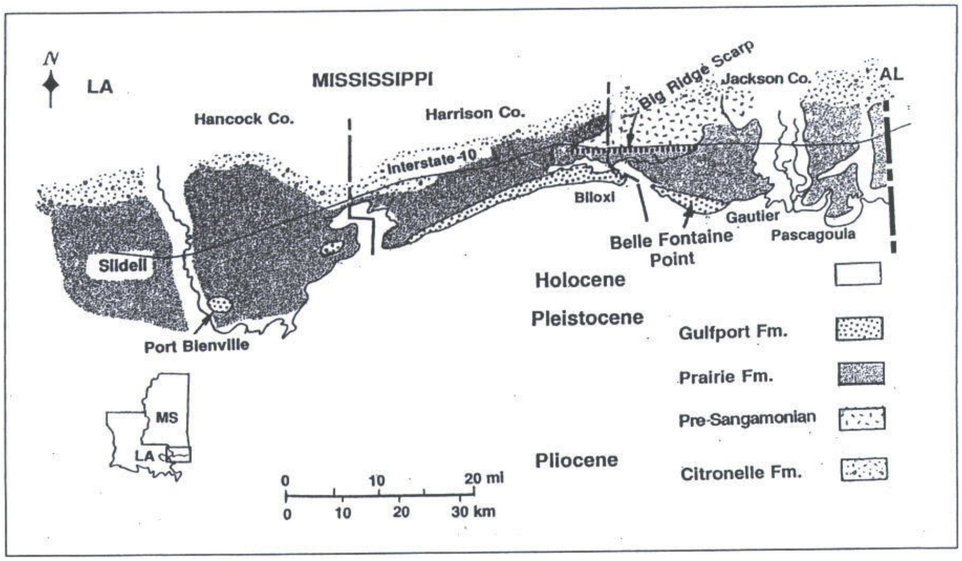

Figure 1. Generalized Geologic Map of Coastal Mississippi (After Otvos, 1997)

The northern portions of the three counties contain limited outcrops of the Miocene aged Pascagoula/Hattiesburg formation. This formation contains interbedded clay, silt, and sand and is exposed along river valleys that have incised through the younger Citronelle formation which overlies it in the study area.

Historical Off-shore Sampling and Geophysical Exploration

Starting in the 1950s, literature contains extensive information about the sediments and shallow strata in the Mississippi Sound and along the shoreline. These studies supported sediment studies, the construction of beaches in Harrison and Jackson County as well as investigations for proposed bridges out to the barrier islands. The Mississippi Office of Geology, Coastal Geology Section, within the Mississippi Department of Environmental Quality maintains extensive records of the borings and sampling that have occurred in the area of the Mississippi Sound, (http://geology.deq.state.ms.us/coastal). There is also an abundance of information available from the Gulf Coast Research Laboratory (Otvos, oral comm.) located in Ocean Springs, MS. Some of the past sampling events have been used to develop geologic sections such as shown in Figure 2 that was developed from borings taken between Gulfport and West Ship Island.

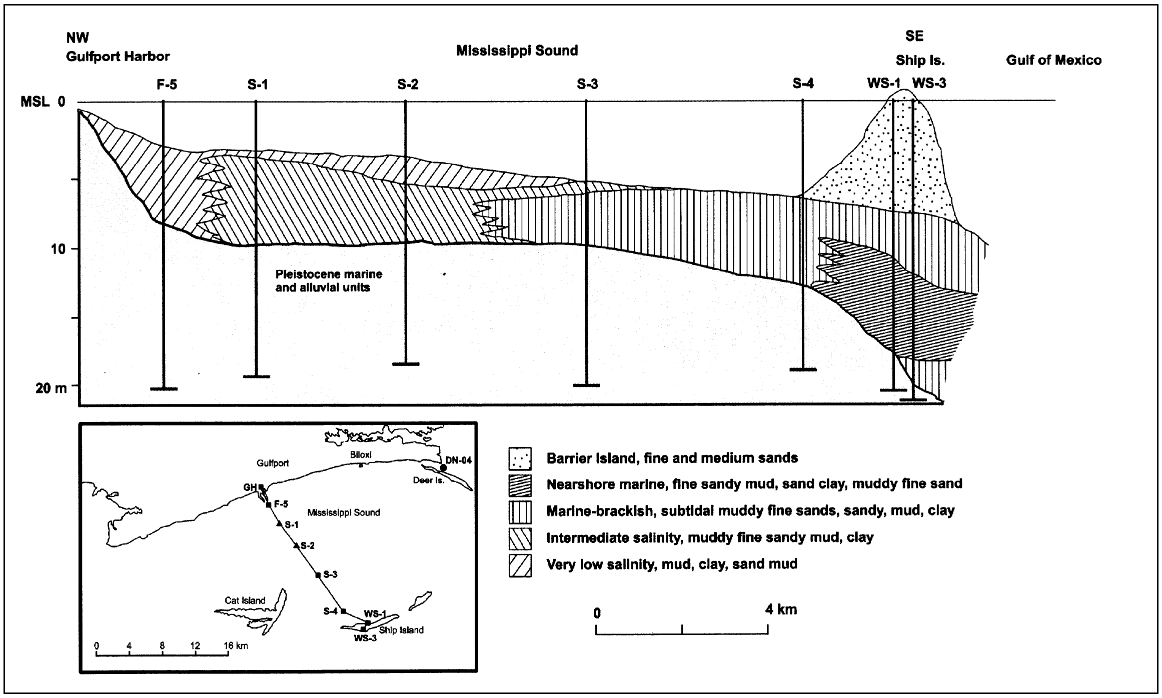

Figure 2. Generalized Geologic Section from Gulfport to West Ship Island (Otvos, 2004)

Proposed Off-shore Geophysical Exploration

To support the nourishment of sand along the mainland and on the barrier islands, extensive acoustic profiling is proposed in off-shore areas within Mississippi Sound and in some areas south of the barrier islands. Acoustic profiling is based on a source of acoustic energy that is generated and acoustic reflections from that noise that are collected after bouncing off firm subsurface strata. The method used to perform the survey consists of towing the energy source and hydrophones behind a boat along traverse lines. The speed of the signal is measured and digitally recorded after it passes through the upper, softer strata, is reflected off the firmer sub-bottom and returns to hydrophones which act as receivers. This measured speed has a correlation to different types and thicknesses of sediments. The exact location of the reflected signal is constantly recorded during the process using GPS technology. Using data from a grid pattern, an isopach or 3-dimentional interpretation will be completed to estimate the volumes of available sand. Areas to be surveyed were selected from prior investigations that indicated large, extractable deposits of sand. This was based both on prior acoustic profiling and sampling. To ensure the resolution is sufficient to allow for proper interpolation, the proposed grid pattern will have a spacing of 500 feet while paralleling the coast and 1000 feet while operating perpendicular to the coastline. The areas proposed for the geophysical survey are shown in Figure 3.

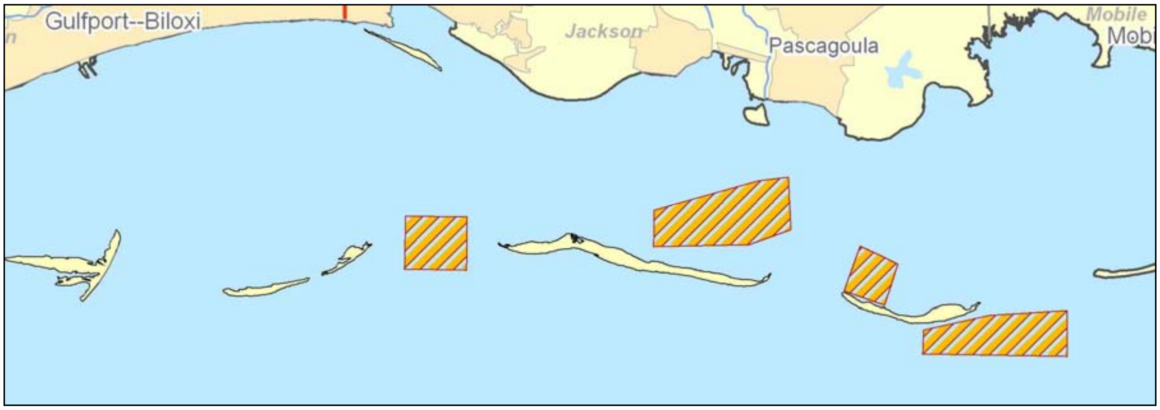

Figure 3. Proposed areas for geophysical surveys

In addition to the acoustic profiles, the bottom of the selected study areas will be surveyed with side-scan sonar. This procedure locates any abrupt change in the bottom contour that may indicate debris, shipwrecks, or even vegetation growing on the bottom. This will prevent damaging dredging equipment if debris is found within the zones selected for borrow areas or damaging vegetation that has high value to marine life.

During the geophysical survey, some locations will be selected to obtain actual samples of the sediments to provide accurate correlation between the interpretations and actual conditions. The contractor that performs the geophysical survey will obtain these samples during the operation. These samples will also provide for a general analysis of grain size distribution, particle shape, and color. All of these are important in selecting the borrow areas prior to placing the sand on beaches.

The results of the geophysical surveys will be used to estimate both location and quantities of the required sand. After the acoustic profiling is completed, the next phase will be a more complete exploration program that will verify the results of the geophysical survey. This phase will consist of taking numerous Vibracore samples which provide a continuous sample from the sound/gulf bottom to a depth of 20 feet. The spacing of these holes will be sufficient to ensure that the extracted sand meets all quality specifications from a given location.

Tectonic and Seismic Considerations

Numerous studies have been made concerning subsidence around the mouth of the Mississippi River. General thoughts have attributed the subsidence to the sediment loading of the lower delta as the river enters the Gulf of Mexico. Other studies have concluded that recent faulting has occurred associated with both subsidence along the coast and uplifting in the coastal plain (Bowen, 1990). While this low order faulting in soft sediments would produce no significant seismic events, associated displacements must be considered even if very small. Actual measured subsidence on first-order benchmarks has concluded that the Mississippi coast had a subsidence rate of 5 mm/year during the later half of the 20th century and continues to subside, (Shinkle and Dokka, 2004). This subsidence will have to be considered for any engineered solution along the coast.

Coastal Mississippi, On-shore

There are a large number of commercial sources for different types of soil along the three coastal counties of Mississippi. Depending on the project, these sources may be utilized for construction of levees, beach nourishment and dune restoration. Deposits of sand found in the Prairie formation may be of beach quality and have potential use for beach nourishment along the mainland beaches. The presence of the Prairie and Citronelle formations in much of the study area can provide necessary reserves for construction of levees. The sands included in these formations can also be evaluated for beach restoration. These sources are permitted by the Mississippi Department of Environmental Quality which publishes a list of permit holders. A review of the listed sources shows that Jackson County has 14 operations, Harrison County has the most with 63 sources and Hancock has 33 sources. These locations are shown in Figure 4. Not all the listed sources are believed to be active operations. At the present time, no information is available on specific soil properties such as classification, gradations or color, all of which will be important characteristics if used for beach nourishment. This information will be collected before any material is selected for use. Attempts will be made to contact each of the listed operators to compile a current list of sources that will provide an estimate of reserves, operational output, and more specific information on the material that is actually produced. A review of the permitted size (acreage) of most of the operations indicates that their reserves may be less than one million cubic yards. Many of the sources list specific information as to what type of material that they produce while some of the permits do not indicate the type of formation that is being mined other than a general statement such as “dirt”. A list of the permitted sources for Jackson, Harrison and Hancock Counties are shown in Table 1a, 1b, and 1c, respectively.

Table 1a.
Permitted Borrow Areas in Jackson County

County	Operator	Permit #	Permitted Acres	Material
Jackson	Bright	N/A	20	sand and clay
Jackson	Ward	P02-037	35	sandy clay
Jackson	Hence	P04-019	25	clay and sand
Jackson	Blain	P83-002	6	sand
Jackson	Yates	P-87-045T	29	sand and clay
Jackson	Jackson C	P91-061	10	sand and clay
Jackson	Mellette	P92-054	19	sand clay
Jackson	Talley	P93-020	24.8	dirt
Jackson	Graham	P93-029	20	sand and clay
Jackson	Dees	P94-036	6	dirt
Jackson	Dees	P95-058	16	dirt
Jackson	Jackson C	P96-014	19.5	soil clay fill
Jackson	Mellette K	P98-057	30	clay & sand
Jackson	Ward	P98-063	60	sandy clay

Table 1b.
Permitted Borrow Areas in Harrison County

County	Operator	Permit #	Permitted Acres	Material
Harrison	Waits	N/A	40	fill dirt
Harrison	Fore	N/A	40	
Harrison	Blacker	N/A	49.6	soil
Harrison	Dirt works	P00-020	9.7	sand
Harrison	Anchor	P00-065	20	fill dirt
Harrison	Dirt works	P01-014A	21.98	dirt/clay
Harrison	Williams D	P02-004	25.6	dirt
Harrison	Edwards	P02-007	12.7	dirt, sand and gravel
Harrison	Wallace T	P02-018	53	dirt
Harrison	Wallace T	P02-045	40	dirt
Harrison	fore	P03-010	38.2	dirt and sand
Harrison	Edwards	P03-044	7	sand, gravel and dirt
Harrison	TCB	P03-046	20	clay/sand
Harrison	Lamely D	P04-006A	25	clay, sand
Harrison	Edwards	P04-017AA	22.5	sand and dirt
Harrison	Du Pont	P04-036	38	clay
Harrison	Wetzel	P04-37	5.6	sand
Harrison	Fore	P04-043A	46.17	sand
Harrison	Fore_W. C.LLC	P05-005	40.02	sand
Harrison	Fore_W. C.LLC	P05-006	40.4	sand
Harrison	Saunders	P05-007	14.2	clay,sand
Harrison	Fore_W. C.LLC	P05-010	44.23	sand
Harrison	Warren Paving	P05-025	14.5	dirt
Harrison	Dirt	P06-002	15	dirt
Harrison	Cams	P80-022	20	fill dirt
Harrison	Griffin	P81-030T	8	fill dirt
Harrison	Fore	P87-027	28	sand and clay
Harrison	Blackmer	P87-029T	8	clay/sand
Harrison	Dirtworks	P87-048T	5	fill dirt
Harrison	Mid C	P88-012	20	fill material
Harrison	Gulf	P88-025T	12	sand and gravel
Harrison	Fore	P88-027	30	sand and clay
Harrison	Fore	P88-027A	76	sand and clay
Harrison	Parker	P89-007	5	fill dirt
Harrison	Cams	P89-019	10	sand clay
Harrison	Lamey D	P89-022	5	fill dirt
Harrison	Ladner	P90-023	6.5	sand and gravel
Harrison	TCB	P90-024T	4	sand and gravel
Harrison	Ray	P92-014	10	soil/borrow
Harrison	Parker	P92-066	3	dirt
Harrison	Holden	P92-079T1	4.5	dirt
Harrison	Blackmer	P92-089	12	clay/sand fill
Harrison	Twin	P92-093	10	clay/sand fill

Table 1b.
Permitted Borrow Areas in Harrison County (continued)

County	Operator	Permit #	Permitted Acres	Material
Harrison	Ladner	P93-009	6	sand and gravel
Harrison	Holden	P93-012	8	sand and clay
Harrison	Holden	P93-041	19.4	sand-clay
Harrison	Lamey D	P93-051	10	fill dirt
Harrison	Breeland	P93-064T	32	fill dirt
Harrison	Dubuison	P93-113	0.7	sand clay
Harrison	Newells	P94-035	11.5	clay sand gravel
Harrison	Holden	P94-064T1	4	fill material
Harrison	Blackmer	P95-018	28	sandy clay
Harrison	Holden	P95-073	20	clay, sand-clay
Harrison	Dirtworks	P95-080T	7	fill dirt
Harrison	Fore P	P95-082	3	sand and gravel
Harrison	Fore P	P95-083	3	sand and gravel
Harrison	Holden	P96-022T1	8	dirt
Harrison	Fore C	P96-047	30	sand and clay
Harrison	Parker	P96-067	3	dirt
Harrison	Holden	P97-021	15	clay and sand clay
Harrison	Twin	P98-048	35	sand and gravel
Harrison	Prince	P98-055	10	sand and clay
Harrison	Wallace T	P99-052T	22	sand clay

Table 1c.
Permitted Borrow Areas in Hancock County

County	Operator	Permit #	Permitted Acres	Material
Hancock	Gibson	P00-034	4	fill dirt
Hancock	Boudin	P00-058	10	sand/clay/fill
Hancock	Phillips Tru	P02-016	40	sand and clay
Hancock	Fore	P02-027	37.25	dirt and sand
Hancock	Cuevas	P02-058	4	clay gravel
Hancock	B&C	P03-011A	12	dirt and sand
Hancock	Henley C	P03-028	8.75	clay and sand
Hancock	DK Agg	P04-007	40	sand and gravel
Hancock	DK Agg	P04-008	20	dirt/clay
Hancock	Frierson	P04-012	6	sand and clay
Hancock	Larry Nicks	P05-001	12	sandy clay
Hancock	Phillips Tru	P05-003	25	sand and dirt
Hancock	Knight	P86-016	1	sand and gravel
Hancock	Fore	P92-024	20	borrow/soil
Hancock	TCB	P93-022	25	sand clay
Hancock	SCI	P93-033	13.1	borrow
Hancock	Fore	P93-048	29	fill dirt

Table 1c.
Permitted Borrow Areas in Hancock County (continued)

County	Operator	Permit #	Permitted Acres	Material
Hancock	Fore	P93-048	N/A	fill dirt
Hancock	Ladner P	P93-079	15	sand and clay
Hancock	Haas	P93-110	16.3	sandy clay
Hancock	Frierson	P95-012	4	dirt
Hancock	Fore	P95-047T	10	sand and sandy clay
Hancock	Henley C	P96-008	3.7	clay/sand
Hancock	C & G	P96-064	5	dirt/sand
Hancock	Ladner R	P97-023	3	fill dirt
Hancock	Pittman	P-97-032	46	sand and clay
Hancock	Fricke's	P97-044	6	sand and sandy clay
Hancock	Fore S	P-97-045T	20	sand and gravel
Hancock	Thigpen	P98-017	9	sand and gravel
Hancock	Fore	P98-064T	10	sand/clay/fill
Hancock	Fricke's	P98-065	8.7	sand, sandy clay
Hancock	Moran	P99-021	31.5	fill dirt
Hancock	Thigpen	P99-034	14	sand and gravel

Some projects along the coast are already under design and will require sand for both compacted backfill and for beaches. These projects are located in all three coastal counties and the in-place quantities are as follows:

- Jackson County Pascagoula Beach 270,000 cubic yards sand
- Harrison County Beach 681,000 cubic yards sand
- Hancock County Bay St, Louis Seawall 159,000 cubic yards sand

All of these projects are limited in scope and could be easily supported by local on-shore commercial operations or sand deposits that have located just offshore. These near-shore sand deposits are limited in size and may be due to past beach construction and nourishment projects where the sand was eroded from the beach due to storms and wave action.

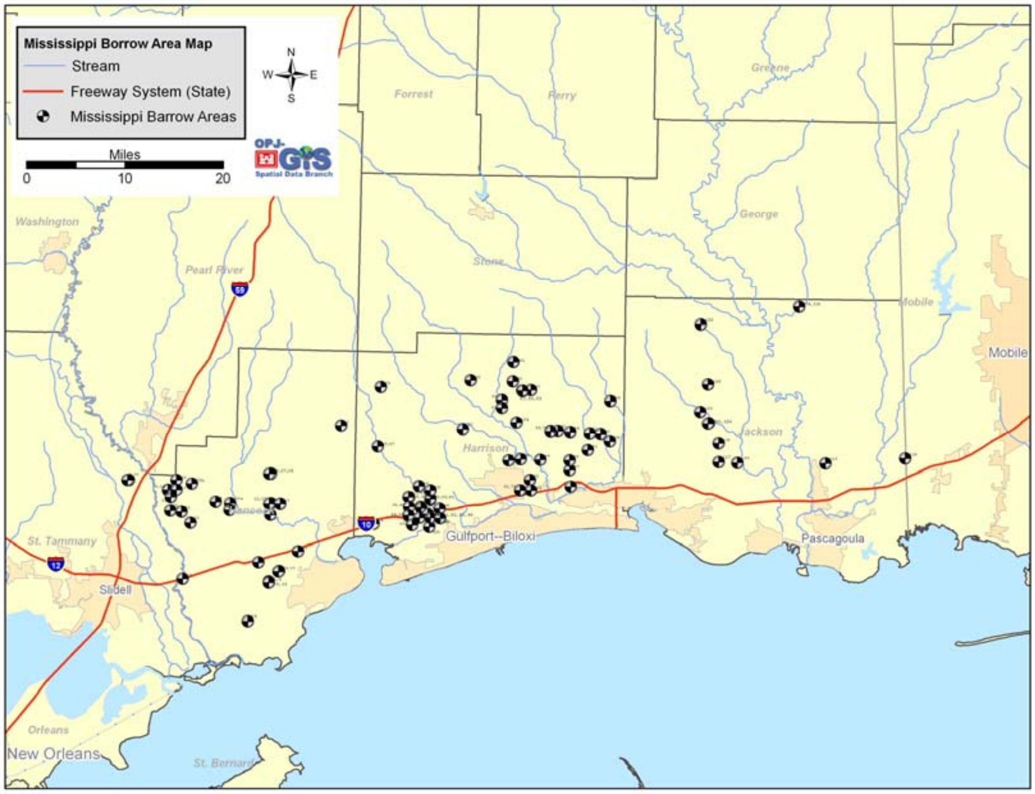

Figure 4. Location of Permitted Mining (Borrow) Operations

Coastal Mississippi – Offshore

To provide the sand necessary to rebuild or nourish the beaches on the barrier islands, large quantities of quality sand must be located. The inventory of these sand resources has been the subject of many studies. One proposed long range goal of this project is restoring the barrier islands of the coast of Mississippi to a general pre-hurricane Camille footprint. This will involve establishing islands of a size similar to a pre-Camille condition with allowances made for migration of the islands over time. While Petit Bois, Horn, and Cat Islands were not subject to the extreme erosion that has breached Ship Island during Hurricane Katrina, all have seen some loss of land mass. Information provided in a report on Hurricane Camille (Corps of Engineers, 1969), described the pre-Camille land mass of each of these islands which can be compared to the land mass post-Hurricane Katrina as shown in Table 2.

Table 2.
Loss of Barrier Island Land Mass

Island	Pre-Camille acres	Post-Katrina acres	Area Lost
Petit Bois	1,329	1,098	231
Horn	3,612	3,077	535
Ship	1,172	631	541
Cat	2,344	1,957	387

At the present time, four areas have been selected for acoustic profiling. An initial quantity of 50,000,000 cubic yards of sand has been estimated for use on the barrier islands and is the target for our survey. This includes an estimated 30 percent loss of volume during placement due to the losing finer sand particles in the outwash. All of these areas may be contained within the littoral drift zone that transports sand along the chain of barrier islands. The impacts of transferring this sand within the littoral drift zone will be evaluated through sediment transport models. Some of these areas also are within the boundaries of the Gulf Islands National Seashore which extend one mile from the shores of Petit Bois, Horn, and Ship Island. Other than close to the mainland and island beaches, most areas within the Sound are expected to have muddy Holocene deposits overlying any sand deposits. These deposits may render the sand unusable without segregation of the different materials prior to being placed along the beaches.

During hurricane Katrina, the channel that cut through Ship Island was widened into a breach approximately three to four miles wide. This also occurred during Hurricanes Fredrick and Camille with a low island reforming over time. This erosion and other lesser amounts of erosion on the other islands has scattered sand on an area of unknown extent. Much of this sand may still remain in the littoral drift zone. It may eventually be transported where it could be naturally deposited on a beach. However, this process is slow and will not aid in storm protection for a very long period of time. Identification of these sand deposits and using them to restore the island would provide a more timely protection for the coast during lower intensity storms.

If completed, the restoration of Ship Island will be the largest single project requiring up to 34,000,000 cubic yards of excavated sand. This volume is roughly based on restoring the breach to an island width of 2,000 feet (including submerged portion) for the full length of the breach (4 miles) and bringing the sand to at least 20 feet above sea level with a 10 foot existing water depth. This height will allow better protection against breaching during future low intensity storms (Otvos, oral comm. 2006).

Based on previous work (Otvos, 1975/76 and Upshaw, Creath, and Brooks, 1966) which involved sampling and sub-bottom profiling, four areas have been selected for exploration using acoustic profiling and vibracore sampling. This procedure has been previously described in Proposed Off-shore Geophysical Exploration and the proposed areas are shown in Figure 3. Three of the areas are located either partly or wholly within the boundaries of the Gulf Islands National Seashore and any work within these boundaries must be coordinated with the National Park Service. These boundaries include Petit Bois, Horn and Ship Islands. Petit Bois and Horn Islands are also designated as Wilderness Areas by the Park Service and receive a higher level of protection than Ship Island.

Review of the samples that were collected during these and other studies also indicate that sand deposits underlie some of the Holocene deposits within the Mississippi Sound. The use of these sands for beach nourishment would be dependant on segregation and removal of the overlying muddy Holocene sediments. The Holocene sediments may have some value for use in the creation of marshes and wetlands that could be considered if the underlying sands were needed to complete a project. An example of this condition exists about two miles south of Deer Island. In a boring referenced as Hole 785 and reported by Otvos (1985), the bottom of the Sound was recorded at 9.0 feet. From 9.0 to 13.3 feet the sample was described as muddy medium sands, poorly sorted. Underlying this muddy sand, the samples showed medium sand from 13.3 to 16.7 feet and very to well/moderately sorted, fine sand from 16.7 to 27.1 feet.

As one might expect, much of the quality sand deposits are within the littoral drift zone of the barrier island chain. This high energy environment provides a sorting process that allows for deposition of sand while preventing finer grained sediments from being deposited. While not removing the sand from the littoral drift zone, the process of relocating of sand from any given area within the drift zone and transporting it to another area within the zone must be considered. Using the same reference as above (Otvos, 1985), a boring taken within the littoral drift zone between Horn and Ship Inland, Boring S-6, the upper eleven feet of sediment to be well to moderately well sorted medium sand with additional sand units below.

Inland River System

After the construction of inland waterways in Alabama and Mississippi, maintenance dredging is sometimes required to maintain the channel depths and alignments. This material is typically moved to disposal areas along the banks of the river where it accumulates in diked areas. Dredging of some of the areas along the river produces large quantities of sand that have potential use for beach nourishment. An inventory of current disposal sites indicates that approximately 30,000,000 cubic yards of sand may be available. Only disposal sites that contain a minimum of 100,000 cubic yards of sand were included in the inventory. Of interest to this study are disposal sites that are located along the Black Warrior – Tombigbee River system and the Tennessee – Tombigbee Waterway. Figure 5 shows the relationship of these disposal areas to the project sites along the Mississippi coast. Material from these sites could easily be transported by barge down the river system for use along the beaches.

Because of the shortage of additional disposal areas, the Corps of Engineers' Operations Division has contracted for several studies on the beneficial use of the sand. Some of these studies have been targeted at using the sand for beach nourishment, (Thompson Engineering, 2001). Using sand samples from some of the inland disposal areas along the Black Warrior – Tombigbee River, a series of analyses were conducted on the samples. For comparison purposes, several samples of actual beach sand and from the littoral drift zone from coastal Alabama were taken and subjected to the same tests. These tests included grain size distribution (gradation), color and roundness. The results of the tests indicated that some of the samples may be suitable for beach nourishment. The sand from the river was typically a finer grain size that the beach sand with the predominant river size being a fine sand while the beach sand was mostly medium sand. It was also noted that the beach sand was slightly more rounded than the river sand.

One factor that warranted further analysis was the color difference of the river sand as compared to the beach sand. All of the river sand had a brown tint described as "very pale brown" or "light yellow brown". This compared to the beach sand samples which were described as "pale olive, white or light grey". These colors were assigned along with evaluations for hue, value and chroma from a Munsell Soil Color chart which provides a standard method of assigning color to soils. The report also noted that beach sand came from a higher energy environment where any staining due the depositional environment may have been removed by abrasion due to wave action. It also noted that the sand might undergo bleaching from the ultraviolet radiation from the sun if the color was caused by a mineral staining. To test these conditions that may change the color of the sand, a series of tests were conducted on samples from the same areas that were used during the initial analyses, (Thompson, 2002). The samples were subjected to two tests. The first involved actual bleaching of the samples using a chemical oxidizer, hydrogen peroxide, for different periods of time. These tests did indicate that the bleaching process was detectable after 72 hours. Other tests were conducted to simulate the process of wave action causing an agitation of the particles which may remove any mineral coating or staining along with exposure to ultraviolet light. This process was conducted for 144 hours without a notable difference in color.

Other studies on the dredge disposal areas by the Bureau of Mines, U.S. Department of the Interior were conducted to characterize the sand for use as an aggregate in making concrete (Smith, 1995). While these tests were not directed at use of the sand for beach nourishment, they did supply information on chemical and physical characteristics of the materials from several locations. These tests provided data that shows the sand to be clean, mostly fine grained, quartz sand with little of no fines, to be non-toxic based on Toxic Characteristic Leachate Procedure (TCLP) and to contain very little heavy minerals. All of these tests would indicate the material would be safe to place on a beach.

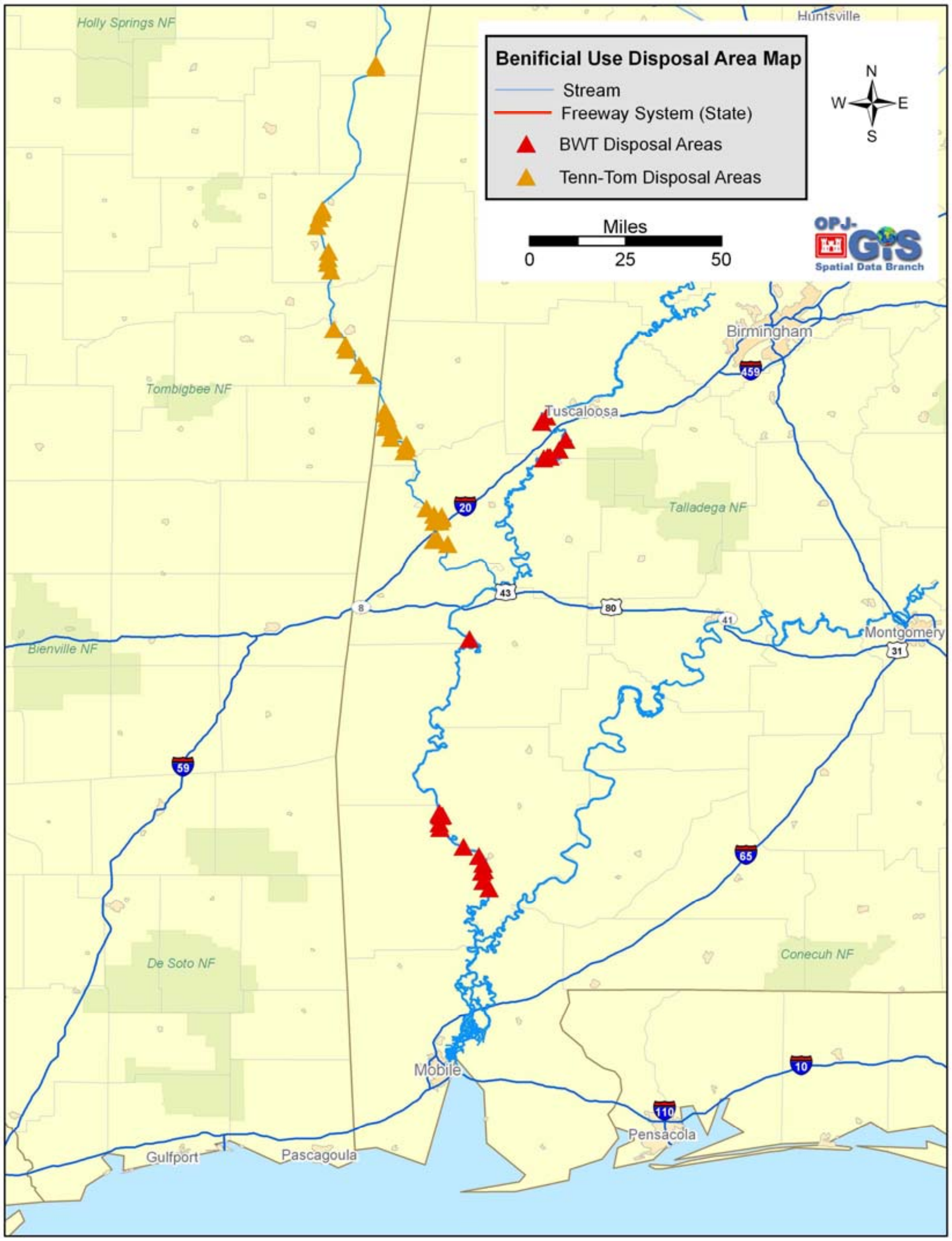

Figure 5. Inland disposal areas that contain economic deposits of sand

Table 3a.
BWT Dredge Material Disposal Areas Over 100,000 CY

Site	River Mile	Acquisition	Access/ Land	Access/ River	Est Material Placed To Date (CY)
C	78.2	Easement	No	Yes	1,500,000
D-1	82	Easement	No	Yes	515,000
E	86	Easement	No	Yes	250,000
E-2	87	Fee	No	Yes	110,000
F	88.5	Easement	No	Yes	315,000
I	91.5	Easement	Yes	Yes	260,000
J	96	Easement	No	Yes	140,000
N	103.5	Easement	No	Yes	1,400,000
R	105	Fee	No	Yes	130,000
X-2	108	Fee	No	Yes	205,000
X	108.2	Easement	No	Yes	1,500,000
X-4	108.4	Fee	No	Yes	810,000
Z	108.6	Easement	No	Yes	1,250,000
CA-1	191.3	Easement	Yes	Yes	135,000
BA	297	Easement	No	Yes	300,000
AD	299.2	Easement	No	Yes	440,000
AE	300.4	Easement	No	Yes	465,000
AF	307	Easement	No	Yes	1,600,000
AG	313	Easement	No	Yes	1,020,000
BE	324	Easement	Yes	Yes	160,000
BD	329	Easement	No	Yes	170,000
TOTAL					**12,675,000**

Table 3b.
TTW Dredge Material Disposal Areas Over 100,000 CY

Site	River Mile	Acquisition	Access/ Land	Access/ River	Est Material Placed To Date (CY)
D-20	243.5	Easement	Yes	Yes	721985
D-24	249.5	Easement	Yes	Yes	196392
D-25	250.6	Easement	No	Yes	257137
D-29	256.5	Easement	Yes	Yes	127014
D-30A	257.3	Easement	Yes	Yes	750654
D-30B	257.7	Easement	Yes	Yes	195291
D-31A	259.3	Easement	Yes	Yes	298684
D-31B	260.3	Easement	Yes	Yes	231121
D-33	263.1	Easement	No	Yes	1825225
D-36	265.4	Easement	Yes	Yes	900317
G-13	287.8	Easement	No	Yes	242129
G-14	289.4	Easement	Yes	Yes	622745
G-15	290.5	Easement	No	Yes	710754
G-18	295.4	Easement	Yes	Yes	249803
G-20A	297.6	Fee	No	Yes	209650
G-21	299.8	Fee	No	Yes	1653977
G-22	301.8	Easement	No	Yes	116938
G-24	303.6	Easement	No	Yes	244175
G-25A	304.8	Easement	Yes	Yes	694172
G-26	305.7	Easement	Yes	Yes	295961
AL-7	317.3	Easement	Yes	Yes	109131
AL-9	320.4	Easement	No	Yes	334863
AL-13	326.4	Easement	Yes	Yes	1274697
AL-14	328.2	Easement	Yes	Yes	271563
AL-16	333.6	Easement	Yes	Yes	130691
C-14	350	Easement	Yes	Yes	575875
C-18	352.1	Easement	No	Yes	140864
C-19	353.3	Easement	Yes	Yes	1049792
C-20B	355	Easement	Yes	Yes	148024
AB-6	362.3	Easement	No	Yes	270663
AB-9	364.3	Easement	Yes	Yes	116522
AB-12	365.9	Easement	Yes	Yes	3171722
AB-13	366.5	Easement	Yes	Yes	448743
PE-3	410.2	Easement	No	Yes	195636
PE-4	411.1	Easement	No	Yes	122290
TOTAL					**18,905,200**

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SUBSIDENCE OF THE MISSISSIPPI GULF COAST

In 2004, the National Oceanic and Atmospheric Administration (NOAA) published the results of an investigation by its National Geodetic Survey (NGS) showing that the Mississippi Gulf Coast was subsiding at a rate of about 5 mm/yr during the later half of the 20th century.[1] This study used a rate of subsidence determined at a long-term NOAA water level gauge at Grand Isle, Louisiana, as a starting point. Figure 1 shows the water level trend at Grand Isle. The rate of subsidence is the water level trend minus the value for global eustatic sea level rise. Rates of vertical displacement at benchmarks along first-order leveling lines were then computed from the changes in the observed height differences over the time span between subsequent leveling projects. Figure 2 shows the computed rates of subsidence for the first-order benchmarks along the U.S. 90 / CSX railroad corridor between the Pearl River (the border between Mississippi and Louisiana) and Mobile, Alabama.

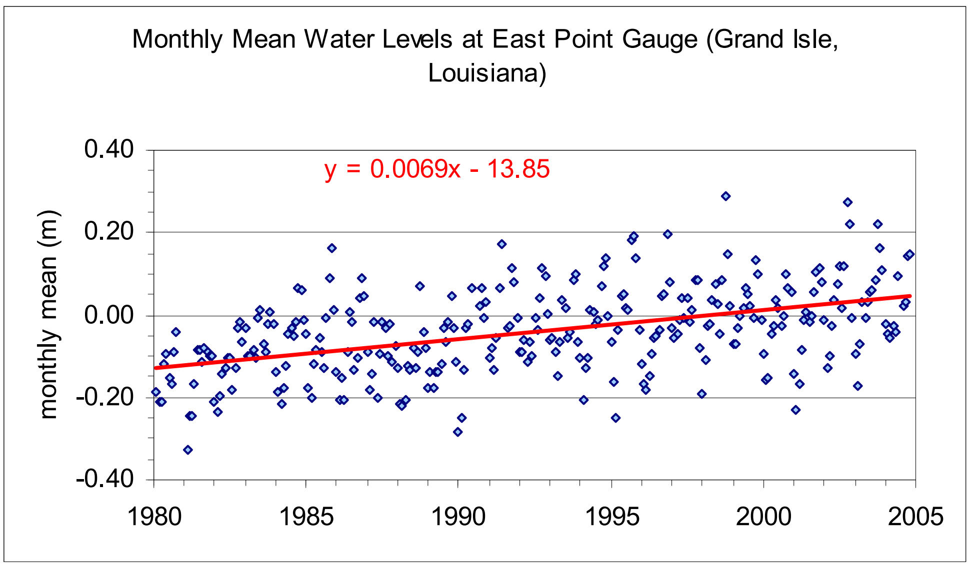


Figure 1. Sea Level Trend at the NOAA Gauge at Grand Isle, Louisiana.

The rates shown in figure 2 were computed for the specific epoch between two leveling projects, i.e., 1955 to 1969 east of Biloxi, and 1955 to 1977 west of Biloxi. These data are the latest observations available over most of this segment of the first-order leveling network. Subsidence rates developed for other segments of the leveling network within this region used data observed as late as 1996. The rates of displacement computed for segments with more recent observations indicate that, while rates appear to vary in a non-linear manner over relatively short time spans, there is no reason to think that subsidence in the region has ceased. The small segment of data available in this part of the network for the 1977 to 1993 implies that rates of subsidence along the Mississippi Gulf Coast continued on generally unchanged into that later epoch. The conclusion is that subsidence of the Mississippi Coast is a real phenomenon that is continuing.

[1] K.D. Shinkle and R.K. Dokka, *Rates of Vertical Displacement at Benchmarks in the Lower Mississippi Valley and the Northern Gulf Coast* (Silver Spring: U.S. Department of Commerce), 2004.

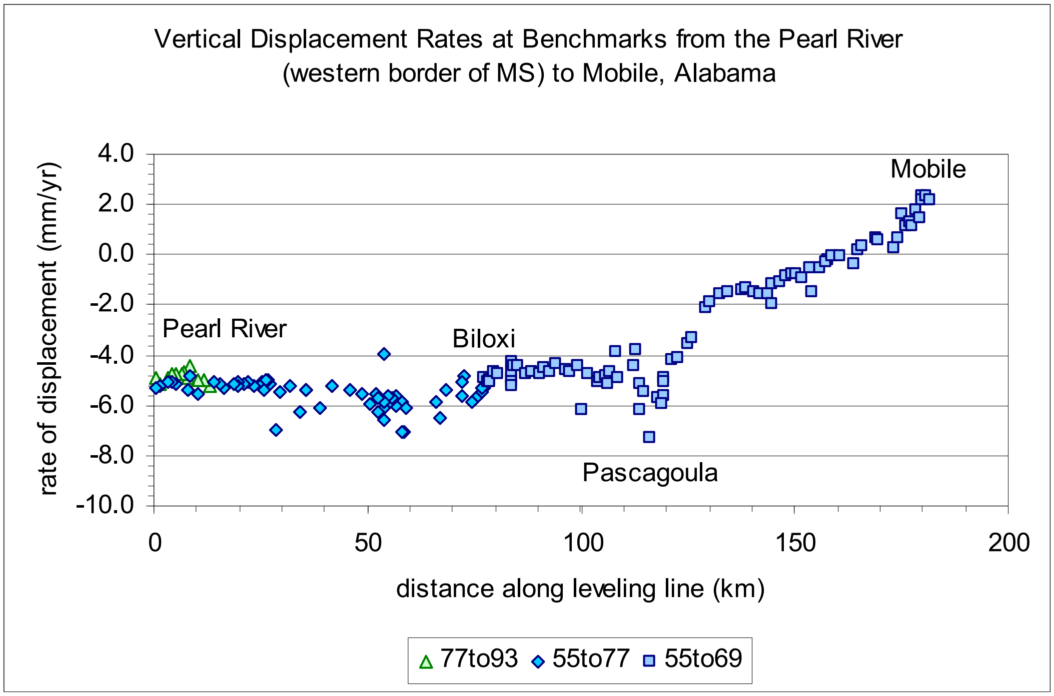


Figure 2. Rates of Vertical Displacement at Benchmarks. These are Preliminary Computed Values; Values Adjusted to the Tide Gauge Control at Pensacola, with Uncertainty Estimates, Have Not Yet Been Reviewed.

Figure 3 shows the rates of subsidence translated into total estimated vertical displacement of each benchmark, for which rates were computed, since 1955.

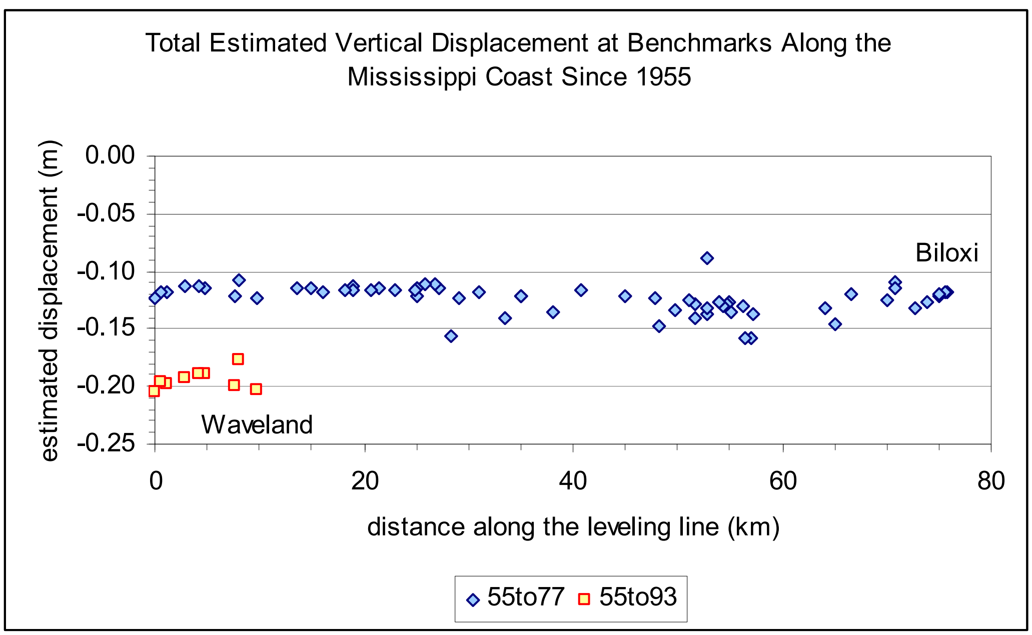
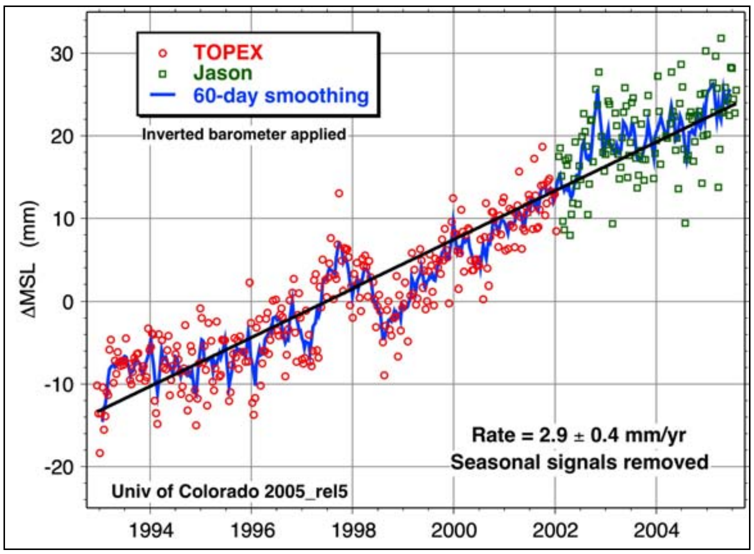


Figure 3. Estimated Total Vertical Displacement.

GLOBAL SEA LEVEL CHANGE

Global mean sea level has been rising over the past century, although estimates of the specific rate of that rise vary. The Third Assessment Report prepared by the International Panel on Climate Change[2] reports a value of 1 to 2 mm/yr for sea level rise during the 20th century. Reviews of published studies show values for the rate global sea level rise that range from 1.0±0.15mm/yr to 2.4±0.9 mm/yr.[3] One recent study examined previous work on both coastal and global average sea level rise and concluded that the best estimate for both "remains 1.8±0.3 mm/yr."[4] Most of the studies of long term change from the 20th century were based on tide gauge observations. Studies based on satellite altimetry cover only about the past decade. These studies seem to indicate a higher rate of global sea level change, e.g., 2.9±0.4 mm/yr.[5] In any case, estimates of global or coastal sea level rise are independent of any effects due to local land surface subsidence.



Sea Level Rise Trend Computed from Satellite Altimetry Data (Image From Http://Sealevel.Colorado.Edu nd Leuliette, Et.Al.)

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[2] J.T. Houghton, et al, eds., *Climate Change 2001: The Scientific Basis* (Cambridge, Cambridge University Press), 2001.

[3] B.C. Douglas, MS. Kearney, and S.P. Leatherman, *Sea Level Rise, History and Consequences* (San Diego, Academic Press), 2001.

[4] N.J.White, et al.; Coastal and global averaged sea level rise for 1950 to 2000; Geophysical Research Letters, 32, L01601, 2005.

[5] E.W. Leuliette, R.S. Nerem, and G.T. Mitchum; Calibration of TOPEX/Poseidon and Jason altimeter data to construct a continuous record of mean sea level change; Marine Geodesy, 27(1-2), 2004; 79-94.

ENGINEERING APPENDIX

II. LONG-TERM ENGINEERING SOLUTIONS

LONG-TERM ENGINEERING SOLUTIONS

The primary difference between the Long-Term Engineering solutions shown in Section II, and the Interim Engineering solutions presented in Section III is scale and level of complexity. The alternatives presented in Section II will generally focus on the entire of Coastal Mississippi, be large and complex in nature, and will likely include innovate technologies. The engineering analysis accomplished for these long-term solutions will be extremely complex and in-depth and will include extensive modeling, independent technical review within the Corps, and extensive peer review from outside of the Corps. The engineering analysis presented for the Interim Engineering Solutions in Section III will generally be much more limited in scope, generally focusing on a discrete portion of the Mississippi Coast with limited design goals.

The Engineering analysis must show the most cost-effective alternative to provide the stated goal. Example: The stated project goal is to provide erosion protection for a 2000-foot section of roadway. The engineering analysis would show protection using vinyl sheet pile, riprap, and a timber bulkhead. The alternative that provides the most cost-effective life-cycle cost would be carried to completion and a fully-funded cost-estimate developed.

In addition to being cost-effective, that each recommended alternative must be safe, efficient, and reliable.

Safe: Minimize potential hazards to humans and property. Identify consequences of storm intensities exceeding the design parameters.

Efficient: Structure cross section, materials, and plan configuration selected to optimize the probability of achieving the degree of protection based on estimated life-cycle costs and project goals.

Reliable: Probability or certainty in the ability to achieve project purposes throughout the project evaluation period and proper functioning of features such as beach nourishment, breakwaters, seawalls, and groins. Periodic renourishment cost should be expressed as the likely minimum, maximum, and expected annual cost at an acceptable level of confidence.

Lines of Defense

The comprehensive Mississippi coast long term solutions will evaluate ranges of natural and engineered measures along five potential lines of defense which will be designed to provide various levels of protection for the Mississippi mainland coast. The strategy is to develop the lowest level of protection along the offshore barrier islands, with the level of protection increasing with distance from the barrier islands toward the mainland shoreline and inland areas. The limit of storm surge inundation resulting from Hurricane Katrina impacting the Mississippi mainland coast is shown in Figure 1. A conceptual plan of the five lines of defense showing the level of protection increasing from the offshore barrier islands to inland coastal Mississippi is depicted in Figure 2.

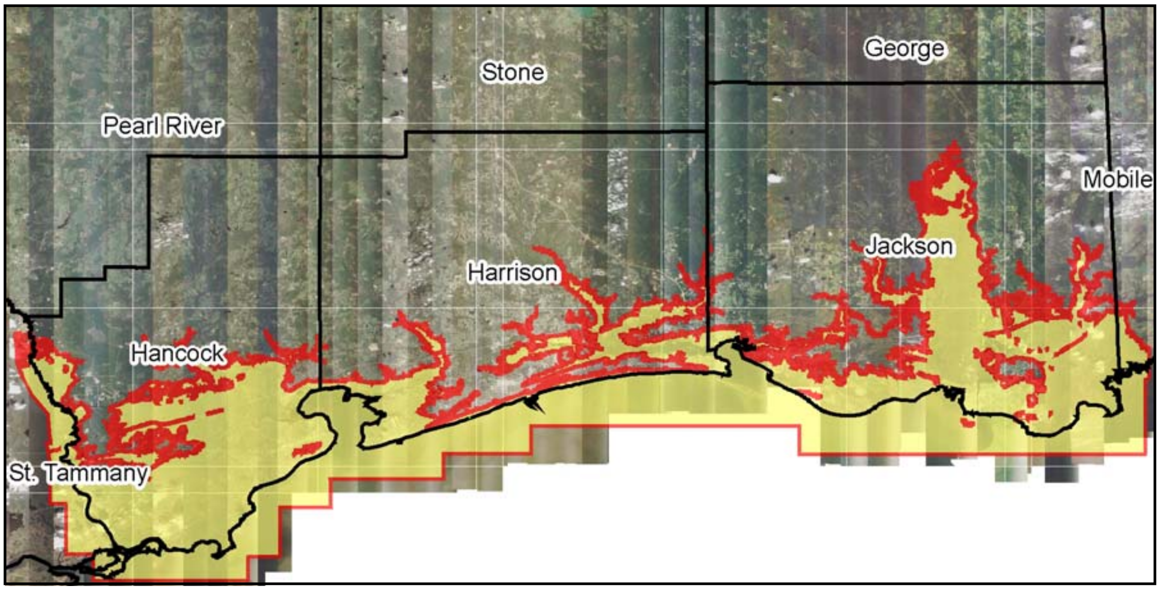

Figure 1. Hurricane Katrina Storm Surge Inundation Limits

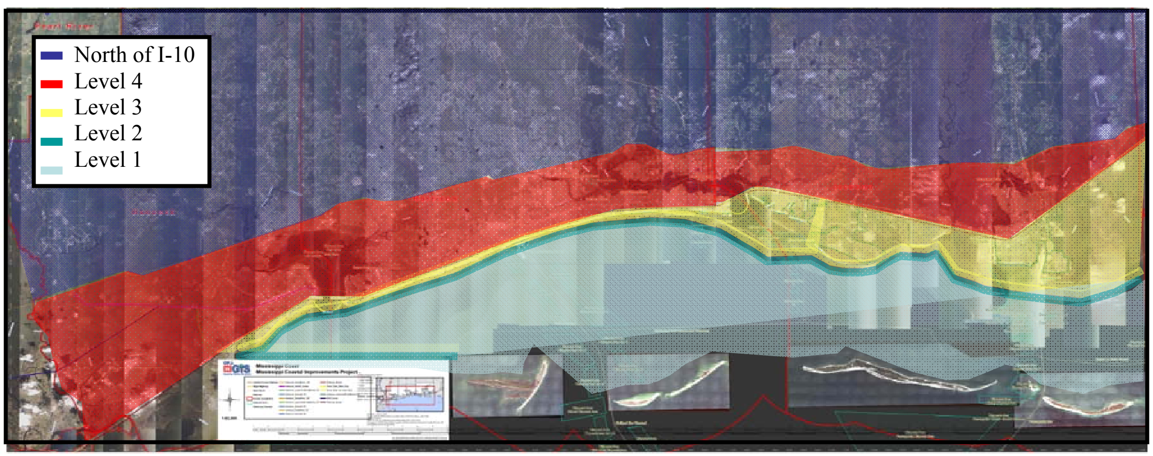

Figure 2. Conceptual Plan of Lines of Defense and Levels of Protection

The following describes the five conceptual lines of defense with varying levels of protection have been developed for further evaluation in this study. A cross section, layout depicting the five lines are defense over the Mississippi coast, and conceptual plan for Hancock County are provided in Figures 3, 4, and 5, respectively.

Defense Line 1: Offshore – evaluate restoration of the Mississippi barrier islands to reduce wave and surge along the Mississippi mainland shoreline. Restore the Mississippi Barrier Islands to pre-storm configurations and +20 ft elevation. Potential sand sources are from adjacent borrow areas in the Mississippi Sound and from upland disposal sites on inland river systems.

Defense Line 2: Beachfront – evaluate improvements in the nearshore zone and adjacent inland; alternatives include creation or restoration of berm and dune systems and seawalls, and other potential methods of protection including landward barrier, raising of roads, etc, Figures 6, 7, and 8. Berm elevations will be designed at the existing elevations (+5 ft Harrison county), dune elevations will be evaluated ranging from 10 to 15 ft. A seawall will be evaluated at elevations ranging from +5 to +15 ft. To prevent flood/surge from entering the Pearl River, St Louis Bay, Biloxi Bay, Pascagoula River, and Middle River which would result in inland inundation, flood/surge gates will be evaluated as potential additions to the lines of defense. The gates would be closed during hurricane and storm conditions with potential for loss of life and damages due to inland inundation, otherwise the gates would remain open providing access and allowing the natural flow exchange between the water bodies and the Sound. Conceptual examples of flood/surge gates are shown in Figures 11a and b.

Defense Line 3: Near Beach – evaluate the benefits of raising the first floor elevations of structures and raising roads to approximately a +22 ft elevation. Install levee/landward barrier systems where appropriate. Flood/surge gates to prevent surge from entering bays and rivers.

Defense Line 4: Railroad Corridor – evaluate improvements in a corridor parallel and adjacent to the existing railroad, figure 8 and 9; alternatives to include levee, and/or landward barrier, and/or highway embankments at elevations (20, 25, 30, 35 ft and the PMH). Storm surge or flood gates located at the three developed bay areas. Flood/surge gates to prevent surge from entering bays and rivers.

Defense Line 5: Interstate-10 Corridor – Relocate critical infrastructure (hospitals and medical facilities, fire stations, emergency management offices, etc north of the Interstate-10 corridor. Provide gates at an elevation of 20 ft at each road penetration through the I-10 embankments.

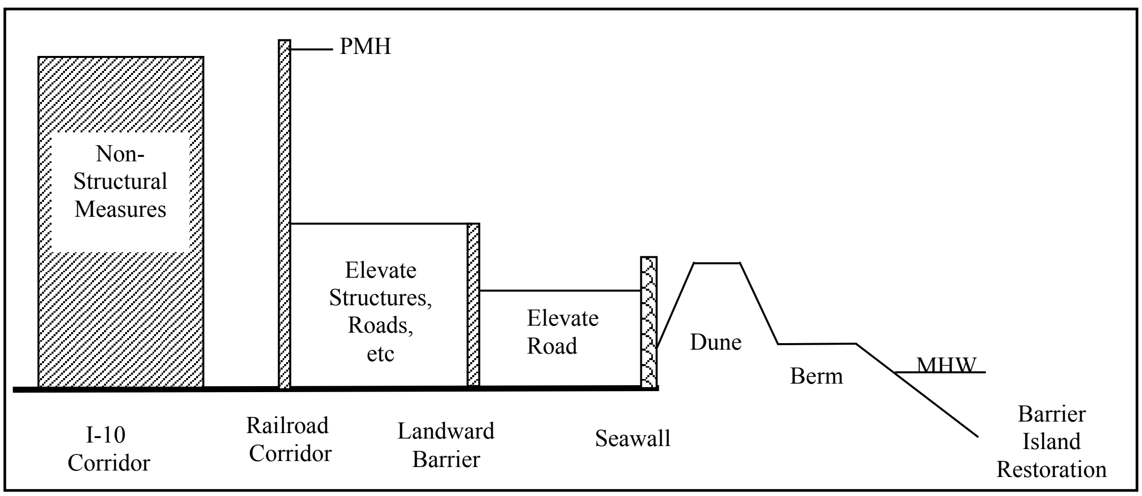

Figure 3. Cross Section, Conceptual Five Lines of Defense.

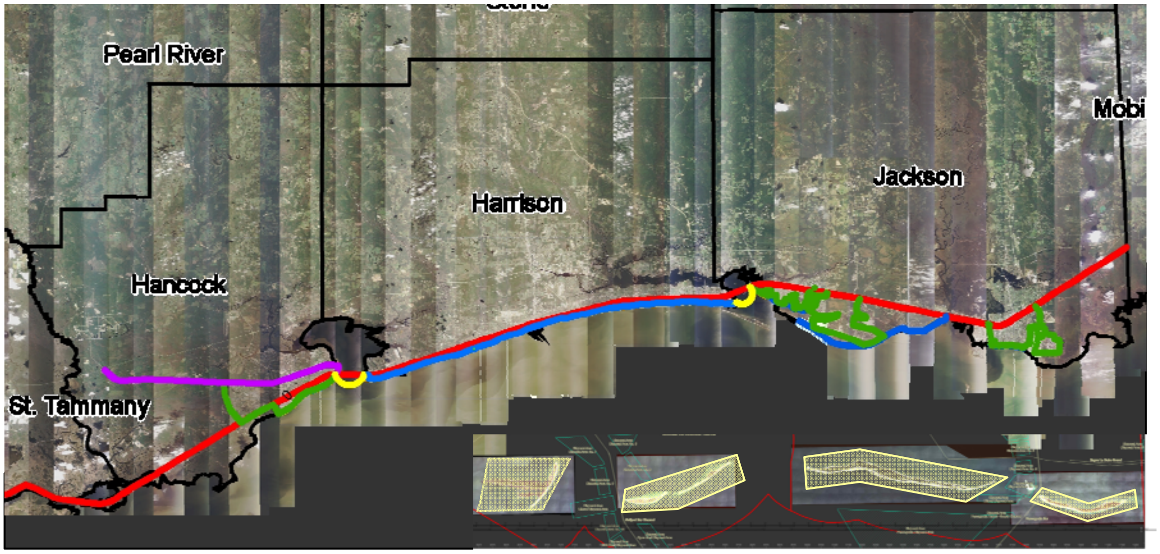

Figure 4. Conceptual Mississippi Sound Five Lines of Defense.

Figure 5. Conceptual Lines of Defense, Hancock County

Figure 6. Berm and Dune System

Figure 7. Example Seawall Concepts

Figure 8. Example Elevated Road/Seawall

Figure 9a. Example Landward Barrier, Railroad Barrier Concept

Figure 9b. Example Landward Barrier, Railroad Barrier Concept

Figure 10. Example Levee System

Figure 11a. Example Flood/Surge Gates

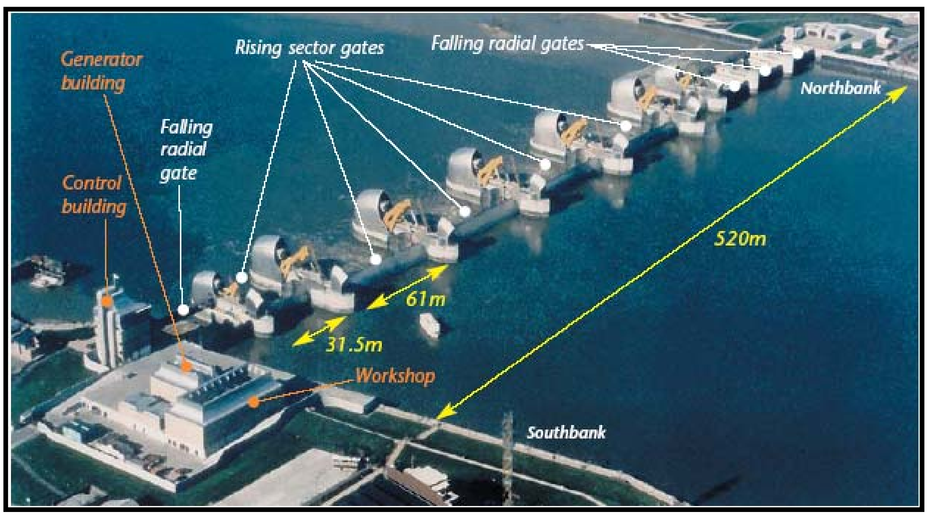

Figure 11b. Example Flood/Surge Gates

The proposed alternatives will be evaluated through the application of a suite of numerical models which predict offshore and nearshore wave conditions, storm surge elevation and inundation, sediment transport, and cross shore/longshore sediment transport and shoreline change, and water quality. A suite of appropriate storm conditions will be developed for evaluation in the numerical simulations. The analysis will determine the level of protection each proposed alternative will provide in reducing storm surge over the Mississippi coast.

HYDRODYNAMIC MODELING

Hydrodynamic modeling of storm surge and waves for the preliminary technical report were conducted to develop a methodology and build a system to predict water level and wave response to hurricanes on the Mississippi coast. Storm parameters defining the initial screening storm were selected based on the probable maximum hurricane (PMH). The storm track selected was the Katrina storm track shifted east and west to make landfall at six different locations along the Mississippi coast. Wind and pressure fields were developed for each of the six PMH storms. The six storms were simulated with a hydrodynamic storm surge model, an offshore deep water wave model, and a nearshore wave model to predict water level and wave response to storms.

Initial Screening Storm

The storm selected to estimate surge from an intense hurricane on the Mississippi coast is based on the probable maximum hurricane (PMH) as documented in NOAA's Technical Report NWS 23 (1979). The PMH has a central pressure of 890 mb. The PMH criteria for the Mississippi coast describe a storm of Category 5 intensity on the Saffir-Simpson Scale (SSC). The radius to maximum winds was approximately 11 nm, that of Hurricane Camille, and the average forward speed applied for the dynamic solution was set at 10 knots. The PMH was run on six tracks with landfalls across coastal Mississippi. The tracks were selected to elicit Category 5 hurricane surge values at locations along the coast. The selected tracks are summarized in Table 1 and plotted in Figure 1.

Table 1.
Modeled Hurricane Tracks

Track	Description	Naming convention
1	Hurricane Katrina shifted 0.0814 deg west	T01
2	Hurricane Katrina shifted 0.0943 deg east	T02
3	Hurricane Katrina shifted 0.2852 deg east	T03
4	Hurricane Katrina shifted 0.5682 deg east	T04
5	Hurricane Katrina shifted 0.7341 deg east	T05
6	Hurricane Katrina shifted 0.9711 deg east	T06

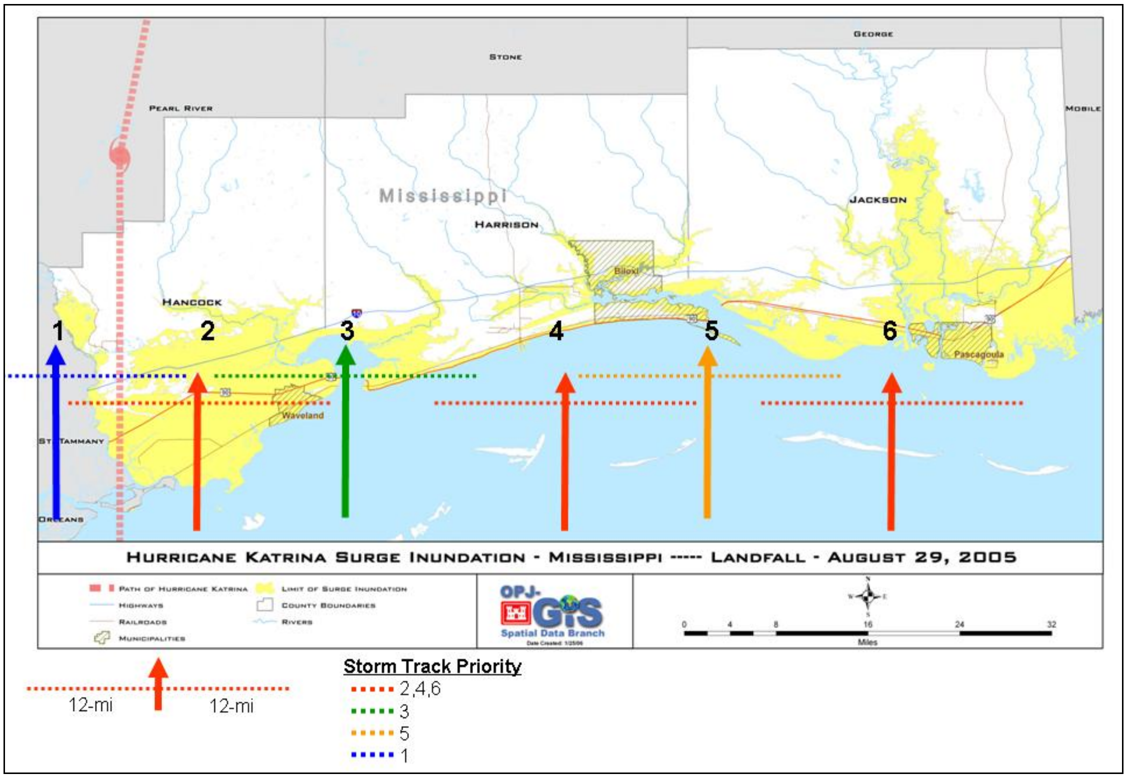

Figure 1. Selected Hurricane Tracks.

Wind and Atmospheric Pressure

Accurate modeling of wave and storm surge levels requires accurate wind and pressure field input to the model. This section describes the methodology to generate wind and pressure fields for the PMH. The wind fields specified with this methodology drive the storm surge simulations and the offshore and nearshore wave simulations.

Methodology

The wind and pressure fields were developed with a highly refined meso-scale vortex numerical model for the specification of surface wind and pressure fields in tropical cyclones. Model inputs include the central pressure index (CPI), radius to maximum wind (RMW), forward velocity, and storm track locations. The inputs for the design storm for this phase of the study are given in the Initial Screening Storm section. The dynamical model operates on these inputs and additional inputs required (defined below as calibration criteria) to produce a wind and pressure field. The simulation assumes that a hurricane is in steady state offshore and does not begin to weaken until the center arrives at the coast. A filling model developed by Vickery et. al. (1995), which describes the land effects in terms of CPI, was applied. The filling model takes the following form where t is time in hours and po is pressure in mb:

$$\Delta p(t) = \Delta p_o \exp(-at) \quad (1)$$

The filling constant a is given as:

$$a = a_o + a_1 \Delta p_o \quad (2)$$

Where ao and a1 are 0.035 and 0.00050 for Gulf Coast hurricanes. A 2-hour delay was incorporated into the decay model.

Model calibration was required to explore and define additional storm criteria that are not included in the PMH criteria. The additional criteria include the pressure profile peakedness parameter, so called Holland's B, the ambient pressure field, peripheral pressure index, the boundary layer depth, the azimuth of the wind maximum, and the landfall-filling model described above. The calibration effort required a period of experimentation with recent Gulf of Mexico storms of intensity comparable to the PMH. Hurricanes Ivan (2004) and Katrina (2005) were selected as they affected the Mississippi coast and excellent kinematic descriptions of the wind field already exist. The objective of the calibration is to ensure that the surface marine wind field specified is consistent with modern thinking as to the relationship between the storm criteria and inner core maximum surface winds for modern averaging intervals, including the standard 30-minute average, and the definition of "sustained" wind speed, which is a stochastic wind variable which may be defined as the median peak 1-minute wind speed within the 30-minute period.

The tropical cyclone boundary layer model was setup on a target domain that covers the range of two working grids. Winds and pressures are output on these two grid systems. The basin scale grid is 0.1 degree, (~10 km) covering the domain 18-30.8N, 98-80W, the fine scale grid is 0.025 degree, (~2 km) covering the domain 28.5N-30.8N, 94.25W-88W. Grid spacing of the fine domain is sufficient to properly resolve the radius of maximum wind (RMW) in the hindcast storms. Output is specified at a 15-minute time step.

Results

The maximum wind speed generated over space and time is approximately 135 mph. This speed is based on a 10-m equivalent neutral stable 30-minute average wind speed. If the maximum wind speed is converted to a 1-minute average (the general average interval to quantify the Saffir-Simpson Hurricane Scale) the magnitude would be about 166 mph or an intense Category 5 hurricane. The final wind and pressure fields provided input to the surge and wave models.

Offshore Wave Modeling

Offshore waves are required as a boundary condition for the nearshore wave modeling. The generation of the wave field and directional wave spectra for the various hurricane storm tracks is based on the implementation of a third generation discrete spectral wave model called WAM, (Komen et al, 1994). This model solves the action balance equation:

$$\frac{\partial N}{\partial t} + \overrightarrow{c_G} \cdot \frac{\partial N}{\partial \vec{x}} = \omega^{-1} \cdot \sum_i S_i \qquad (5)$$

where: N is the action density defined by F(f,θ,xi,t)/ω, where F is the energy density spectrum defined in frequency, (f) direction (θ) over space, (xi)and time, (t) and the radial frequency ω is equal to 2πf. Si represent the source-sink terms:

$$\sum_i S_i = S_{in} + S_{nl} + S_{ds} + S_{w-b} + S_{bk} \qquad (6)$$

and Sin is the atmospheric input, Snl represents the nonlinear wave-wave interactions, Sds is the high frequency breaking (white-capping), Sw-b is wave bottom effects (bottom friction), and Sbk is depth limited wave breaking. The solution is solved for the spatial and temporal variation of action in frequency f, direction θ, over a fixed grid defined in xi (generally a fixed longitude latitude geospatial grid).

Computationally Equation 5 is solved in two steps. The advection term (second term in Equation 1) is solved first accounting for the propagation of wave energy. Each packet of energy in frequency and direction is moved based on the group speed of that particular frequency band and water depth it be situated. This assumes linear theory and superposition of wave packets. In a fixed longitude latitude grid system curvature effects are resolved where the energy is propagated in a spherical coordinate system (or along great circle paths). As the water depth decreases, the full dispersion relationship is applied. Wave shoaling and refraction will effect the propagation of the energy packets.

After every propagation step the solution to the time rate change of the action density is solved including the source term integration. The wind field is read, and the atmospheric input source (Sin) is applied. The nonlinear wave-wave interaction source term is the mechanism that self-stabilizes the spectral energy, transferring portions of the energy to the forward face and high frequency tail. Dissipation (Sds) removes portions of energy that become too energetic for the given frequency band. For application in arbitrary depths energy is removed via the wave-bottom sink (Sw-b) and ultimately in very shallow water the spectrum releases much of its available energy due to breaking (Sbk). A more complete theoretical derivation, formulation of the source terms can be found in Komen et al. (1994).

Methodology

A grid nesting approach was applied for the offshore wave simulations. This effectively reduces the computational demand on the solution technique, and also maximizes the use of higher resolution wind estimates in the coastal area. The two grids are defined in graphical form provided in Figure 2, and documented in Table 2. These grids were developed and calibrated during the Interagency Performance Evaluation Taskforce (IPET) Task 4 Hurricane Katrina study. Comparison of WAM model results to data measurements at NDBC buoy 42007 are provided in Figure 3. A complete description of the validation is found in Volume 4 of the IPET final report.

Two time steps are applied in the wave model simulations. The propagation time step is set to attain numerical stability. The second time step for source term integration is set to the physical processes and relaxation times of Sin, Snl, Sds, Sw-b. In addition the time steps are required to be integer multiples of the wind input, and the fine-scale grid time step is a divisor of the basin-scale propagation time step.

All simulations are initiated from simple fetch laws using the first wind field. During the basin-scale simulation, boundary condition information is generated at the defined propagation time step and consists of two-dimensional wave spectra (in frequency, and direction) along the domain defined by the red box in Figure 2. In addition wave field information files are built to illustrate the time, and spatial variation of various wave related parameters for each of the six hurricane track cases. Upon completion of each of the WAM basin-scale simulations, the regional simulations are executed. These model runs are forced with the higher resolution regional-wind fields and the boundary condition information derived from the basin level simulations.

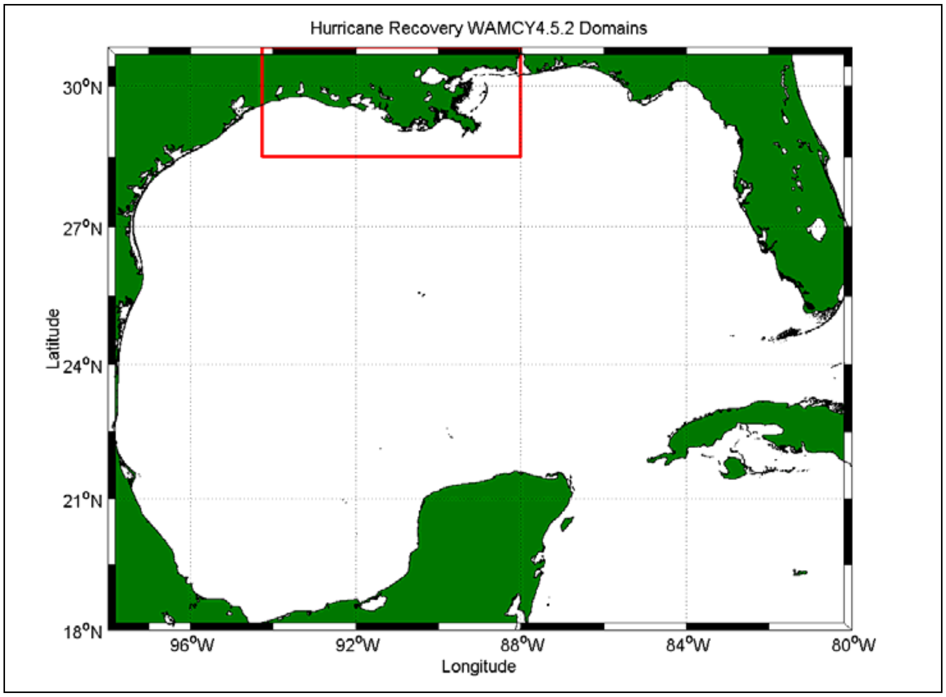

Figure 2. WAM Cycle 4.5.2 Wave Model Grid Domains, Where the Basin-Scale is Defined as the Entire Graphic and the Region-Scale is the Red Box.

Table 2.
Wave Field Domain Characterization

	Longitude (deg)		Latitude (deg)			Δt(prop) /
Domain	**West**	**East**	**South**	**North**	**Res. (deg)**	**Δt(source) (sec)**
Basin	-98.00	-80.00	18.00	30.80	0.1	150/300
Region	-94.20	-88.00	28.50	30.50	0.05	75/300

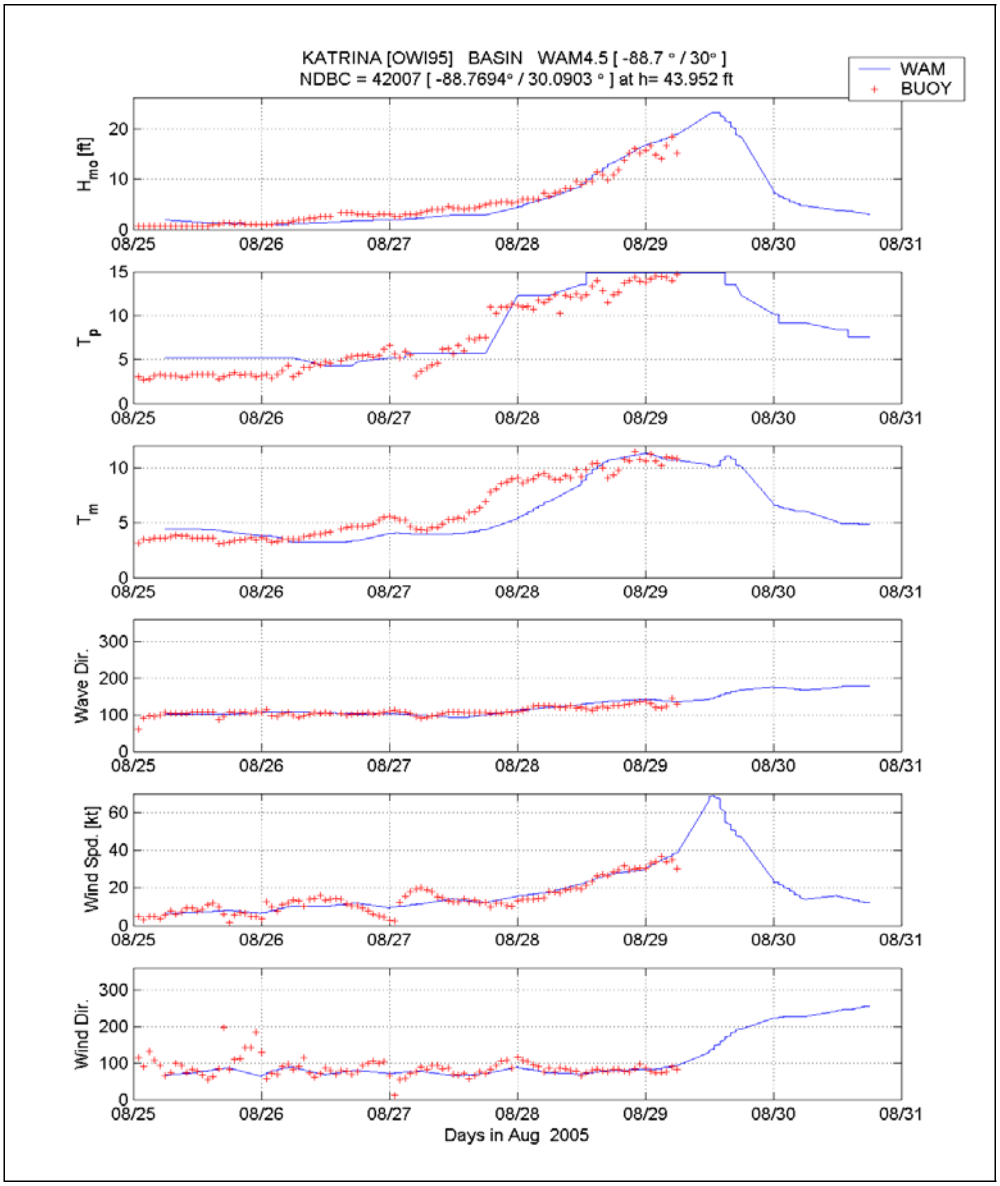

Figure 3. WAM Wave Model Validation with NDBC Buoy

The purpose of the offshore wave simulations is to supply the nearshore wave modeling effort supported by STWAVE (Smith, et al, 2001). Thirty-eight output locations were defined in the region-scale WAM grid. The WAM directional wave spectra are output every 15-min at 28 discrete frequency bands (exponential distribution where fn+1 = 1.1·fn and f0 = 0.031384), and 24 direction bands centered every 15-deg starting at θ0 = 7.5).

Results

As previously mentioned, the wave model simulations reflect the time and spatial variation of one hurricane wind field projected onto various storm tracks. This will depict the growth and propagation of the wave energy in the target domains. A summary of the absolute maximum wave heights are documented in Table 3.

Table 3.
Basin Maximum Wave Height Locations

Track #	Description	Hmo (ft)	Storm Duration (Days)	Location Long	Location Lat
01	Hurricane Katrina shift 0.0814-deg W	56.4	4.50	-87.5	25.6
02	Hurricane Katrina shift 0.0934-deg E	56.1	4.50	-87.4	25.6
03	Hurricane Katrina shift 0.2852-deg E	56.1	4.50	-87.2	25.6
04	Hurricane Katrina shift 0.5682-deg E	56.1	4.50	-86.9	25.6
05	Hurricane Katrina shift 0.7341-deg E	56.1	4.50	-86.7	25.6
06	Hurricane Katrina shift 0.9711-deg E	56.1	4.50	-86.5	25.6

Nearshore Wave Modeling

Nearshore waves are required to calculate wave runup and overtopping on structures and beaches and wave forces on structures. The numerical model STWAVE (Smith, Sherlock, and Resio 2001; Smith and Smith 2001; Thompson, Smith, and Miller 2004) was applied to generate and transform waves to the shore. STWAVE numerically solves the steady-state conservation of spectral action balance along backward-traced wave rays:

$$(C_{ga})_x \frac{\partial}{\partial x} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} + (C_{ga})_y \frac{\partial}{\partial y} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} = \sum \frac{S}{\omega_r} \quad (7)$$

where

Cga = absolute wave group celerity
x,y = spatial coordinates, subscripts indicate x and y components
Ca = absolute wave celerity
μ = current direction
α = propagation direction of spectral component
E = spectral energy density
f = frequency of spectral component
ωr = relative angular frequency (frequency relative to the current)
S = energy source/sink terms

The source terms include wind input, nonlinear wave-wave interactions, dissipation within the wave field, and surf-zone breaking. The terms on the left-hand side of Equation 7 represent wave propagation (refraction and shoaling), and the source terms on the right-hand side of the equation represent energy growth and decay in the spectrum.

The assumptions made in STWAVE are as follows:
Mild bottom slope and negligible wave reflection.
Steady waves, currents, and winds.
Linear refraction and shoaling.
Depth-uniform current.

STWAVE can be implemented as either a half-plane model, meaning that only waves propagating toward the coast are represented, or a full-plane model, allowing generation and propagation in all directions. Wave breaking in the surf zone limits the maximum wave height based on the local water depth and wave steepness:

$$H_{mo_{\max}} = 0.1 L \tanh kd \quad (8)$$

where

H_{mo}	= zero-moment wave height
L	= wavelength
k	= wave number
d	= water depth

STWAVE is a finite-difference model and calculates wave spectra on a rectangular grid. The model outputs zero-moment wave height, peak wave period (Tp), and mean wave direction (αm) at all grid points and two-dimensional spectra at selected grid points. Option has been added to input variable wind and surge fields. The surge significantly alters the wave transformation and generation for the hurricane simulations in shallow areas (such as Lake Pontchartrain) and where low-laying areas are flooded. Spatially varying wind input is important to simulate the complex wind fields in hurricanes.

a. Wave Model Inputs

The inputs required to execute STWAVE include:

- Bathymetry grid (including shoreline position and grid size and resolution).
- Incident frequency-direction wave spectra on the offshore grid boundary.
- Current field (optional).
- Surge and/or tide fields, wind speed, and wind direction (optional).
- Bottom friction coefficients (optional).

b. Wave Model Outputs

The outputs generated by STWAVE include:

- Fields of energy-based, zero-moment wave height, peak spectral wave period, and mean direction.
- Wave spectra at selected locations (optional).
- Fields of radiation stress gradients to use as input to ADCIRC (optional).

Methodology

STWAVE was applied on two grids for the Mississippi and Alabama Coasts: Eastern Mississippi/Alabama grid and Western Mississippi/Eastern Louisiana grid. The input for each grid includes the bathymetry (interpolated from the ADCIRC domain), surge fields (interpolated from ADCIRC output), and wind (interpolated from ADCIRC output). The model output includes wave parameters (height, peak wave period, and mean direction) and radiation stresses to be applied as forcing in ADCIRC to calculate wave setup.

The bathymetry grids cover the entire Gulf of Mexico coastline of Mississippi and extend east into Alabama and west into Louisiana at a resolution of 656 ft (200 m). The East MS-AL grid domain covers Eastern Mississippi and Alabama The domain is approximately 70 by 75 miles (112.6 by 121 km). The West MS-Southeast LA grid is approximately 85 by 92 miles (136.6 by 148.8 km) and extends from Mississippi Sound to the Mississippi River. The domain was broken into two parts to capture the transformation of offshore waves from approximately the 100 ft (30 m) depth contour to the shoreline. The grid parameters are given in Table 4. Figure 4 shows the bathymetry for the MS-AL grid and Figure 5 shows the bathymetry for the MS-SE LA grid. Brown areas in the bathymetry plots indicate land areas at 0 ft or higher elevation. These simulations are forced with both the local winds interpolated from ADCIRC and waves interpolated on the offshore boundary from the regional WAM model. The simulations were run with the half-plane version of STWAVE for computational efficiency.

Table 4.
STWAVE Grid Specifications

Grid	State Plane	X origin ft	Y origin ft	Δx ft	Δx ft	Orient Deg	X cells	Y cells
East MS-AL	LA Offshore	4463976.4	1653950.1	656	656	90	563	605
West MS-SE LA	LA Offshore	4294586.6	1639491.5	656	656	141	683	744

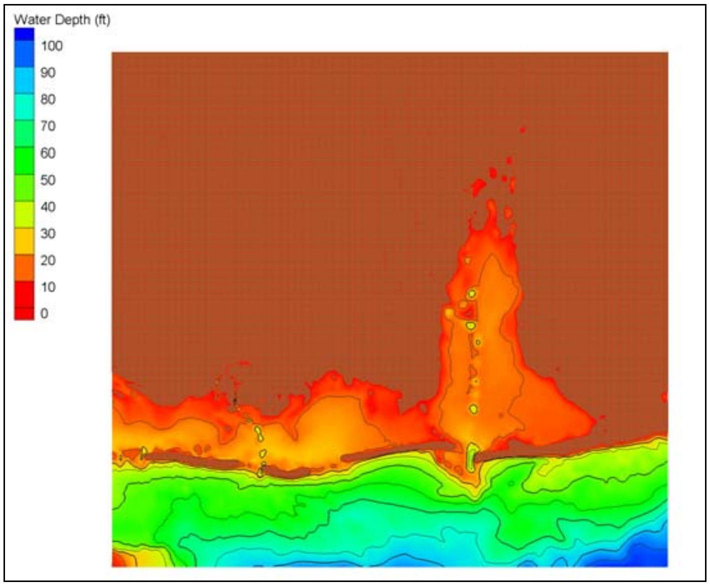

Figure 4. MS-AL Bathymetry Grid (Depths in Feet).

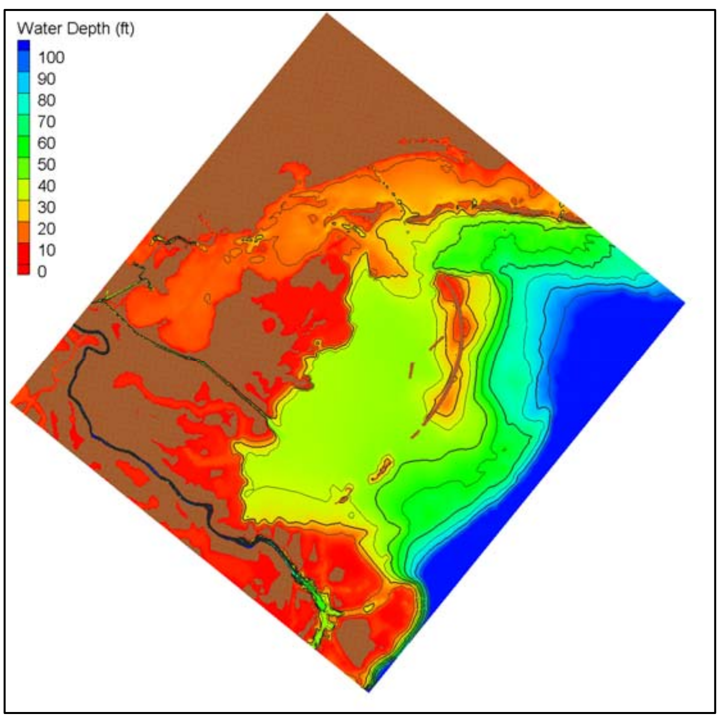

Figure 5. West MS-SE LA Bathymetry Grid (Depths in Feet).

Results

The STWAVE simulations include six storm tracks. All storms were run on both STWAVE grids. STWAVE was run for approximately a two-day period for each storm to capture the peak wave conditions. Because STWAVE is a steady state model, spin-up time is not required for the simulations. To provide the wave height and period for runup calculations, the STWAVE output is processed to extract the largest significant wave height for each grid cell in each domain. Radiation stress gradients were calculated and applied as a forcing condition to the surge model.

Example output generated from the model results are provided in Figures 6-9. Figures 6-7 show the maximum significant wave height and coincident direction produce by track T05 for the MS-AL and MS-SE LA grids, respectively. Figures 8-9 are the peak wave periods at the time of maximum wave height. The maximum significant wave heights and periods in representative sections can be selected for calculating wave runup and overtopping, wave forcing on structures, or other design purposes.

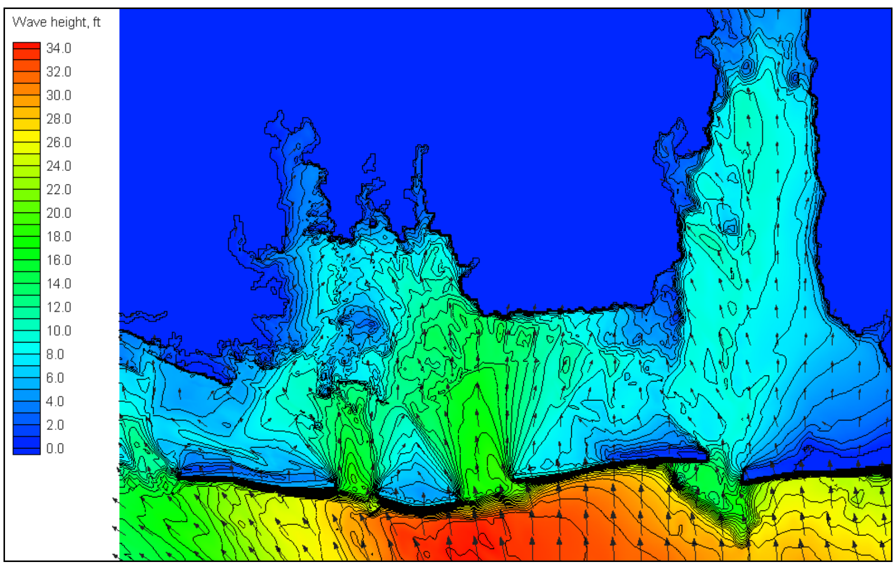

Figure 6. Maximum Significant Wave Height and Coincident Direction for the MS-AL Grid for Track T05.

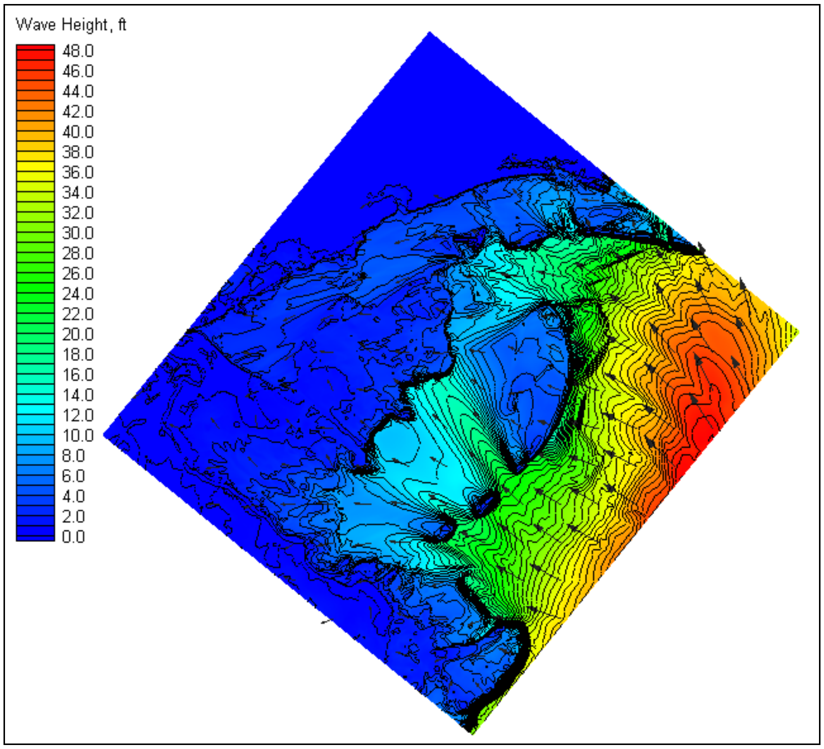

Figure 7. Maximum Significant Wave Height and Coincident Direction for fhe MS-SE LA Grid for Track T05.

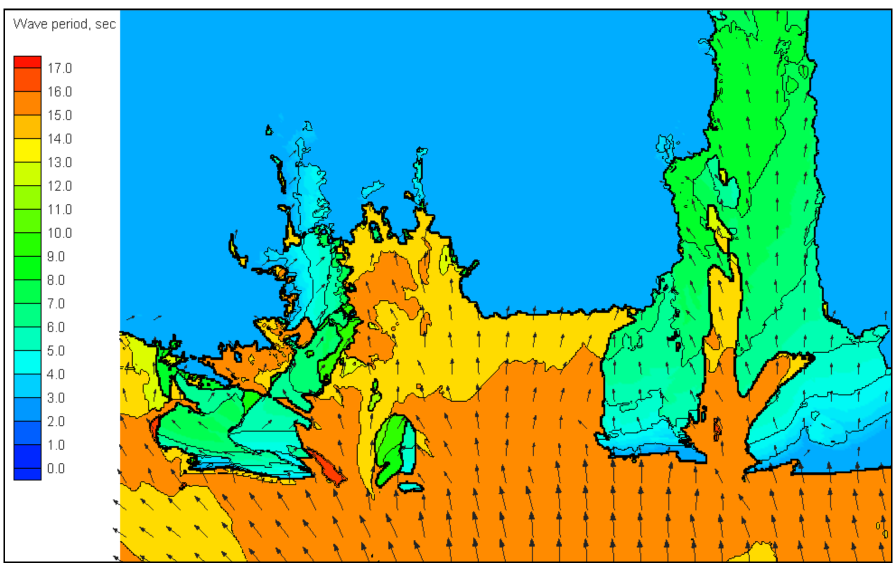

Figure 8. Peak Wave Period at the Time of Maximum Wave Height for the MS-AL Grid for Track T05.

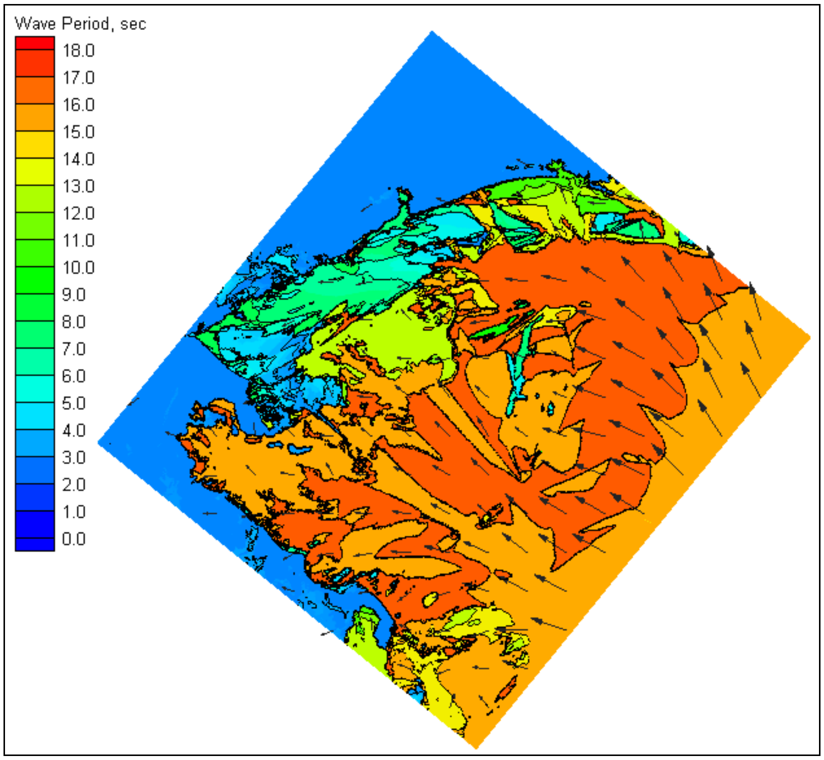

Figure 9. Peak Wave Period at the Time of Maximum Wave Height for the MS-SE LA Grid for Track T05.

Storm Surge Modeling

The ADvanced CIRCulation (ADCIRC) (Leuttich et al. 1992) hydrodynamic model is being applied to estimate storm surge. ADCIRC is a finite element hydrodynamic circulation numerical model for the simulation of water level and current over an unstructured gridded domain. ADCIRC is a two-dimensional depth integrated (2DDI) model that can simulate tidally-, wind- and wave-driven circulation in coastal waters as well as hurricane storm surge and flooding. Extensive storm surge model development, application, and validation efforts have been conducted in southern Louisiana and parts of Mississippi.

Computational Model

ADCIRC was chosen for simulating the long-wave hydrodynamic processes in the study area. Imposing the wind and atmospheric pressure fields, the ADCIRC model can replicate tide induced and storm-surge water levels and currents. In two dimensions, the model is formulated with the depth-averaged shallow water equations for conservation of mass and momentum. Furthermore, the formulation assumes that the water is incompressible, hydrostatic pressure conditions exist, and that the Boussinesq approximation is valid. Using the standard quadratic parameterization for bottom stress and neglecting baroclinic terms and lateral diffusion/dispersion effects, the following set of conservation equations in primitive, nonconservative form, and expressed in a spherical coordinate system, are incorporated in the model (Flather 1988; Kolar et al. 1993):

$$\frac{\partial U}{\partial t} + \frac{1}{r\cos\phi} U \frac{\partial U}{\partial \lambda} + \frac{1}{R} V \frac{\partial U}{\partial \phi} - \left[\frac{\tan\phi}{R} U + f\right] V =$$

(9)

$$- \frac{1}{R\cos\phi} \frac{\partial}{\partial \lambda}\left[\frac{p_s}{\rho_0} + g(\zeta - \eta)\right] + \frac{\tau_{s\lambda}}{\rho_0 H} - \tau_* U$$

(10)

$$\frac{\partial V}{\partial t} + \frac{1}{r\cos\phi} U \frac{\partial V}{\partial \lambda} + \frac{1}{R} V \frac{\partial V}{\partial \phi} - \left[\frac{\tan\phi}{R} U + f\right] U =$$

$$- \frac{1}{R} \frac{\partial}{\partial \phi}\left[\frac{p_s}{\rho_0} + g(\zeta - \eta)\right] + \frac{\tau_{s\lambda}}{\rho_0 H} - \tau_* V$$

$$\frac{\partial \zeta}{\partial t} + \frac{1}{R\cos\phi}\left[\frac{\partial UH}{\partial \lambda} + \frac{\partial (UV\cos\phi)}{\partial \phi}\right] = 0$$

(11)

where

t = time,

λ and φ = degrees longitude (east of Greenwich is taken positive) and degrees latitude (north of the equator is taken positive),

ζ = free surface elevation relative to the geoid,

U and V = depth-averaged horizontal velocities in the longitudinal and latitudinal directions, respectively,

R = the radius of the earth,

H = ζ + h = total water column depth,

h = bathymetric depth relative to the geoid,

f = 2Ω sin φ = Coriolis parameter,

Ω = angular speed of the earth,

ps = atmospheric pressure at free surface,

g = acceleration due to gravity,

η = effective Newtonian equilibrium tide-generating potential parameter,

ρ0 = reference density of water,

τsλ and τsφ = applied free surface stresses in the longitudinal and latitudinal directions, respectively, and

τ = bottom shear stress and is given by the expression Cf (U2 + V2)1/2 /H where Cf is the bottom friction coefficient.

The momentum equations (Equations 9 and 10) are differentiated with respect to λ and τ and substituted into the time differentiated continuity equation (Equation 11) to develop the following Generalized Wave Continuity Equation (GWCE):

$$\frac{\partial^2\zeta}{\partial t^2} + \tau_0\frac{\partial\zeta}{\partial t} - \frac{1}{R\cos\phi}\frac{\partial}{\partial\lambda}\left[\frac{1}{R\cos\phi}\left(\frac{\partial HUU}{\partial\lambda} + \frac{\partial(HUV\cos\phi)}{\partial\phi}\right) - UVH\,\frac{\tan\phi}{R}\right]$$

$$\left[-2\omega\sin\phi\,HV + \frac{H}{R\cos\phi}\frac{\partial}{\partial\lambda}\left(g\,(\zeta - \alpha\eta) + \frac{p_5}{[}\,p_0\right) + \tau_*\,HU - \tau_0\,HU - \tau_{s\,\lambda}\right]$$

$$-\frac{1}{R}\frac{\partial}{\partial\phi}\left[\frac{1}{R\cos\phi}\left(\frac{\partial HVV}{\partial\lambda} + \frac{\partial HVV\cos\phi}{\partial\phi}\right) + UUH\,\frac{\tan\phi}{R} + 2\,\omega\sin\phi\,HU\right] \qquad (12)$$

$$+\frac{H}{R}\frac{\partial}{\partial\phi}\left(g(\zeta - alfa\eta) + \frac{p_s}{\rho_0}\right) + \tau_* - \tau_0 HV - \frac{\tau_{s\lambda}}{\rho_0}]$$

$$-\frac{\partial}{\partial t}[\frac{VH}{R}\tan\phi] - \tau_0[\frac{VH}{R}\tan\phi] = 0$$

The ADCIRC model solves the GWCE in conjunction with the primitive momentum equations given in Equations 9 and 10. The GWCE-based solution scheme eliminates several problems associated with finite-element programs that solve the primitive forms of the continuity and momentum equations, including spurious modes of oscillation and artificial damping of the tidal signal. Forcing functions include time-varying water-surface elevations, wind shear stresses, and atmospheric pressure gradients.

The ADCIRC model uses a finite-element algorithm in solving the defined governing equations over complicated bathymetry encompassed by irregular sea/ shore boundaries. This algorithm allows for extremely flexible spatial discretizations over the entire computational domain and has demonstrated excellent stability characteristics. The advantage of this flexibility in developing a computational grid is that larger elements can be used in open-ocean regions where less resolution is needed, whereas smaller elements can be applied in the nearshore and estuary areas where finer resolution is required to resolve hydrodynamic details.

Methodology

The ADCIRC grid utilized during this study is that which was calibrated during the Interagency Performance Evaluation Taskforce IPET Task 4 Hurricane Katrina study (Figure 10). The model incorporates the western North Atlantic Ocean, the Gulf of Mexico and Caribbean Sea to allow for full dynamic coupling between oceans, continental shelves, and the coastal floodplain. The grid is locally refined to resolve features such as inlets, rivers, navigation channels, levee systems and local topography/bathymetry. Figure 11 provides a plot of the high water mark error analysis for Mississippi storm surge calibration conducted under the IPET study Task 4. A complete description of the calibration and validation is found in Volume 4 of the IPET final report. The storm surge modeling consists of 12 ADCIRC model simulations. The PMH storm was run on the six identified tracks at the historical Hurricane Katrina translation speed and with radiation stress gradients derived from the nearshore wave model to feed back into the surge model as a surface stress.

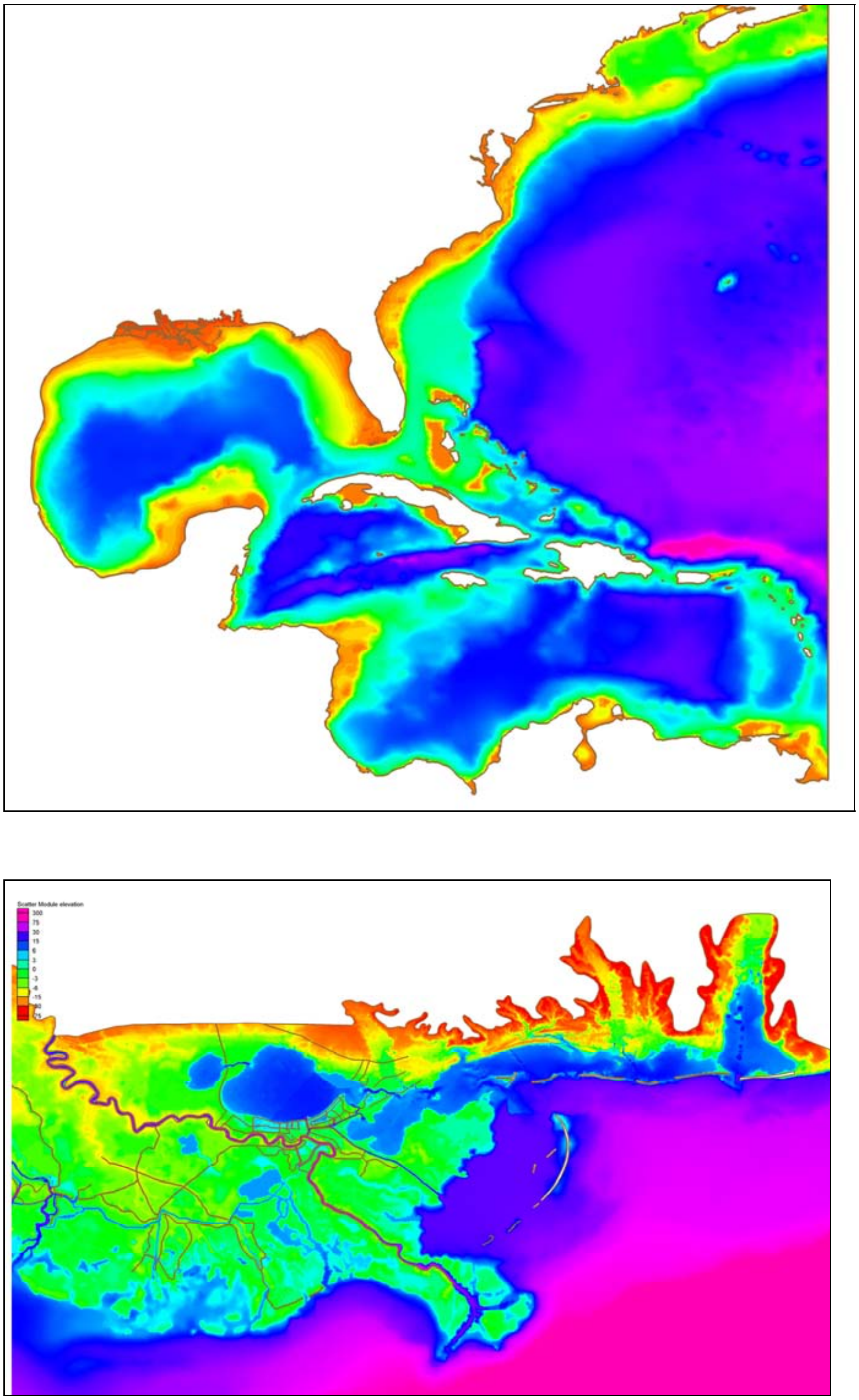

Figure 10. A. Computational Domain. B. Detailed Bathymetry and Topography for Southern Mississippi and Louisiana.

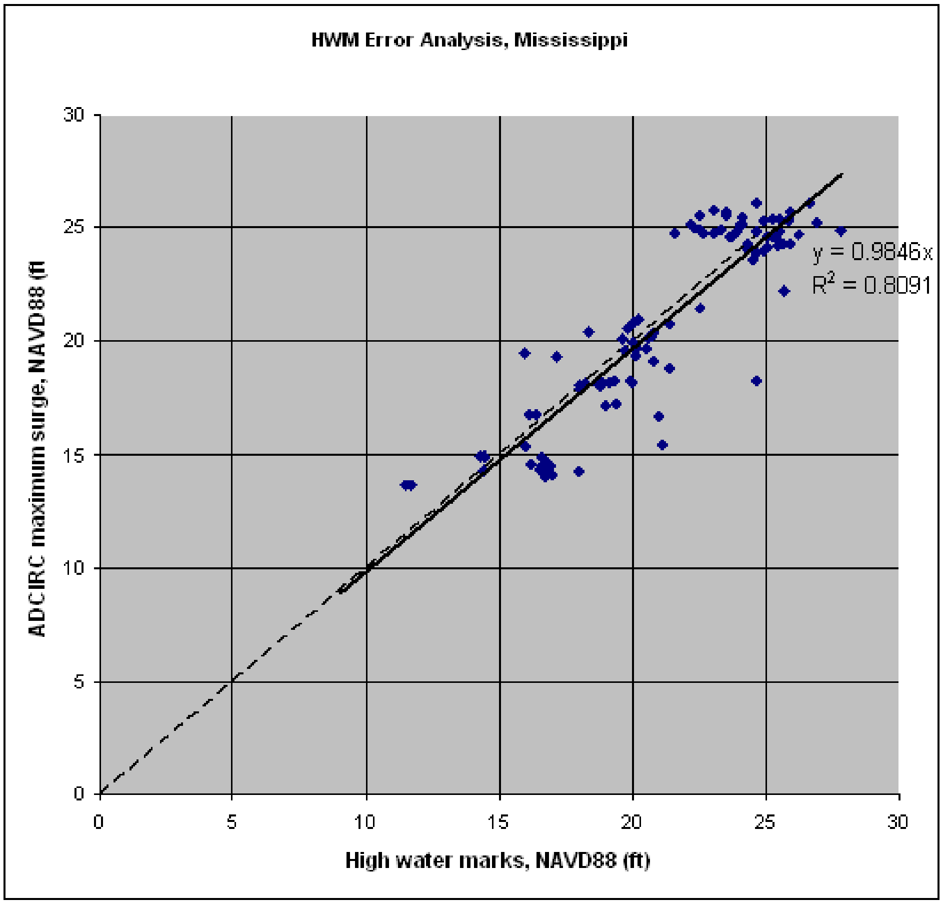

Figure 11. High Water Mark Error Analysis for Mississippi Storm Surge Calibration Conducted Under the IPET Study Task 4.

Results

The six storms were simulated with and without wave radiation stress gradients. The primary goal of the simulation analysis was to the capability to estimate overall peak water level for a given storm. This involved an examination of the entire spatial domain every 900 seconds (15 minutes) to determine if water levels exceeded the previous time steps maximum water level at any point in the domain. The result of this analysis is a maximum envelope of water level for a given simulation.

Figure 12 is a composite of peak storm surge for all tracks, showing that the six tracks selected for simulating represent inundation along the entire Mississippi coast. The maximum water level for each track is summarized in Table 5.

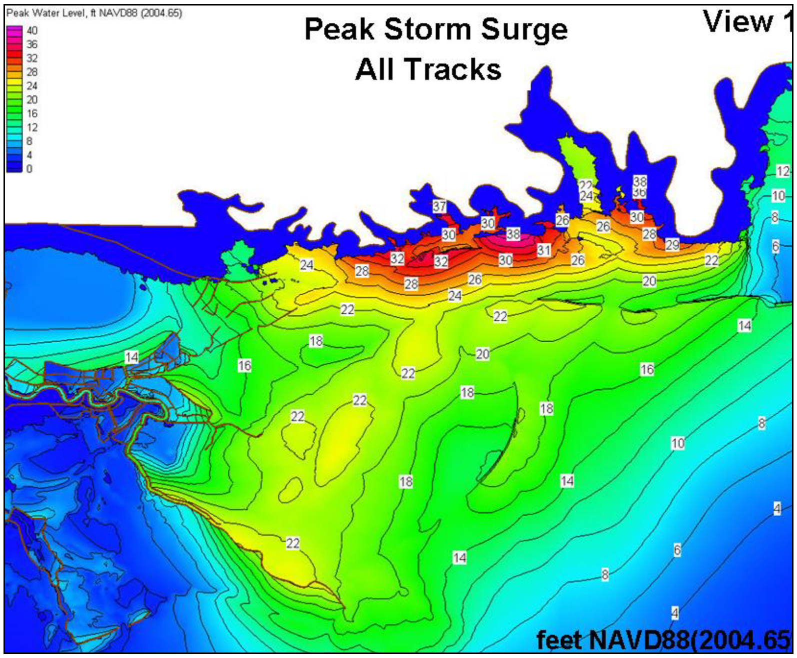

Figure 12. Peak Storm Surge for all Tracks.

Table 5.
Maximum Water Level For Each Track (without waves)

Track	Maximum Water Level (ft) (NAVD88 2004.65)	Location
01	28	Near Diamondhead
02	32	North of Long Beach
03	34	North of Gulfport
04	38	North of Biloxi Bay in D'Iberville
05	30	Near Big Point
06	38	East of Hurley

Storm hydrographs can be produced at any location within the modeling domain. An example hydrograph for all storm tracks was produced for a location near Waveland, MS (see Figure 1) and is presented in Figure 13. Some hydrographs show a drawdown as the storm passes along the eastern most tracks.

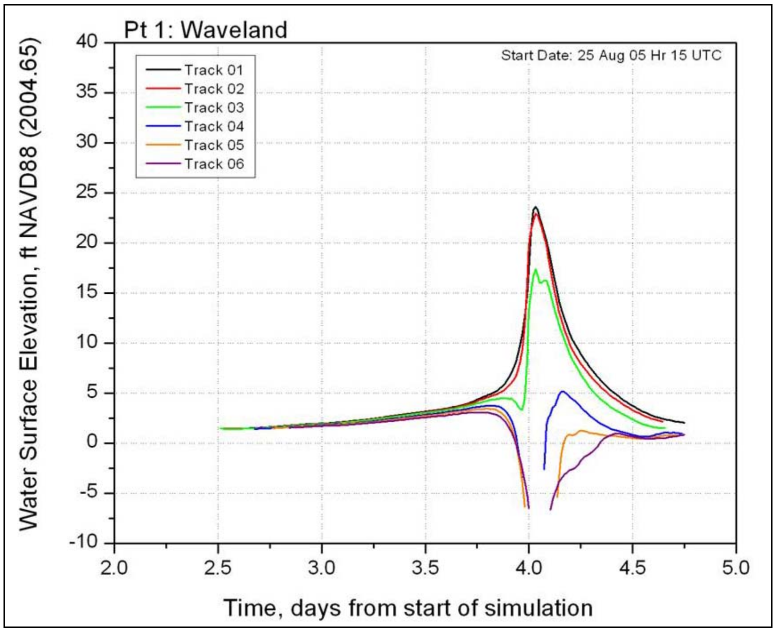

Figure 13. Hydrograph Near Waveland, MS for all Tracks.

Figure 14 provides the peak storm surge from all tracks with the inclusion of wave radiation stress gradients. Generally, the maximum increase in water level is approximately 1 ft and occurs on the sound side of the barrier islands off the Mississippi coast. The results presented are preliminary.. In shallow regions, where the ADCIRC grid is less resolved than the STWAVE grid, significant portions of the integrated wave setup may be lost (through grid aliasing). Also, at the shoreline, the STWAVE and ADCIRC resolution may not be sufficient to accurately capture the radiation stress gradients, so the setup from the final breaking at the beach may be largely missed (approx 15% of breaking wave height). For the second phase of the study, additional analysis will be performed and adjustments to the grid made, if necessary, to ensure the wave effects are properly represented.

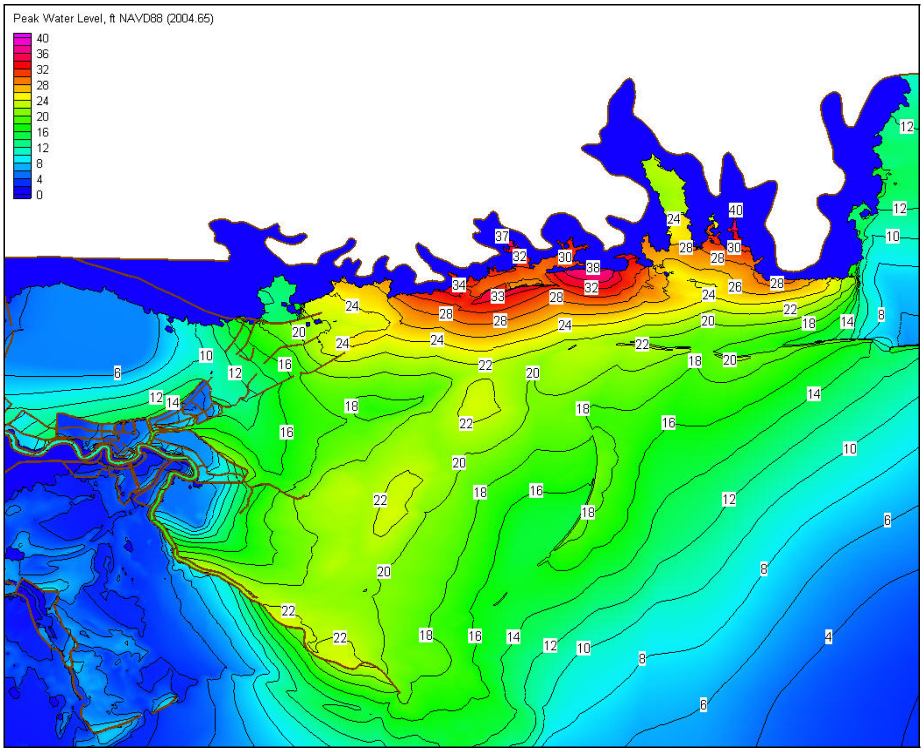

Figure 14. Peak Storm Surge for all Tracks Including Radiation Stress Gradient Forcing.

Sensitivity

Sensitivity runs were made to investigate the dependency of intense hurricane surge on drag law specification. The standard method for applying surface wind stress within storm surge models, such as ADCIRC, is the quadratic stress law via a surface drag coefficient Cw. This coefficient is based on regression fits of field measurements, under conditions of moderate to strong wind speed, and has been found to be directly related to wind speed, wave state and atmospheric stability (Garratt, 1977, Large and Pond, 1981 and Trenberth et. al. 1989). Recent research (Powell, 2003) has found that under extreme winds, the linear extrapolation of the drag coefficient provides a clear overestimate of Cw and that the enforcement of a drag coefficient limit may be appropriate. Within this initial phase of this study, a preliminary investigation into the dependency of hurricane surge on the drag law specification was investigated by specifying drag coefficient upper limits of 0.0025 and 0.004. The results of this investigation are shown in Figures 15-18. The regression fit of Large and Pond (1981) has been modified to impose lower limits on the drag coefficient (Trenberth et. al. 1989).

Peak surge elevations for track T04 applying drag coefficient cutoffs of 0.004 and 0.0025 are plotted in Figures 15 and 16, respectively. The difference in the peak surge elevations from the simulations with no drag cutoff imposed are plotted in Figures 17 and 18. Peak surges are reduced by as much as 1.5 feet with a cutoff of 0.004 and 10 feet applying 0.0025. The 0.0025 is an extreme case. In offshore wave modeling, drag cutoffs of 0.003 to 0.004 are typically employed. The issue of limits on the specification of the drag coefficient for use in computing hurricane storm surge simulation will be investigated further during the second phase of this study.

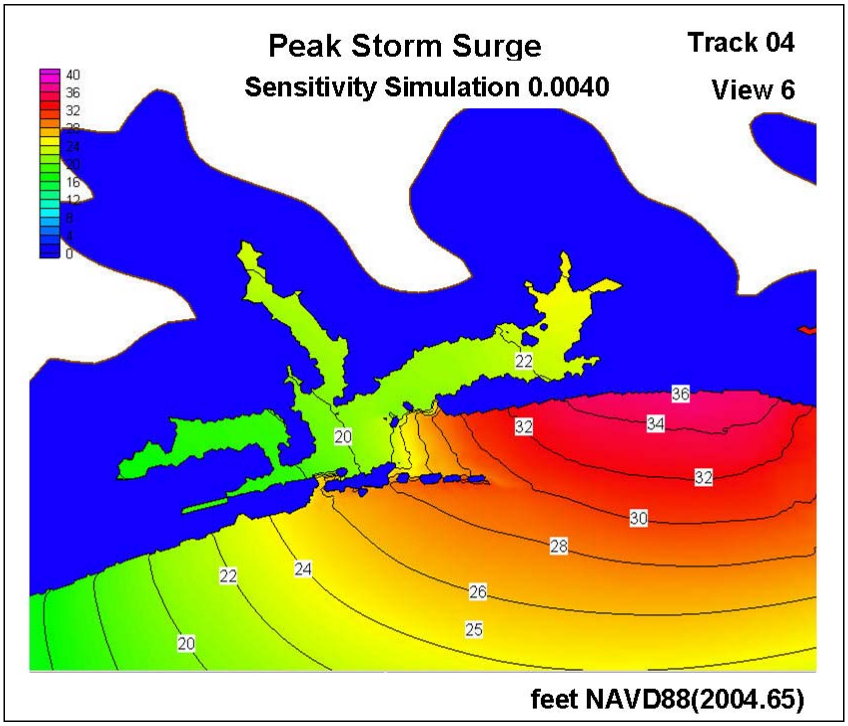

Figure 15. Peak Surge Elevation Applying 0.0040 Drag Coefficient Cutoff.

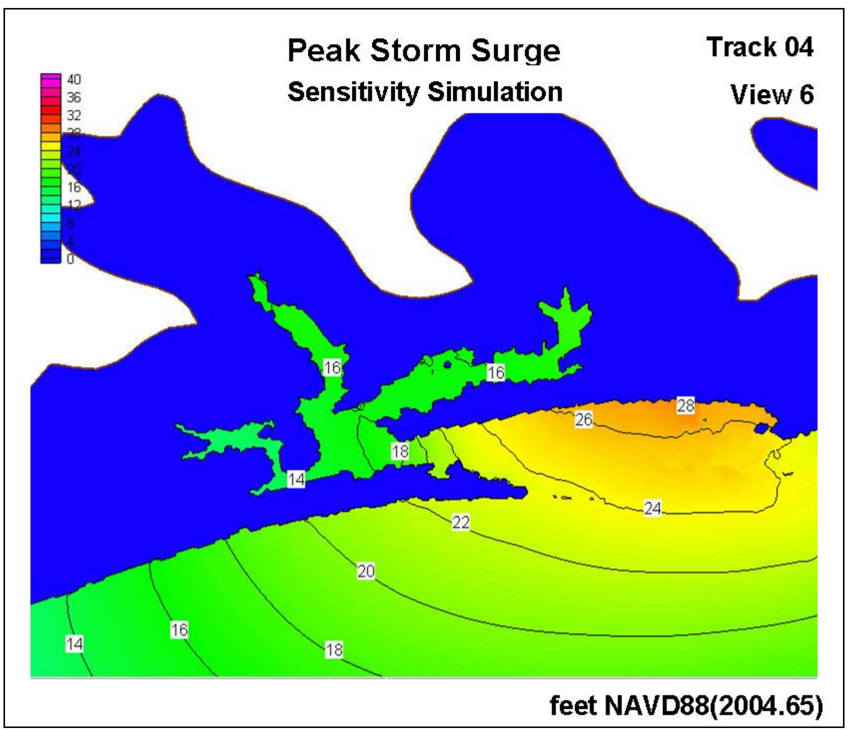

Figure 16. Peak Surge Elevation Applying 0.0025 Drag Coefficient Cutoff.

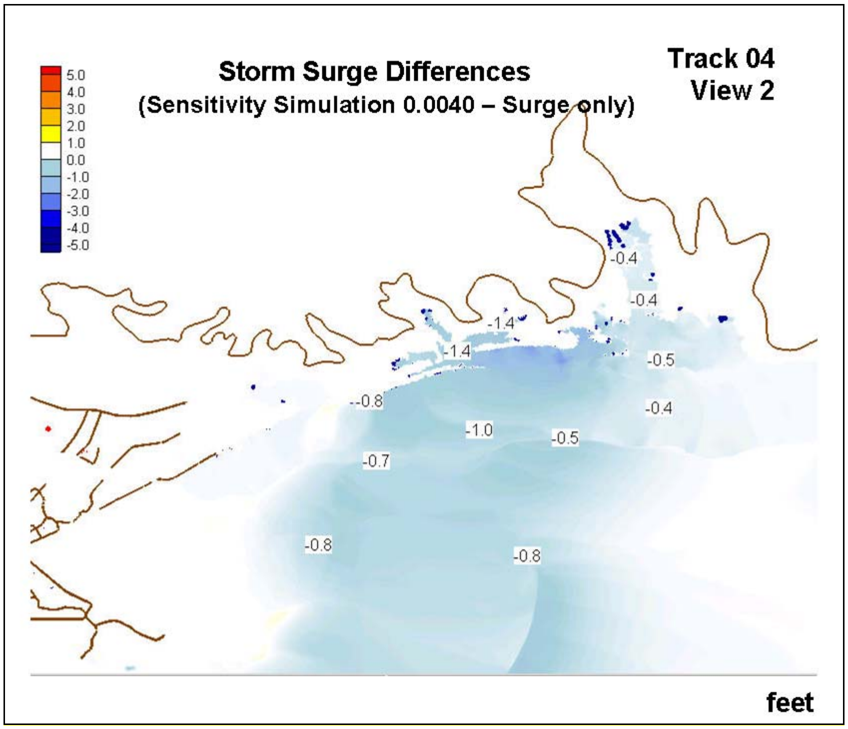

Figure 17. Peak Surge Elevation Difference Applying 0.0040 Drag Coefficient Cutoff.

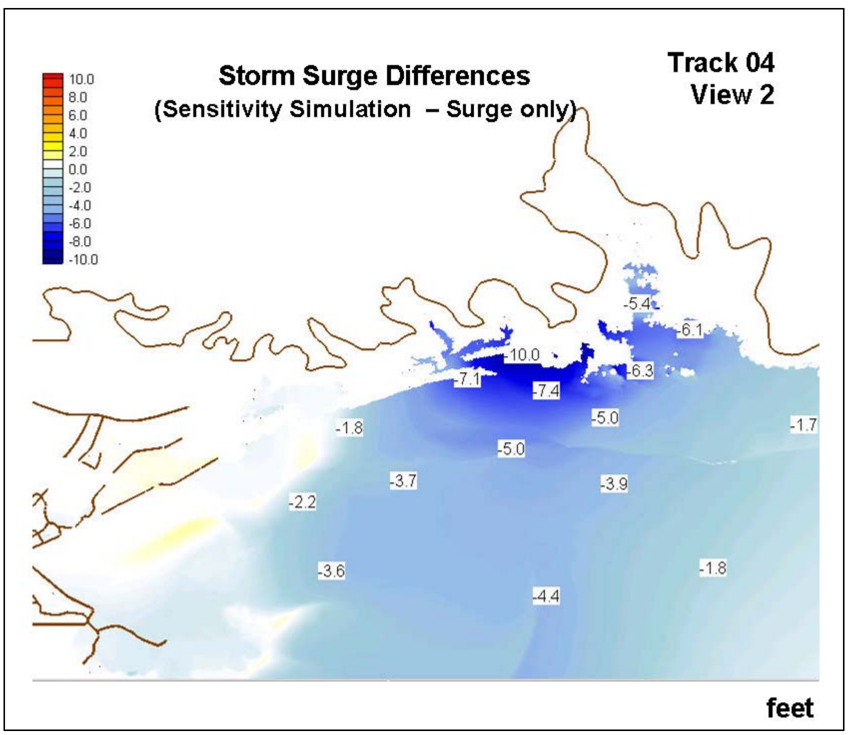

Figure 18. Peak Surge Elevation Difference Applying 0.0025 Drag Coefficient Cutoff.

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WETLANDS, LANDSCAPE FEATURES, AND STORM SURGE: A REVIEW OF STUDIES TO DATE IN MISSISSIPPI

Introduction

As hurricanes and extratropical storms approach the coast, four storm-related phenomena can occur to modify local water levels: setup due to wind, low barometric pressure, set up due to wave forcing, and rainfall (Harris 1963). Storm winds force water towards the coast and typically create the greatest change in local water elevation. During hurricanes, winds create a positive storm surge on the right side of hurricanes in the Northern Hemisphere and negative surge on the left (Figure 1). Low barometric pressure provides a secondary effect, creating a bell-shaped bulge in the water surface that is symmetrical around the center of the storm. Wave forcing also creates a local setup on the coast, with highest waves on the right side of a hurricane in the Northern Hemisphere. A lower magnitude of wave setup may also occur on the left side of the storm, depending on the path, speed, and strength of the storm. Rapid storm rainfall can also increase the local water elevation. A fifth factor not related to the storm itself is the astronomical tide at the time the storm reaches the coast; a spring (high) tide occurring at the time of the storm will result in greater storm inundation than if the storm made landfall during a neap (low) tide.

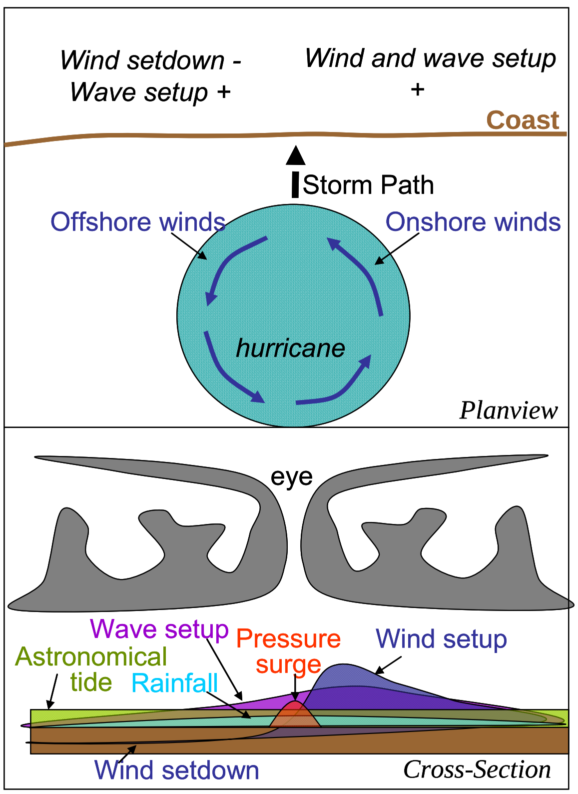


Figure 1. Storm Surge Components for Hurricane Landfall in Northern Hemisphere (Relative Magnitudes of Surge are Hypothetical)

Bathymetry and topography also modify the storm surge. A mildly sloping continental shelf, such as in the Gulf of Mexico, results in a higher storm surge as compared to a coast with a steeper bathymetry. The reason for this is, in deeper water, the surge can disperse downwards, whereas in shallow water, it cannot and is pushed inland by wind stresses. However, a milder slope reduces wave height as waves dissipate further offshore as compared to the steeper bathymetry.

Topography, landscape features, and vegetation also have the potential to reduce storm surge elevations. Land elevations greater than the storm surge elevation provide a physical barrier to the surge. Landscape features (e.g., ridges and barrier islands) and vegetation (e.g., maritime forests and wetlands) are typically below the surge elevation, but they have the potential to create friction and slow the forward speed of the storm surge. The surge then has time to dissipate offshore and alongshore, reducing inland surge elevations.

The purpose of this literature review is to document studies that have measured storm surge elevations with the goal of understanding how landscape features and vegetation modify the surge elevation. Numerical modeling studies of this phenomenon are also reviewed. As illustrated in Figure 1, many factors control the elevation of the surge. To best characterize the influence of landscape features and vegetation on storm surge, ideal measurements are those that are (1) in line with the path of the storm, (2) on the same side of the storm, (3) not so far apart that processes (e.g., barometric pressure, winds, rainfall) are significantly different, (4) inside an enclosed space, to remove the influence of wave height on the measurements, and (5) representative of a homogeneous landscape feature (Figure 2). Information from the literature review is culled and near-ideal measurements and studies are identified to isolate the influence of these landscape features on storm surge elevations.

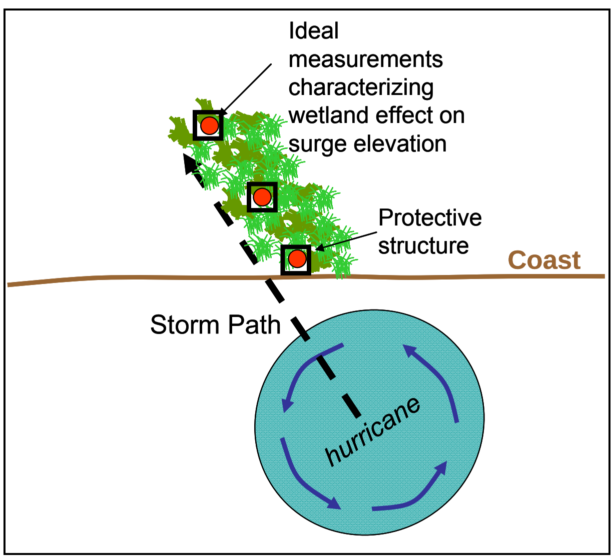


Figure 2. Ideal Measurements for Isolating the Influence of Landscape Features On Storm Surge Elevations

History of Storms Impacting the Mississippi Coast

The historical record of hurricanes and tropical storms brushing (within 96.6 km (60 miles) or making landfall at three cities on the Mississippi Gulf coast is shown in Table 1. This record of more than 130 years indicates that the Mississippi Gulf coast has experienced a tropical storm or hurricane approximately every 4 years, with a direct hurricane hit occurring every 10 to 17 years. The frequency of storms brushing the Mississippi coast is roughly equal for all locations, whereas hurricane landfall has occurred more frequently on the eastern part of the coast. Table 2 shows the storm path, intensity, and identifies the landfall location for most storms listed in Table 1.

In an initial review of literature, maximum storm surge elevations due to the 1947 Hurricane (Sanders 1947), Hurricanes Betsy (U.S. Army Engineer District (USAED) New Orleans 1965, and USAED Mobile 1967), Camille (USAED Mobile 1970), Georges (USAE South Atlantic Division 1999), Ivan (USAED Mobile 2004), and Katrina (NOAA 2005) have been detailed for the Mississippi coast. These data are listed in Table A1, and are being incorporated into a Geographic Information System (GIS) of landscape features

Table 1.
Storms within 96.6 km (60 mi) of Mississippi Cities 1872 through 2005*

Location	Storms t=tropical storm; b=brush; h=hurricane	Frequency of Occurrence (yr) Brush or hit	 Direct Hit
Gulfport	1872t,1879b,1881b,1885t,1885tb,1887t,1892t,1893h, 1895t,1900t,1901b,1904tb,1905tb,1906h,1907tb,1912b, 1914tb,1916h,1923t,1926t,1932b,1934tb,1944t,1947h, 1947t,1955tb,1960t,1965b,1969h,1979b,1985h,1988b, 1998h,2002tb,2002t(2),2004b,2005t,2005h	4	17
Biloxi	1879b,1880b,1881t,1885t,1885tb,1887t,1892tb,1893h, 1895h,1900t,1901h,1906h,1907tb,1912h,1916h,1923t, 1926h,1932h,1934tb,1947h,1955tb,1960t,1969h,1985h, 1997b,1998h,2002t,2002tb,2004b,2005t,2005h	4	11
Pascagoula	1872b,1881t,1885t,1885tb,1887t,1893h,1893b,1895t, 1900t,1901h,1902tb,1904tb,1906h,1912h,1914tb,1916h,1923tb, 1926h,1932h,1934tb,1944tb,1947b,1950b,1960b,1969h,1979h, 1985h,1998h,2002t,2004h,2005t,2005h	4	10

* http://www.hurricanecity.com/

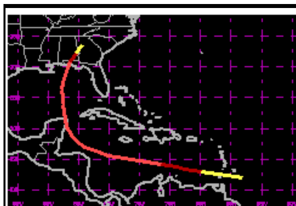
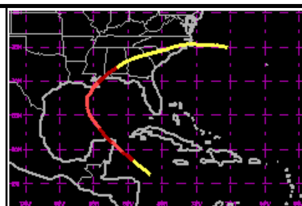
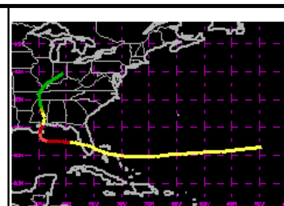
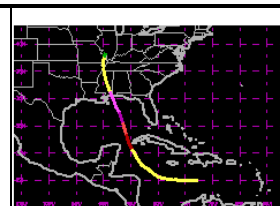
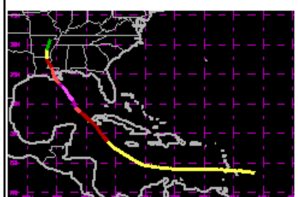
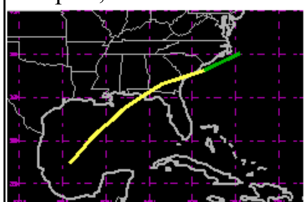
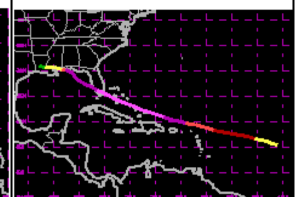
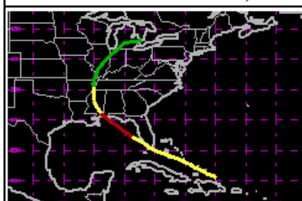
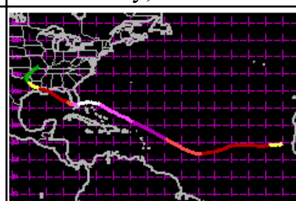
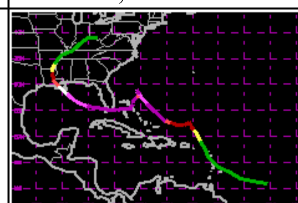
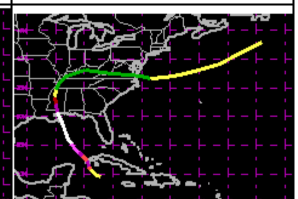
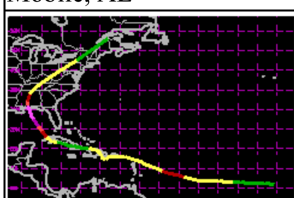
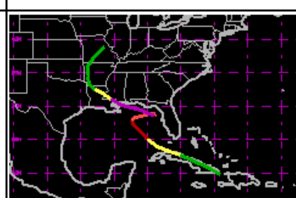
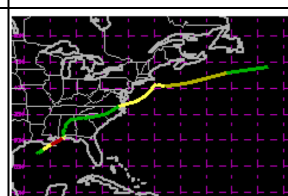
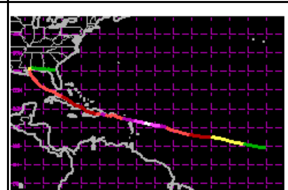
and vegetation for the Mississippi and Louisiana coasts. More storm surge elevation data may be available and will be incorporated into the GIS database as acquired.

Existing Relationships

Relationships documenting the reduction in storm surge elevation due to landscape features and vegetation have been determined based on limited measurements in Louisiana. In this initial review of the literature and studies completed to date, no relationships have been found based on measurements in Mississippi. Thus, a review of the studies based on Louisiana measurements is given as preliminary guidance. As more literature is reviewed and data are acquired, it is anticipated that more robust relationships will be developed.

In a Letter from the Chief of Engineers (1965) documenting an interim hurricane survey of Morgan City and vicinity, Louisiana, measurements of high water marks due to hurricane surge were correlated with distance inland from the coast. Surge elevations at 16 locations near Morgan City due to seven hurricanes (Sep 1909, Aug 1915, Sep 1915, Aug 1926, Sep 1947, Sep 1956, and Jun 1957) were documented giving 42 data points (Figure 3). The report states that this area has numerous bays and marshes, but the data evaluated include the western part of Louisiana with cheniers (relatively high wooded ridges). Inconsistent results were obtained when attempting to correlate hurricane translation speed, surge hydrograph at the coast, and surge elevations inland. However, a trend was observed for the decrease in storm surge as a function of distance inland, and is independent of hurricane translation speed, wind speed, and direction. The relationship indicates that storm surge was reduced by 1 foot for every 2.75 miles inland (1 cm decrease in storm surge per 145 m inland).

Table 2.
Storm Path for Hurricanes and Tropical Storms of Significance in Mississippi*
Date, Name (if available), and Location of Landfall

 July 20-28, 1887 E. Choctawatchee Bay, FL	 October 1-12, 1893 Gulfport, MS	 August 4-8, 1901 Perdido Key, FL	 September 24, 1906 Mobile, AL
 September 14, 1909 South of New Orleans, LA	 September 27-29, 1907 Panama City, FL	 June 29, 1916 Pensacola, FL	 September 11, 1926 Mobile, AL
 August 26-September 4, 1932; Mobile, AL	 September 19, 1947 New Orleans, LA	 August 27-13, September, 1965; Betsy; MS Delta, LA	 August 17, 1969 Camille Bay St. Louis, MS
 August 28, 1979 Frederic West of Dauphin Is., AL	 September 2, 1985 Elena Ocean Springs, MS	 July 16-27, 1997 Danny Mobile Bay, AL	 September 28-30, 1998; Georges Ocean Springs, MS
 September 16, 2004 Ivan Near Gulf Shores, AL	 September 29, 2005 Katrina Burras-Triumph, LA	* From http://www.eglin.af.mil/weather/hurricanes/history.html and http://en.wikipedia.org/. Storm paths are not available for all storms listed in Table 1. Key for storms through 1998: green=tropical depression; yellow = tropical storm; maroon = Cat 1; red = Cat 2; purple = Cat 3; pink = Cat 4; white = Cat 5. Key for Ivan and Katrina: blue = tropical depression; turquoise = tropical storm; light yellow = Cat 1; yellow = Cat 2; gold = Cat 3; orange = Cat 4; red = Cat 5.	

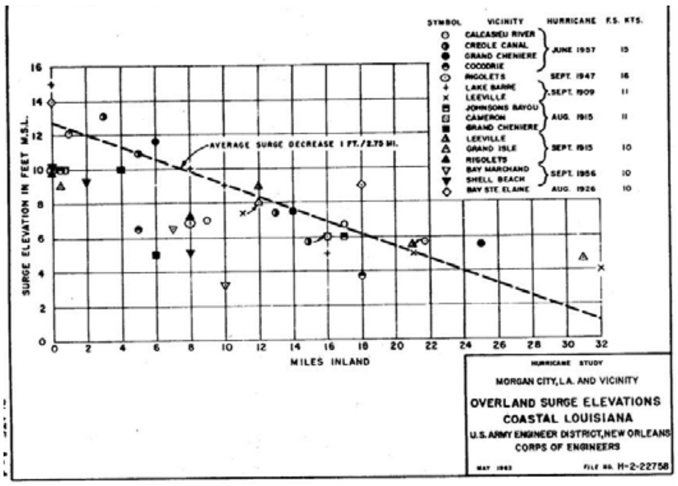


Figure 3. Observed Maximum Surge High Water Marks Versus Distance Inland (USACE 1965)

Lovelace (1994) documented storm surge elevations after Hurricane Andrew in Louisiana. These data are being compiled into a GIS for future reference. Citing this study, the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004) suggest that storm surge reduces about 3-inch (0.25 ft) per mile (1 cm per 211 m) of marsh along the central Louisiana coast.

Stone et al. (2003) modeled a Category 3 hurricane that made landfall in 1915 and compared wave and storm surge for the south-central Louisiana coast in 1950 (1.09 million acres of land) to that in 1990 (0.85 million acres of land). Models used were a hurricane planetary boundary model, ADCIRC circulation model, and SWAN wave model. Acreage impacted by a 2.1 m (7 ft) surge and 3.7 m (12 ft) increased to 69,000 and 49,000 acres, respectively, between 1950 and 1990 (Figure 4). Surge levels greater than 4.6 m (15 ft) were not significantly different between the two time periods.

The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004; Chapter 6, p. 55) discuss that it is “commonly acknowledged that barrier islands and wetlands reduce the magnitude of hurricane storm surges and related flooding; however, there are scant data as to the degree of reduction.” At the time the report was written, the best information documenting this phenomenon came from gages measuring water elevations during the second landfall of Hurricane Andrew (data documented by Lovelace 1994), which occurred in the vicinity of Point Chevreuil, Louisiana on August 26, 1992. Gage data from Cocodrie, Louisiana indicated a maximum water level elevation equal to 9.3 ft (2.8 m) during this Category 3 Hurricane. Over a 23-mile (37 km) stretch of marsh and open water from Cocodrie to the Houma Navigation Canal, the water elevation decreased from 9.3 ft (2.8 m) to 3.3 ft (1 m), equating to a reduction in surge amplitude equal to 3.1 inch (0.26 ft) per mile of marsh and open water (1 cm per 203 m). A similar set of measurements showed reduction of the storm surge from 4.9 ft (1.5 m) at Oyster Bayou to 0.5 ft (0.15 m) at Kent Bayou, located 19 miles (30.6 km) north. This second set of measurements indicated 2.8-inch (0.23 ft) decrease in surge per mile (1 cm per 230 m) over “fairly solid marsh.” The report cautions that these represent measurements from one storm; other factors, such as storm characteristics, coastal geomorphology, and track of the storm influence the degree to which wetlands decrease storm surge.

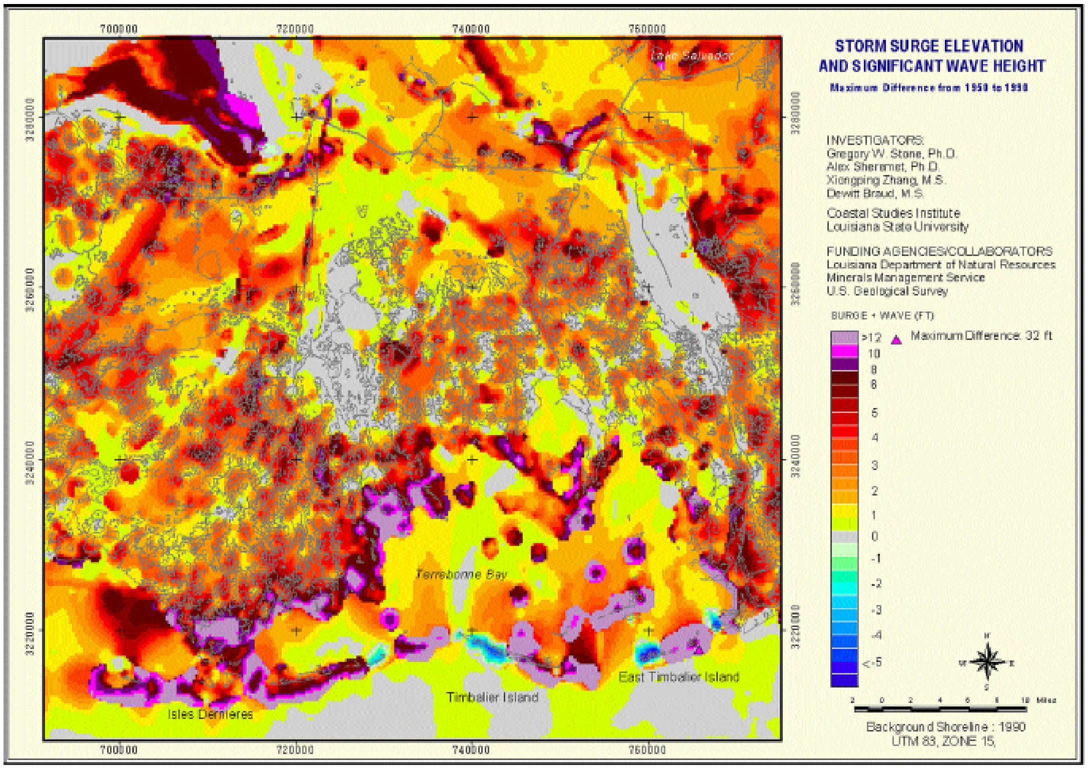


Figure 4. Difference Between Maximum Surge Plus Significant Wave Height, 1950 to 1990 (Stone Et Al. 2003)

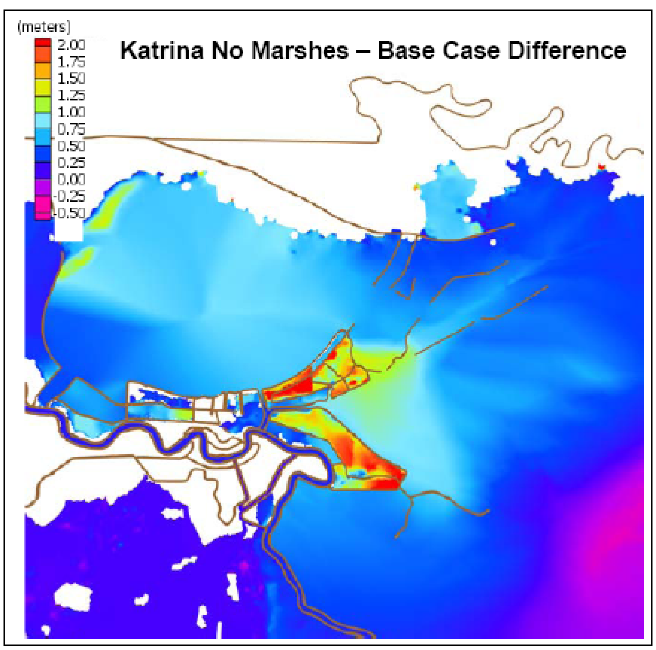


Figure 5. Differences In Computed Storm Surge For Hurricane Katrina With The Disappearance Of Wetland Landscapes East Of The Mississippi River Gulf Outlet (Working Group For Post-Hurricane Planning For The Louisiana Coast 2006)

The Working Group for Post-Hurricane Planning for Louisiana Coast (2006) wrote “barrier islands, shoals, marshes, forested wetlands and other features of the coastal landscape can provide a significant and potentially sustainable buffer from wind wave action and storm surge generated by tropical storms and hurricanes.” ADCIRC results from Rick Luettich (Dec 30, 2005) indicated if wetlands east of the Mississippi River Gulf Outlet (MRGO) were removed and the lake was deepened to 2.5-m (8-ft), the storm surge from Hurricane Katrina would increase by 1-2 m (3-6 ft) for St. Bernard Parish and Eastern New Orleans (Figure 5).

Engineering Relationships

Overview. Coastal landscape features affect the intensity and spatial patterns of storm winds, currents, waves, and water levels. These landscape features include wetlands, barrier islands, interior landscape ridges, navigation channels, bays and estuaries. This section presents a preliminary review of the engineering literature about the quantitative relationships between coastal landscape features and the characteristics of hurricane storms. The effects of each landscape feature on each of the hurricane storm characteristics are reviewed.

Wetlands contain a variety of vegetation types. The physical properties of wetlands that modify storm characteristics include the vegetation type, location, height and density. Vegetation has an effect on storm waves. Waves become depth limited, not fetch limited, over relatively short distance if the friction factor is high enough. Wind stress is also affected by land cover. The sediment geotechnical properties and morphology of each wetland can modify wave height and direction.

Barrier islands and interior landscape ridges modify storm surge as a function of location, elevation, width, vegetation cover, and foreshore slope. The degree to which a barrier island decreases storm surge elevation depends on whether the island is overtopped and if the adjacent tidal inlet cross sectional area is in equilibrium with the bay tidal prism. Inlet parameters include location, cross sectional area, depth, width, and frictional roughness.

Navigation channels are anthropogenic features that affect the landscape hydrology by their location, length, depth, width and roughness. Bays and estuaries affect bottom friction through their location, depth, bottom roughness, and bottom sediment shear strength. Suspended mud or a muddy seabed in the bay or estuary increases the rate of wave energy dissipation.

Winds. The strength and impact of hurricane winds in coastal areas is affected by landscape features in two distinct manners. First, the intensity of hurricane storms undergoes a significant decrease in intensity after landfall. Data suggest that this process, referred to as “filling,” is initiated before the eye of the storm crosses over land. The filling gradually reduces the wind velocity within the storm. The rate of wind speed reduction has been related to the numbers of hours after landfall and to the geographic region (NWS 23 1979). This rate of reduction is of highest category for the Mississippi coast, showing a reduction of the wind speed of about 15% at 5 hours after landfall and a reduction of about 30% at 10 hours after landfall.

Landscape features also affect hurricane winds because vegetation which extends above the water surface, both before and during flooding, reduces the speed of the wind at the water surface. This reduction in wind speed translates to a reduction in the wind stress which generates both storm waves and surges. The reduction in wind stress due to the presence of vegetation has been described with a “stress reduction factor” or SRF (Federal Emergency Management Agency (FEMA) 1985). The SRF is affected differently by various land covers and the most important contribution is the areal distribution of the various land covers.

Wooded areas have the greatest effect, with the type, height and density of the trees being of primary importance. The SRF may be as low as 0.10, indicating a 90% reduction of the open water wind stress. The SRF for wooded areas is related to the fractional projected area of the trees. This fractional area is the area of the trees divided by the total flow area, with both areas being projected on a vertical plane perpendicular to the wind velocity. The effect of trees on the SRF is not linear. For a fractional projected area of 10% the SRF is 0.85, while for 40%, the SRF is 0.30. The effect decreases with higher fractional areas. At fractional areas equal to 60% and 80%, the SRF is 0.20 and 0.10, respectively.

Marsh grasses also affect the SRF, although this effect is very complex. Overall marsh grass has a smaller roughness than wooded areas, and has a smaller effect on wind velocity. Marsh grass is quite flexible and can be blown over during the hurricane. Also the marsh grasses can become inundated exposing the water surface to the full effect of the wind. The expected range in SRF for marsh is 0.70 to 0.90 with the higher value being used when the surge height is higher than the average height of the marsh grass.

A value for 0.30 for the SRF has been used successfully by the USGS in the SWIFT2D hydrologic modeling of coastal wetlands (Swain 2005). The value of SRF equal to 0.30 was used for all computational grids having a Manning's coefficient greater than 0.10, implying that the vegetation is emergent.

Open water near land can experience a reduction in the wind stress when the wind is blowing offshore. This "downwind sheltering effect" results from the modification of the winds surface boundary layer as it passes a land surface having high roughness. This effect may extend to a distance of 2 to 10 nautical miles from the upwind land, and would be particularly important behind barrier islands. The approach used by FEMA is to linearly increase the wind stress from the reduced overland value to the open water value over a distance of from 2 to 10 nautical miles.

Waves. Storm waves are affected by several coastal landscape properties. These properties include the water depth (before and during flooding), bottom roughness or friction, water column friction, and bottom sediment characteristics.

The effect of water depth on waves becomes fundamental as waves propagate into shallow water and controls wave kinematics and dynamics (U.S. Army Corps of Engineers 2003). Shallow water wave processes includes generation, shoaling, refraction, diffraction, reflection, breaking, setup, run-up, bottom friction, water column friction, and dissipation of wave energy through wave/bottom interaction. The water depth and variations in water depth associated with coastal landscape features become particularly important when they cause wave breaking. Wave breaking occurs when the still water depth equals about 78% of the wave height and involves intense energy loss and can, for example, reduce wave heights by 90% over a distance of 10 meters. Wave run-up and overtopping occur if the height of a barrier island or an interior ridge equals or is less than the still water elevation.

Bottom friction and wave/bottom interaction in shallow bays dissipates wave energy and can limit the height of waves to values considerably below the breaking criteria. This effect depends upon the type of bottom sediment in the bay. Muddy bottom sediments have a response that can involve actual motion of the bottom due to the elastic properties of clay and mud.

The wave energy loss through vegetation results from the drag force of the wave current on the plants (FIA 1984, FEMA 1988). The rate of energy loss depends upon the geometry of the individual plants and the density of the plants in a given area. For areas containing a variety of plant types, the number of plants of each type can be specified as the fraction of the total area covered by a plant type and the average number of plants per square foot in the fractional area. The total energy loss for all plants along a transect is the sum of the energy loss associated with all of the individual plant types. The time average energy loss, $E_{i,j}$ for all plants of all plant types is given by:

$$E_{i,j} = \frac{\int_0^T \int_0^{h_i} \left| F_{i,j} u \right| dz\, dt}{T} \qquad (1)$$

where z is the elevation, $F_{i,j}$ is the drag force for the j^{th} member of the i^{th} plant type, h_i is the height of the submerged plant or the wave crest height if the plant is exposed, u is the horizontal wave current, and T is the total time being evaluated. The drag force on each individual plant is given as:

$$F_{i,j} = \frac{\rho C_D D_{i,j} \left| u \right| u}{2} \qquad (2)$$

where ρ is the water mass density, C_D is the plant drag coefficient, and $D_{i,j}$ is the effective diameter of the j^{th} member of the i^{th} plant type. The drag coefficient generally varies with plant roughness and the Reynolds number, but is taken as 1.0 for most plants. The contribution from the flat parts of the plant leaves is generally ignored.

The growth or decay of wind waves propagating over vegetated areas can estimate the effects of high friction by adjusting the fetch length (Camfield 1977). In this analysis the friction factors associated with vegetation can be up to 100 times the friction factor associated with unvegetated shallow water. The friction factor for various vegetation types are given as a function of water depth for thick stands of marsh grass; dense grass, brush or bushy willows and scattered tress; and dense stands of trees. Based upon a water depth of 3 m (10 ft), the friction factor for marsh grass is 0.20, for dense grass and brush it is 0.48 and for dense stands of trees, 0.90. These values represent an increase over the unvegetated bottom friction by factors of 20, 48, and 90, respectively. An example can be cited of the effectiveness of vegetated wetlands to dissipate wave energy (U.S. Army Corps of Engineers 2003). Storm waves having an initial height of 3 m (10 ft) are predicted to be reduced to a height of 1.5 m (4.8 ft) after passing over 1000 m (3300 ft) of tall grass and brush.

Currents and Storm Surge Elevation. Currents and surge are affected by coastal landscape features through two mechanisms. Bottom friction is the generated by fluid shear stresses on the water bottom, while flow-drag resistance is generated by fluid stresses on objects extending through the water column (FEMA 1985). Only bottom friction occurs in bays whereas bottom friction and flow-drag resistance can occur in vegetated areas.

The most widely used formulation of bottom friction for flow in shallow water is the Manning-Chezy formula,

$$\tau = \frac{g \left| U \right| u}{C^2}, \ and \ \ C = \frac{1.486 h^{1/6}}{N} \qquad (3)$$

where τ is the bottom stress, $|U|$ is the flow speed, u is the vector velocity, C is the Chezy coefficient, h is the flow depth, and N is the Manning's coefficient. The Manning's coefficient is not a constant and varies with water depth and bottom roughness. For bays the Manning's coefficient has been represented as an exponential function of the water depth, by the following formula (FEMA 1985),

$$N = A h^{-B} \qquad (4)$$

where A and B are curve fitting parameters. Calibration data for various studies indicate B is about 0.5 and A varies between 0.08 and 0.12, with a mean value of 0.10. This formula indicates the Manning's coefficient decreases as the water depth increases, with values of N of about 0.044 for a depth of 1.5 m (5 ft), 0.032 for a depth of 3 m (10 ft) and 0.022 for a water depth of 6 m (20 ft). Since the Manning's N is typically used as a tuning factor in calibrating hydrodynamic models, in this formulation *A* can be used for the same purpose. For flooded wetlands, the Manning's *N* is assumed to be a constant that varies with vegetation type. Table 3 gives the range of values of Manning's *N* for various vegetation types.

Table 3.
Estimated Values of Manning's Coefficient, N

	Manning's *N*		
Land Cover Type	**Minimum**	**Mean**	**Maximum**
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Scattered brush	0.035	0.050	0.070
Medium to dense brush	0.045	0.080	0.160
Marsh grass (0.3-1 m)	0.05	0.075	0.10
Marsh grass (1 – 2 m)	0.10	0.125	0.15
Marsh grass (>2m)	0.15	0.20	0.25

Flow-drag resistance also occurs in vegetated areas and represents flow resistance within the water column is a force that cannot be readily represented as a stress. Taking the approach that the flow-drag force on natural vegetation can be expressed as some the force on an equivalent cylinder, the total drag force for a given area of wetland can be given by

$$F_d = \frac{\rho C_d n D h_p V^2}{2} \qquad (5)$$

where F_d is the drag force, C_d is the drag coefficient for the cylinder, *n* is the total number of plants, *D* is the diameter of each cylinder, h_p is the height of the submerged part of the cylinder, and *V* is the flow velocity. The drag coefficient C_d is not a constant and depends upon the size and proximity of each plant. An equivalent stress can be defined as the total drag force over an area, divided by the size of the area.

An alternative representation of the drag force on a number of plants is based upon the Darcy-Weisbach formulation,

$$F_d = \frac{\rho f V^2}{8} \qquad (6)$$

where *f* is the Darcy-Weisbach resistance coefficient. This coefficient has been related to the "roughness concentration" given as

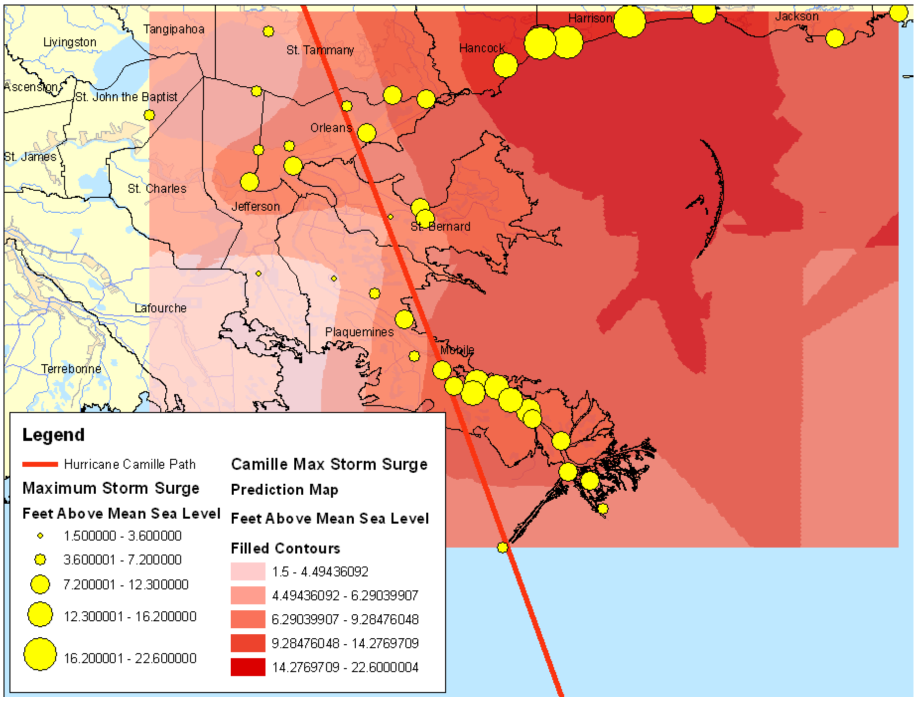
$$f = a\sigma^b, \, and \;\; \sigma = nDh_p \qquad (7)$$

where σ is the roughness concentration, and *a* and *b* are calibration parameters.

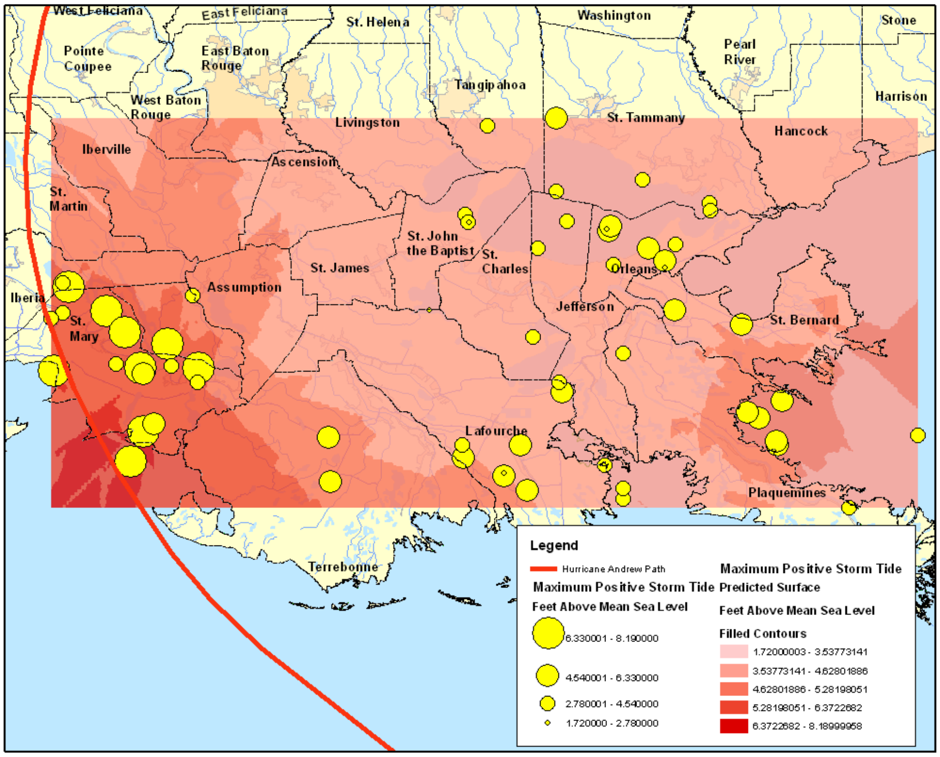
The effect of wetland vegetation density on the Manning's coefficient for overland flow was studied in a series of laboratory experiments (Hall 1994). The experiments involved placing bulrushes in various spatial densities in a 1.2 m (4 ft) wide channel and then subjecting them to discharges of 0.009, 0.026, 0.044 and 0.057 m^3/sec. The results of the tests indicated that for flow velocities in the range of 0.01 to 0.05 m/sec (0.03 to 0.16 ft/s), the Manning's *N* decreased as the average flow velocity increased, ranging about 0.3-0.9 at the lowest velocity to 0.2-0.3 at the highest velocity. A linear relationship was found between the density of plants and the Manning's *N*, with the value of *N* being about 0.6 for a density of 800 stems per square meter.

GIS Database

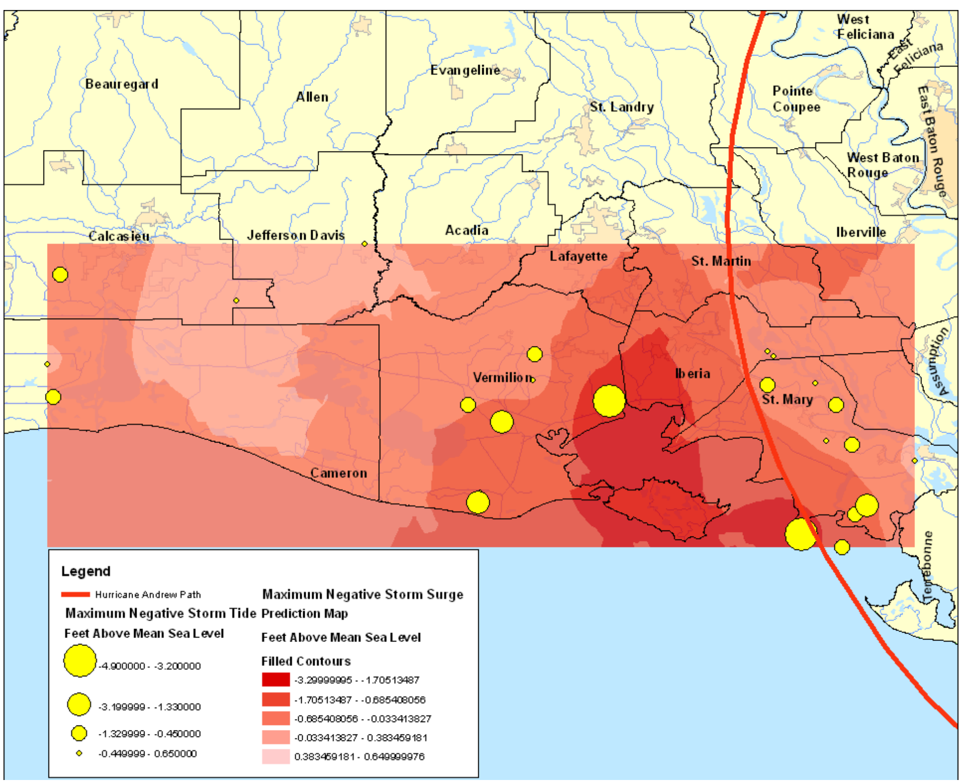
Measurements of storm surge elevation from Hurricanes Camille, Andrew, and Katrina have been incorporated into a GIS database. These data will be evaluated with the storm path and vegetation type to develop an understanding of how landscape features and vegetation modify storm surge elevation. Figure 6 shows preliminary contours of storm surge elevations measured during these hurricanes. Note that both positive and negative surge elevations were measured after Hurricane Andrew. More storm surge elevation data are available and will continue to be incorporated into the GIS.



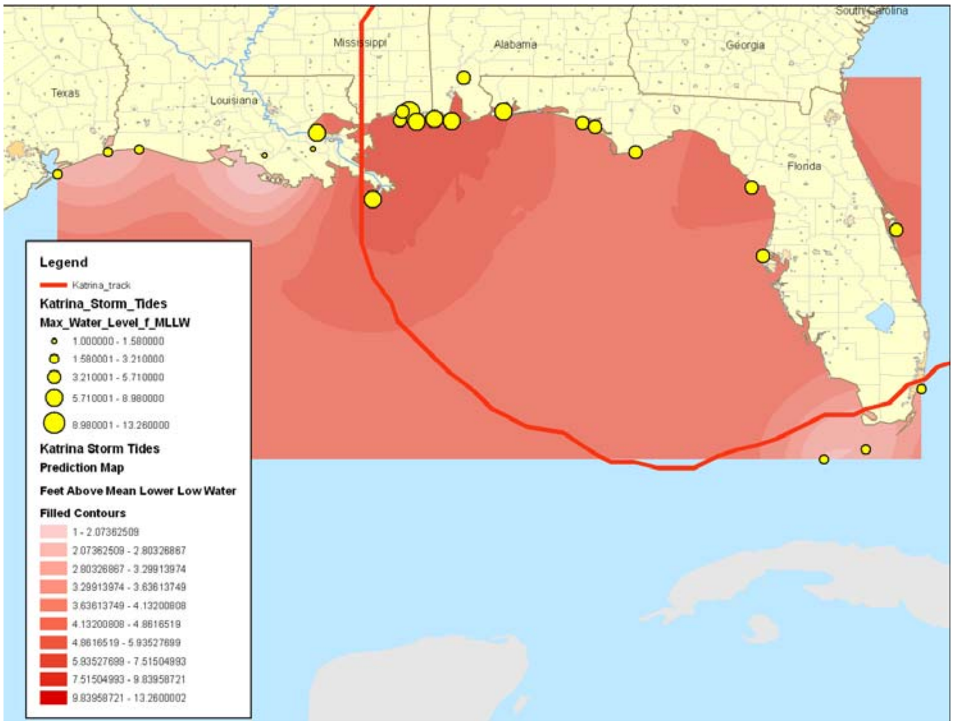
A. Hurricane Camille – Positive Storm Surge



B. Hurricane Andrew – Positive Storm Surge



C. Hurricane Andrew – Negative Storm Surge



D. Hurricane Katrina – Positive Storm Surge
Figure 6. Initial GIS Database

Preliminary Recommendations and Future Work

Table 3 summarizes the relationships that have been reviewed for storm surge reduction as a function of overland distance of landscape features and vegetation.

Table 3.
Relationships for Storm Surge Reduction as a Function of Overland Distance

Distance Required for 1 cm Reduction in Surge Elevation (m)	Landscape Feature	Database	Reference and Notes
145	Cheniers, marsh, bays	7 hurricanes, 42 data points	USACE (1965); data on which relationship is based may represent both sides of storm path (see Figures 1 and 2 herein)
211	Central LA coast (assumed to be marsh and open water)	Hurricane Andrew; 2 data points	Lovelace (1994); more elevations are available and are being input to GIS database
203	Marsh and open water	Hurricane Andrew; 2 data points	The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004), based on Lovelace (1994)
230	“Fairly solid marsh”	Hurricane Andrew; 2 data points	

Based on this preliminary review, it appears as if a conservative estimate is 1 cm reduction in storm surge elevation for every 200-250 m of marsh. However, the location of each of these data points relative to the storm path and the quality of each data point must be evaluated. For example, data may be located on either side of the storm track and thus changes in elevation may represent differences in forcing conditions rather than a reduction in surge due to presence of a landscape feature. Elevations of the data point are also suspect as datums in the region have shifted through time. More recent measurements from Hurricanes Andrew and Katrina with more accurate datum control and broader coverage are available to infer relationships.

In their study of the south-central Louisiana coast, Stone et al. (2003) indicated that storm surge elevations greater than 4.6 m (15 ft) were not affected by changes in the landscape. From this finding, we might expect that the influence of *submerged* landscape features would decrease as storm surge increases. Landscape features only partially submerged would provide more resistance and thus reduce surge until they are submerged. Future work will continue with the literature review, conduct idealized numerical modeling tests to evaluate the reduction in surge as a function of landscape feature and vegetation type. A surge elevation database is being developed within a GIS for determining relationships based on available measurements, as well as for comparison with numerical modeling results.

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Table A1.
Maximum Storm Surge Elevation Measurements
(ongoing effort; this table represents a partial draft)

Storm	Location	Elevation (ft)	Reference and Datum
Unnamed Hurricane, 19 Sep 1947	Burrwood, LA	4	Sanders (1947) (no datum given, assumed to be mean sea level)
	Chandeleur Light, LA	14	
	Morgan City, LA	6	
	Bay St. Louis, MS	12	
	Biloxi, MS	12	
	Gulfport, MS	12	
	Pascagoula, MS	12	
Hurricane Betsy, 8-11 Sep 1965 (Partial listing of measure-ments) s=still high water mark; g=gage	Pascagoula, MS (g)	6.4	U.S. Army Engineer District, New Orleans (1965) (Mean Sea Level)
	Biloxi, MS (g)	8.6	
	Gulfport (s)	10.7	
	Pearlington, MS (s)	8.8	
	Lake St. Catherine (west side), LA (s)	10.6	
	Opening to Lake Pontchartrain (g)	7.0	
	Slidell, LA (s)	6.7	
	Lacombe, LA (s)	5.8	
	Pontchartrain Causeway (north), LA (g)	6.5	
	Lake Pontchartrain (north side), LA (s)	5.1	
	Lake Pontchartrain (west side), LA (s)	10.2	
	Pontchartrain Causeway (central), LA (g)	5.5	
	New Orleans on Lake Pontchartrain, LA (g)	5.0	
	New Orleans on east side, LA (s)	5.3	
	Algiers, on MS River (g)	12.6	
	Shell Beach, LA (s)	9.3	
	Yscloskey, LA (g)	11.7	
	Delacroix, LA (s)	11.0	
	Phoenix, LA (g)	8.3	
	Between Phoenix and Pointe a la Hache Bohemia (northeast side of river), LA (s)	8.8, 9.8, 11.9, 10.7 (from north to south)	
	Pointe a la Hache Bohemia, LA (s)	14.4	
	South of Port Sulphur, LA (s) (northeast and southwest sides of river, respectively)	13.7, 5.7	
	Empire, LA (s)	7.4	
	Ostrica (north side of river), LA (s)	13.6	
	Buras, LA (s)	7.7	

Table A1.
Maximum Storm Surge Elevation Measurements (continued)

Storm	Location	Elevation (ft)	Reference and Datum
	Between Ostrica and Venice (south side of river), LA (s)	14.6	
	Venice, LA (s)	8.8	
	Head of Passes, LA (s)	6.6	
	Garden Island Bay, LA (s)	7.7	
	Port Eads, LA (g)	5.2	
	Burrwood, LA (s)	5.5	
	Grand Isle, LA (g)	8.8	
	Leeville, LA (g)	5.4	
	Pascagoula, MS (s)	6.4	U.S. Army Engineer District, Mobile (1967) (mean sea level)
	Biloxi, MS (s)	8.6	
	Gulfport, MS (s)	10.7	
	Long Beach, MS (s)	12.3	
	Pass Christian, MS (s)	10.8	
	Waveland, MS (s)	12.7	
	Clermont Harbor, MS (s)	12.0	
	Pearlington, MS (s)	8.8	
	Bay St. Louis, MS (seawall) (s)	12.5	
	St. Louis Bay, MS (s)	11.2	
Hurricane Camille, 17-18 Aug 1969; s=still high water mark; g=gage; d=debris line	Alabama border (s)	9.2	U.S. Army Corps of Engineers, Mobile District (1970, Plate 6) (mean sea level)
	Pascagoula, MS (s)	11.4	
	Biloxi, MS (s)	15.5	
	Gulfport (s)	21.0	
	Bay St. Louis (east side), MS (s)	22.6	
	Bay St. Louis (west side), MS (s)	21.7	
	Clermont Harbor, MS (d)	16.2	
	Lake St. Catherine (east side), LA (s)	12.3	
	Opening to Lake Pontchartrain (g)	9.0	
	Pontchartrain Causeway (north), LA (g)	4.6	
	Lake Pontchartrain (west side), LA (g)	4.6	
	Pontchartrain Causeway (central), LA (g)	4.1	
	New Orleans on Lake Pontchartrain, LA (g)	5.2	
	New Orleans on canal, LA (g)	6.5	
	Canal confluence with MS River, LA (s)	10.2	
	MS River near New Orleans, LA (g)	10.8	
	Between Lake Borgne and Lake Pontchartrain, LA (g)	8.7	
	Shell Beach, LA (g)	11.1	
	Yscloskey, LA (d)	2.6	

Table A1.
Maximum Storm Surge Elevation Measurements (continued)

Storm	Location	Elevation (ft)	Reference and Datum
	End of Hwy. 46, LA [s]	8.9	
	Barataria, LA [g]	1.5	
	Phoenix, LA [d]	2.6	
	Between Phoenix and Pointe a la Hache Bohemia (northeast side of river), LA [d]	5.4	
	Pointe a la Hache Bohemia, LA [d]	11.0	
	Port Sulphur, LA [s]	5.2	
	Empire, LA [s]	10.9	
	Ostrica (north side of river), LA [s]	15.9	
	Buras, LA [s]	13.4	
	Between Ostrica and Venice (south side of river), LA [s]	14.6	
	Venice (north), LA [d]	15.9	
	Venice (south), LA [s]	9.1	
	Head of Main Pass, LA [s]	10.7	
	Head of Passes, LA [s]	12.0	
	Garden Island Bay, LA [s]	9.0	
	Port Eads, LA [s]	5.2	
	Burrwood, LA [g]	5.0	
	Grand Isle, LA [d]	3.6	
	Leeville, LA [g]	2.1	
	Pearlington, MS	5.2	
Hurricane Georges, 1988	Waveland, MS	6.6	U.S. Army Corps of Engineers, South Atlantic Division (1999) (NGVD 1929) See Figure A1
	Bay St. Louis, MS	5.8	
	Pass Christian, MS	7.9	
	Gulfport, MS	7.1	
	Biloxi – Pt. Cadet, MS	8.1	
	Biloxi – Back Bay, MS	8.3	
	Belle Fontaine Point, MS	11.0	
	Pascagoula – Hwy 90, MS	8.1	
	Pascagoula, MS	8.4	
	Pascagoula – MS Sound, MS	10.8	
	Pascagoula – Bayou Chico, MS	9.6	
	Bayou La Batre, AL	8.3	
	Dauphin Island, Gulf, AL	6.6	
	Dauphin Island, Bay, AL	5.0	
	Mobile Bay, Hollingers Island, AL	8.4	
	Downtown Mobile, AL	8.3	

Table A1.
Maximum Storm Surge Elevation Measurements (continued)

Storm	Location	Elevation (ft)	Reference and Datum
	Mobile Bay – Causeway, AL	9.4	
	Weeks Bay, AL	6.5	
	Fort Morgan – Bay, AL	6.4	
	Pine Beach -- Bay, AL	8.5	
	Pine Beach – Gulf, AL	10.8	
	Gulf Shores, AL	9.5	
	Perdido Pass, AL	5.6	
	Ono Island, AL	5.4	
	Pensacola, FL	6.4	
	Pensacola Beach, FL	7.7	
	Gulf Breeze – Santa Rosa Sound, FL	4.5	
	Navarre – Santa Rosa Sound, FL	4.5	
	Choctawhatchee Bay, FL	5.2	
	Destin Harbor, FL	4.6	
	Grayton Beach, FL	4.6	
	Panama City Beach, FL	5.1	
	Panama City Harbor, FL	3.5	
	Port St. Joe – North side, FL	2.3	
	Apalachicola Bay, FL	4.5	
	Carrabelle, FL	4.6	
Hurricane Ivan, 2004 (partial listing of measure-ments; some are repetitive and others outside study area)	Miss. Sound at Waveland, MS	4.56	U.S. Army Engineer District, Mobile http://chps.sam.usace.army.mil/USHESdata/Assessments/2004Storms/Ivan/slosh/table_1.htm (NGVD)
	Gulfport Harbor at Gulfport, MS	4.63	
	Mississippi Sound at Ship Island, MS	5.15	
	Biloxi Bay at Point Cadet, MS	4.23	
	W. Pascagoula river at Hwy. 90 at Gautier, MS	4.10	
	Pascagoula river at Pascagoula, MS	6.72	
	Miss. Sound at Pascagoula PI – Rear Range	5.83	
	Miss. Sound at Petit Bois Island	4.83	
	Escatawpa River at I-10 near Orange Grove	3.93	
	Middle Gage at Bayou LaBatre	4.66	
	Mobile Bay at Cedar Point, AL	6.90	
	Dauphin Island Bay at Dauphin Island	7.80	
	Mobile Bay at Dauphin island	8.00	
	Mobile River at Bucks, AL (Barry Steam Plant)	6.82	
	Mobile River at Mobile, AL	4.87	

Table A1.
Maximum Storm Surge Elevation Measurements (continued)

Storm	Location	Elevation (ft)	Reference and Datum
Hurricane Katrina, 2005 *sensor malfunction did not record max **sensor malfunction at higher water levels	Waveland, MS*	8.98	NOAA (2005) (Mean Lower Low Water)
	Pilots Station, SW Pass, LA	7.75	
	Pensacola, FL	6.69	
	Dauphin Island, AL	6.37	
	Horn Island, MS*	6.23	
	East Bank, LaBranch, LA*	6.12	
	Grand Isle, LA**	5.71	
	Panama City Beach, FL	4.34	
	Biloxi, MS*	4.32	
	Lower Bryant Landing, AL**	3.89	
	Panama City, FL	3.83	
	Panama City, FL	3.83	

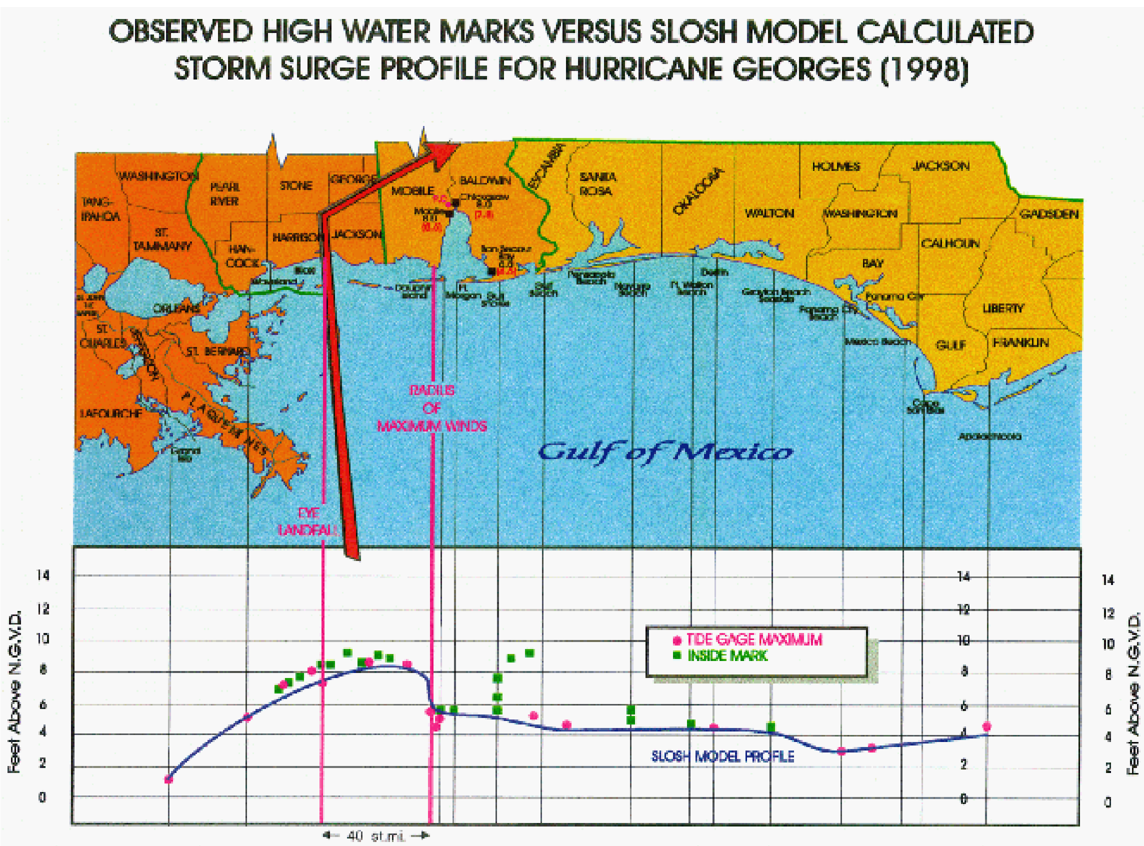


Figure A1. High Water Marks From Hurricane Georges And Predicted With SLOSH Model (Sea, Lake, And Overland Surges From Hurricanes; Jelesnianski Et Al. 1992) http://Chps.Sam.Usace.Army.Mil/Ushesdata/Assessments/Georges/Chapter%202.Htm

Engineering Appendix

III. Interim Engineering Solutions

BAYOU CADDY

Purpose

The purpose of this document is to provide engineering information for analysis and design as related to ecosystem restoration at Cadet Bayou (referred to as Bayou Caddy) in Hancock County, Mississippi.

Location

The proposed project site is located along the shoreline of Mississippi Sound in Hancock County, Mississippi, south and west of the Federally authorized Bayou Caddy navigation project (See Figure 1). The Bayou Caddy area is an exposed shoreline facing to the north and east. The north terminus of the project site is the entrance channel to Bayou Caddy. The entrance channel extends from the -8 feet mean lower low water (MLLW) contour in the sound for a distance of about 7,800 feet to the mouth of the bayou. The shoreline and adjacent area of Bayou Caddy consists mostly of marshland. A map of the area showing the Federal project is shown as Figure 1. A photograph of the proposed project site area is shown as Figure 2.

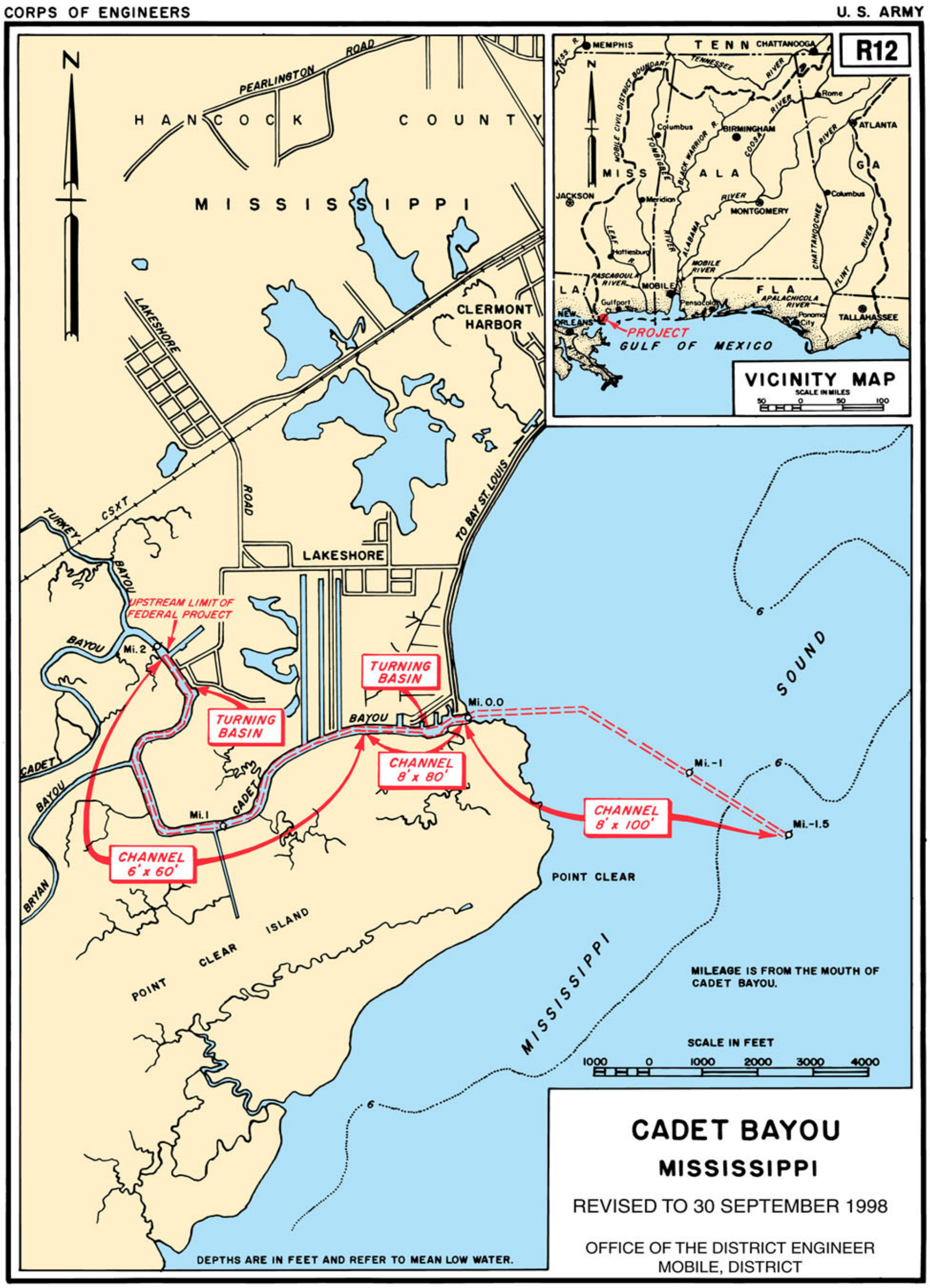


Figure 1. Project Map

Figure 2. Proposed Project Site

Existing Conditions

Tidal marsh borders the estuarine and adjacent waters in Mississippi and provides natural protection from the wave and wind energy. Erosion from wave attack under average conditions, coupled with hurricanes and other storms in the area, have undermined and eroded the marsh habitat at the proposed project site. Concrete seawalls armor the shoreline further to the north and east, and a large section of the Mississippi mainland. Sediment budgets are supplemented in these areas by periodic replenishment projects. Extensive areas of coastal wetlands located in western Hancock County are experiencing land losses due to erosion. Average rates of erosion in the Hancock County marshes are on the order of 12 to 13 feet per year over the past 70 years.

The erosion and disappearance of marsh habitat in Mississippi has exposed shorelines along both the mainland of Mississippi and its barrier island system to increased wave energy and accelerated erosion. In addition, the natural migration of the barrier islands alters the sheltering of these areas from erosive forces. Commercial and recreational fishermen also frequently use Bayou Caddy. As a result of this high level of boat activity and other natural erosive forces, the mouth and western face of the bayou are eroding and losing marsh. With the erosion of the western shoreline at Bayou Caddy, the area has become more prone to disturbance from waves, resulting in marsh habitat degradation.

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

Bayou Caddy is a tidal stream which empties into Mississippi Sound. The sound is a shallow coastal lagoon extending 80 miles along the coast of the Gulf of Mexico from Mobile Bay, Alabama westward to Lake Borgne, Louisiana. The average depth in the sound is 10 feet, and 99 percent of the sound is less than 29 feet deep.

Circulation patterns within the vicinity of the project site are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in Mississippi Sound is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The magnitude of normal tidal currents ranges from 0.5 to 1.0 feet per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. Freshwater discharge into Mississippi Sound comes primarily from the Pearl River and averages approximately 12,800 cubic feet per second (cfs). Wave heights in Mississippi Sound exceed 5 feet more than 20 percent of the time in winter, but only 5 percent of the time in summer. The project area has been impacted by several tropical storms and hurricanes, most recently from Tropical Storm Cindy, and Hurricanes Dennis and Katrina, all in 2005. Frequency estimates of stillwater storm tide elevations based on preliminary post-Katrina analysis of gage data at Biloxi, MS are shown in Table 1.

Table 1.
Storm Tide Frequency (feet, NGVD)

2-YR	5-YR	10-YR	20-YR	50-YR	100-YR
3.7	4.5	5.7	7.5	12.5	19.1

Geotechnical Data

A subsurface investigation that included the project site was made jointly by Law Engineering and GBA in November, 2001. The subsurface investigation consisted primarily of 51 probes made using ½" steel pipes with capped ends. Borings were not made and soil samples were not obtained. Soil classifications shown on the probing logs from this investigation were visually estimated based on soil coating or stains remaining on the outside of the probe pipe when it was removed from the ground. One probe (P-03) was made within the area enclosed by the piling alignment, one probe (P-04) was made very close to the alignment, and four other probes (P-01, P-05, P-25, and P-27) were made at locations such that they are somewhat likely to be representative of the subsurface conditions at the site. The depth of investigation was typically to refusal of the manually pushed probe, which limited the investigation to relatively shallow depths. The soil penetration depth in the previously identified six probes varied from 0.5 to 14 feet.

The ground surface is underwater, so all soils are saturated. The soil at the generally shallow depths in the 6 previously identified probes consists of silty clay (CL), sandy silt (ML), and silty and clayey sand (SM-SC and SP-SM). The thickness of this stratum varied from 0.5 to 14 feet, averaging 5.1

feet at the six probe locations. The top stratum generally appears to be underlain by fine sand (SP) of unknown thickness. However, other materials could also be present. Generally, the foundation at the site is very soft and consists of fine-grained sands, silts and clays, and presents some engineering challenges for construction of any recommended plan.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the proposed project site. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527. Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed project. Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project area. Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the area of the proposed project. Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations. The proposed project site has been severely impacted by hurricane-driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane-damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways. Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternative Plans

Four alternative plans for shore protection and marsh restoration and creation were previously evaluated in the August, 2003 Preliminary Restoration Plan prepared by the Mobile District. Each plan involved the use of concrete bridge rubble available from Hancock County as a result of their local construction project. The rubble was to be used to construct a breakwater as the outer perimeter for a containment dike structure at the proposed project site. The concrete rubble breakwater would protect the site from wave action, but could not contain dredged material from the Bayou Caddy channel. The preferred alternative in that report included an earthen dike as the containment structure. Following that construction, material from the next maintenance and/or new work dredging of the Bayou Caddy navigation channel would then be beneficially used to restore tidal marsh at the site. Wetlands would then be restored with vegetative plantings.

A permit has now been issued to others for their placement of about 25,000 cubic yards (CY) of concrete bridge rubble at the proposed project site. The rubble would be processed to remove all reinforcing steel. Since that construction would effectively function as a breakwater, three alternative inner containment structures have been evaluated for this report. A breakwater or other erosion protection measures along the east side of the project are necessary for protection of the containment dike or structure from erosion due to waves for each of the three considered alternatives. A plan view of the proposed alternatives is shown as Figure 3.

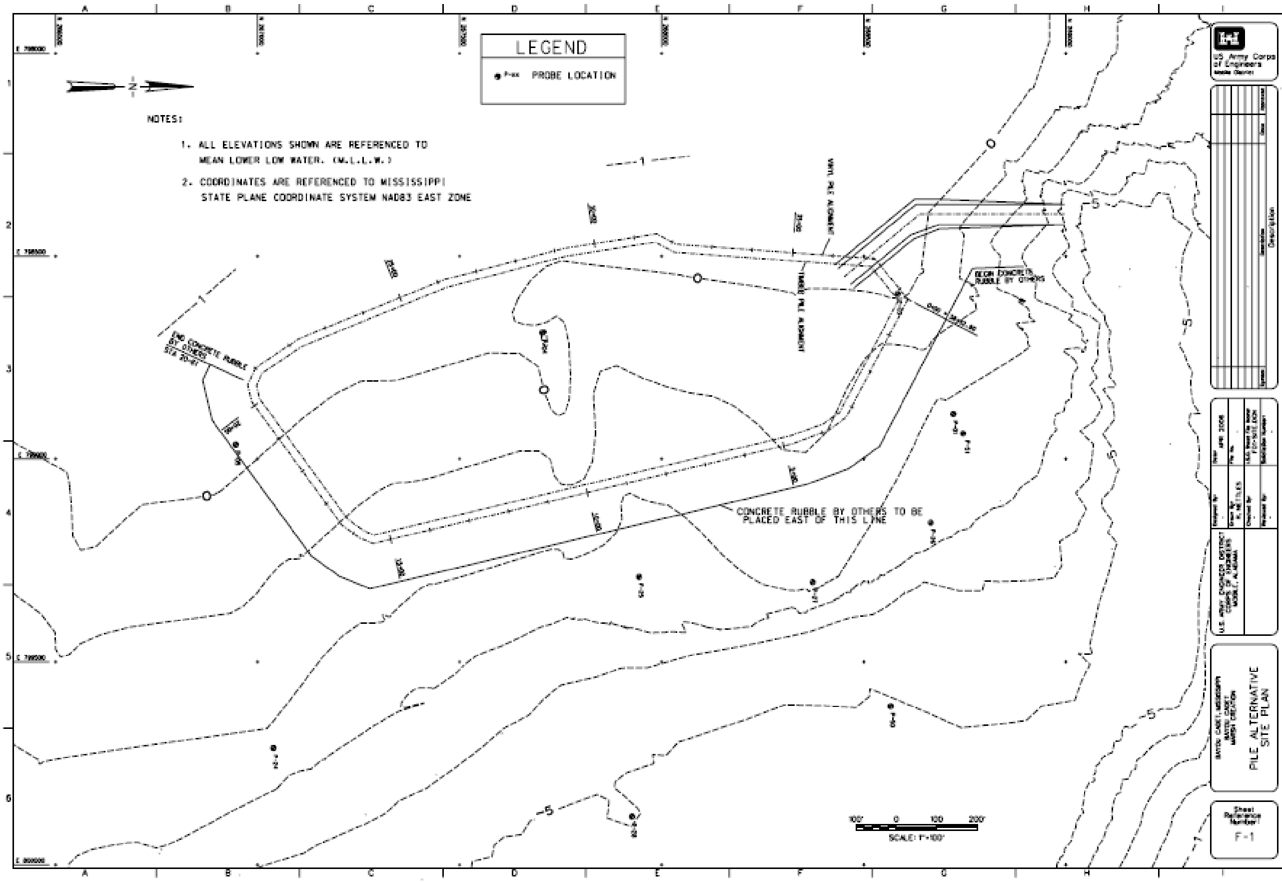


Figure 3. Plan View of Alternatives

- Earth Dike Containment Structure. This alternative would consist of an earth dike with an 8-foot crest width at elevation +6 feet MLLW and 1V:3H side slopes. The estimated fill volume includes template fill volume and additional fill likely to be needed to replace settled dike fill. Existing soil at the site is considered unsuitable for dike fill, based on limited available subsurface data as previously discussed. Dike fill would therefore be obtained from an upland source.
- Steel Sheetpile Containment Structure. The layout for the steel sheetpile alternative would follow the same horizontal alignment as the earth dike alternative, and would accommodate the same material storage volume. The average bottom elevation assumed for the site would be fixed at approximately -1 feet MLLW, approximately following this natural contour over much of the alignment length. The surface of the retained material used for preliminary design of the sheetpile wall was assumed to be at elevation +6 feet MLLW. This would allow for material settlement over time and nominal freeboard above the initial placement elevation. The lateral extent of the wall would be approximately 3,900 linear feet.
- Vinyl Sheetpile Containment Structure. The layout for the vinyl sheetpile alternative would follow the same horizontal alignment as the earth dike and steel pile alternatives, and would accommodate the same material storage volume. Average bottom elevation, surface

elevation of the retained material, and lateral extent of the vinyl pile wall would be the same as for the steel pile wall.

Structural Considerations

The material placed behind the pile structures was assumed to be in a totally fluid condition (i.e., to exhibit no material strength in resisting its own movement). It was assumed to have a saturated unit weight of 110 pounds per cubic foot. In keeping with the limited soils data available, the material from the existing ground line to a depth of 10 feet (elevation -11 feet MLLW) was assumed to have a saturated unit weight of 110 pounds per cubic foot, a phi angle of zero degrees, and a unit cohesion value of 200 pounds per square foot. All material below elevation -11 feet MLLW was assumed to have a saturated unit weight of 115 pounds per cubic foot, a phi angle of 30 degrees, and no unit cohesion. The water pressures encountered at this site will be from sea water, and were thus computed using a unit weight of 64.4 pounds per cubic foot of water. In applying the soil values to the cantilever sheet pile wall, a safety factor of 1.5 was applied to the passive soil resistance, resulting in reduction of the lateral soil resistance by approximately 33% over the upper 10 feet of the pile embedment depth and approximately 40% reduction of these values below 10 feet of embedment.

Even though the wall height is to be near the upper limit for cantilevered wall design using conventional steel sheet piles, it is considered preferable to use either a vinyl or a vinyl composite pile because of the extremely corrosive environment. From limited research, there initially appeared to be two plastic pile products that might possibly serve the purposes for this work. One is a heavy vinyl pile which has significant strength but has a relatively low elastic modulus. Use of the largest section included in this product line (ShoreGuard 950 Vinyl Pile manufactured by Crane Materials International, having a material thickness of 0.65 inches, a section depth of 11.75 inches, an individual pile width of 18 inches, and a moment of inertia of 346.6 in4 per running foot of wall in place) would result in inordinately large elastic deformations (estimated by computation at approximately 3 feet of deflection at the top of the wall). Another stronger and more rigid pile is made by extrusion of vinyl and other higher strength plastics (manufactured by Northstar and labeled ENDURANCE CSP, Composite Sheet pile, and having a material thickness of 0.25 inches, a section depth of 8 inches, an individual pile width of 18 inches, and a moment of inertia of 51.58 in4 per running foot of wall in place). Even though this pile has very good strength properties and a relatively high elastic modulus, roughly 10 times that of the purely vinyl product, they are currently only manufactured in 8-inch depth sections, which materially effects the pile stiffness. It was estimated by computation that piling made of this material would deflect approximately 20 inches at the top of the wall.

A tied-back wall system was then developed, again using vinyl/vinyl composite materials. It was concluded that a vinyl sheet pile wall with tie-backs and wales and pole-type anchor piles of treated timber would be sufficient to resist the applied material loads. The resulting sheet pile wall would extend vertically from elevation +6 feet MLLW to elevation -13 feet MLLW, for a total wall height of 19 feet. By using a tied back design, the pile section was reduced to an 8-inch plain vinyl section and the penetration was reduced to only that required to key the piling into the sand layer described above. Furthermore, the predicted deflections were negligible. The entire system proposed for this alternative would consist of an 8-inch vinyl sheet pile wall, a wale system of 8-inch by 8-inch treated timbers attached to the wall using non-corrosive bolts, cable tiebacks, and treated timber anchor piles placed 12 feet on centers at approximately 20 feet behind the sheet wall. The steel pile alternative would require a deeper pile penetration, and thus more square feet of steel piling for that alternative. When viewed from a service life perspective, the vinyl should be the better system in the environment in which the facilities are to be installed.

Construction Procedures and Water Control Plan

The concrete rubble would be placed to function as a breakwater for the inner containment structure. The rubble would either be placed by others directly at the breakwater site to construct the breakwater or would be placed by others at an offsite stockpile and used later to construct the breakwater as part of this project. Since water depths at the proposed project site are shallow, dredging of an access channel would be required for construction of the inner containment structure. If not dredged by others, dredging of another access channel to construct the breakwater would be required. Both channels would be trapezoidal and would have a 50-foot bottom width at elevation -4 feet MLLW, and 1V:3H side slopes. The barge access channel to the inner containment structure would extend about 600 feet from the -4 feet contour at the Bayou Caddy channel to the interior of the north part of the site. This route is the shortest suitable path that would bypass the concrete rubble breakwater to be placed. The access channel to the breakwater would extend about 3100 feet from the -4 feet contour at the Bayou Caddy channel around the seaward (east) side of the site. The Contractor would have the option on how to move around within and construct the site (i.e., either excavate more barge canal inside the area and/or construct a haul road on the earth dike). For the pile containment structure alternative, the barge access channel would need to be extended around the interior perimeter of the site to allow for construction access. Construction of a weir would also be included as part of the containment structure. The landward side of the containment structure would be filled with dredged material from the next maintenance and/or new work dredging of the Bayou Caddy navigation channel. The material would be allowed to settle and consolidate, and appropriate vegetation would be planted. All construction features except for the concrete rubble breakwater would then be removed after planting to allow for naturalization of the marsh area.

Project Security

Development of a physical security plan in accordance with Army Technical Manuals 5-853-1, -2, -3, and -4, as produced by the Protective Design Center of Expertise at the Omaha District, is not required for this project.

Operations and Maintenance

It is anticipated that the concrete rubble breakwater feature of each alternative plan will require some operations and maintenance (O&M) over the project life. That maintenance is estimated at about 10% of the initial construction quantity every 5 years over the expected 50-year life of the project. However, since the other construction features will be removed after planting of the marsh, no other O&M will be required.

Cost Estimates

Estimated costs for initial construction and O&M of each alternative plan are shown in Tables 2 through 6. Cost for breakwater is not included in Alternatives 1, 2 and 3 and is included separately because it may be constructed by others. Quantity estimates are based on surveys performed by the Mobile District in December, 2003. These estimates include costs for contingencies, engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, subsurface investigation, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Maintenance dredging of the Bayou Caddy navigation channel occurs approximately every 5 to 6 years. The amount of material dredged has varied from 123,739 CY to 234,877 CY. Under the proposed restoration project, maintenance dredging of the navigation project would be accomplished as scheduled under the normal cycle. The proposed restoration site is immediately adjacent to the navigation channel and is within the typical pumping distances to the open water disposal areas normally used for maintenance. Based on records of past maintenance dredging, it is anticipated that there is sufficient quantity of material in the channel segments proximate to the proposed marsh creation sites to provide yields sufficient to construct the proposed marsh. The construction cost of the proposed restoration effort includes dredging of an access channel, constructing and stabilizing containment dikes, placement of a weir, managing the fill material to achieve the desired final site elevation, planting marsh grasses, and subsequent removal of all construction features after planting except for the concrete rubble breakwater to be placed by others.

Table 2.
Initial Construction Cost Estimate for Steel Sheet Pile Containment

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Coastal MS Study, Bayou Caddy
LOCATION: Hancock County MS.
WORK ITEM: **Steel Pile Structure**

ITEM NO. 1 — DATE 27-Jul-06
SHEET NO. 2 — OF 3
PREPARED: Parmer — CHECKED: Ellsworth
BASIS of ESTIMATE: Info furnished per PDT Team
FILE NAME: bayou caddy6-26.xls

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Steel Pile Structure				
Mobilization, Preparatory Work, Demobilization	1	job	allow	75,000
Excav channel for steel pile placement	26,000	cy	12.00	312,000
Steel Sheet pile	83,850	sf	35.00	2,934,750
Containment fill	120,000	cy	7.50	900,000
Weir	1	ls	75,000	75,000
Site Grading & Shaping (18 acres)	87,000	sy	0.25	21,750
Planting	18	acr	10000	180,000
Remove Steel Sheet pile	41,925	sf	10.00	419,250
Misc. Site Items	1	ls	allow	70,000
	Total Direct Construction Cost			$4,987,750
	Indirect Cost @		15%	748,163
				5,735,913
	Profit @		9%	516,232
				6,252,145
	Bond @		1.5%	93,782
06 Account, Fish & Wildlife	Current Contract Cost, Oct 06			$6,345,927
	01 Account, Lands & Damage (PCA)		LS	525,000
				6,870,927
	30 Account, Plan, Engr.& Design		10%	687,093
				7,558,019
	31 Account, Constr. Management		6%	453,481
				8,011,501
	CONTINGENCY		20%	1,077,300
				9,088,801
				$9,088,801
				rounded
	TOTAL PROJECT COST, FY-07			**$9,090,000**

Table 3.
Initial Construction Cost Estimate for Steel Vinyl Sheet Pile Containment

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Coastal MS Study, Bayou Caddy
LOCATION: Hancock County MS.
WORK ITEM: ***Vinyl Pile Structure***

ITEM NO. 1 | DATE 27-Jul-06
SHEET NO. 2 OF 3
PREPARED: Parmer | CHECKED: Ellsworth
BASIS of ESTIMATE: Info furnished per PDT Team
FILE NAME: bayou caddy6-26.xls

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Vinyl Pile Structure				
Mobilization, Preparatory Work, Demobilization	1	job	allow	75,000
Excav channel for vinyl pile placement	26,000	cy	12.00	312,000
vinyl pile	74,100	sf	25.00	1,852,500
Containment fill	120,000	cy	7.50	900,000
Weir	1	ls	75,000	75,000
Site Grading & Shaping (18 acres)	87,000	sy	0.25	21,750
Planting	18	acr	10000	180,000
remove vinyl pile	37,050	sf	8.00	296,400
Misc. Site Items	1	ls	allow	70,000

	Total Direct Construction Cost		$3,782,650
	Indirect Cost @	15%	567,398
			4,350,048
	Profit @	9%	391,504
			4,741,552
	Bond @	1.5%	71,123
06 Account, Fish & Wildlife	Current Contract Cost, Oct 06		$4,812,675
	01 Account, Lands & Damage (PCA)	LS	525,000
			5,337,675
	30 Account, Plan, Engr.& Design	10%	533,768
			5,871,443
	31 Account, Constr. Management	6%	352,287
			6,223,729
	CONTINGENCY	20%	719,746
			6,943,475
			$6,943,475
			rounded
	TOTAL PROJECT COST, FY-07		**$6,940,000**

Table 4.
Initial Construction Cost Estimate for Earth Dike Containment with Breakwater

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Bayou Caddy	ITEM NO.	1	DATE	27-Jul-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Earth Dike with Breakwater Modification***	BASIS of ESTIMATE:		Info furnished per PDT Team	
		FILE NAME:	bayou caddy6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Earth Dike with Breakwater Modification				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
Excav Channel for concrete rubble placement	16,000	cy	12.00	192,000
Load, haul and place concrete rubble	25,000	cy	30.00	750,000
Excav Channel for earth dike work	4,400	cy	12.00	52,800
Haul Road	1	ls	50,000	50,000
Earth Dike	50,000	cy	15.00	750,000
Containment fill	120,000	cy	7.50	900,000
Weir	1	ls	75,000	75,000
Site Grading & Shaping (18 acres)	87,000	sy	0.25	21,750
Planting	18	acr	10000	180,000
Remove earth dike	7,000	cy	8.00	56,000
Misc. Site Items	1	ls	allow	70,000
	Total Direct Construction Cost			$3,147,550
	Indirect Cost	@	15%	472,133
				3,619,683
	Profit	@	9%	325,771
				3,945,454
	Bond	@	1.5%	59,182
06 Account, Fish & Wildlife		Current Contract Cost, Oct 06		$4,004,636
	01 Account, Lands & Damage (PCA)		LS	225,000
				4,229,636
	30 Account, Plan, Engr.& Design		10%	422,964
				4,652,599
	31 Account, Constr. Management		6%	279,156
				4,931,755
	CONTINGENCY		20%	761,351
				5,693,106
				$5,693,106
				rounded
	TOTAL PROJECT COST, FY-07			**$5,690,000**

Table 5.
Summary of Initial Construction Cost Estimates for All Alternatives

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Bayou Caddy	ITEM NO.	Summary	DATE	27-Jul-06
LOCATION:	Hancock County MS.	SHEET NO.	1	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	Summary	BASIS of ESTIMATE:	Info furnished per PDT Team		
		FILE NAME:	bayou caddy6-26.xls		

Alt No.	DESCRIPTION	Quantity	Unit		ESTIMATED AMOUNT
1.	*Earth Dike*	1	job	xxx	$4,140,000
2.	*Steel Pile Structure*	1	job	xxx	$9,090,000
3.	*Vinyl Pile Structure*	1	job	xxx	$6,940,000
4.	*Breakwater*	1	job	xxx	$2,010,000
5.	*Earth Dike with Breakwater Modification*	1	job	xxx	$5,690,000

Notes:
Price Level, Oct 06
Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Table 6.
O&M Cost Estimate (5-Year Cycle) for All Alternatives

PROGRAMMING & PLANNING COST ESTIMATE

O & M COST ESTIMATE

PROJECT:	Coastal MS Study, Bayou Caddy	ITEM NO.	1	DATE	21-Apr-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Stone Replacement***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m bayou caddy4-22.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Stone Replacement					
Mobilization, Preparatory Work, Demobilization		1	job	allow	50,000
Stone		2,500	cy	150.00	375,000
	Total Direct Construction Cost				$425,000
	Indirect Cost	@		**15%**	63,750
					488,750
	Profit	@		**9%**	43,988
					532,738
	Bond	@		**1.5%**	7,991
		Current Contract Cost, Oct 06			$540,729
	01 Account, Lands & Damage (PCA)			**LS**	0
					540,729
	30 Account, Plan, Engr.& Design			**10%**	54,073
					594,801
	31 Account, Constr. Management			**6%**	35,688
					630,490
	CONTINGENCY			**20%**	126,098
					756,587
	ESCALATION, FY-07			**1%**	7,566
					$764,153
					rounded
	TOTAL PROJECT COST, FY-07				**$760,000**

Schedule for Design and Construction

A schedule for preparation of P&S through construction is shown in Table 7 below.

Table 7.
E&D And Construction Schedule

Task	Start	End
Draft P&S	Receipt of funds	3 months after start
ITR/BCOE reviews		1 week after start
Final P&S/RTA		1 week after ITR/BCOE reviews
Advertise		2 weeks after RTA
Open bids		30 days after advertise
Award		30 days after open bids
NTP		3 weeks after award
Construction of breakwater (by others)	TBD	TBD
Construction of inner containment dike		4 months after NTP
Placement of dredged material		3 months after inner dike construction
Marsh plantings (time delay needed for consolidation)	24 months after dredged material placement	3 months after start
Complete construction/Project closeout		4 months after plantings

Additional References

U.S. Army Corps of Engineers, “Preliminary Restoration Plan for Cadet Bayou Marsh Creation Project,” Section 204 Aquatic Ecosystem Restoration in Connection with Construction and Maintenance Dredging of an Authorized Project, Mobile District, August, 2003.

HANCOCK COUNTY BEACHES

General

The purpose of this document is to provide engineering information for planning and design of environmental restoration measures, interior drainage infrastructure and storm damage reduction for areas damaged by Hurricane Katrina near Bayou Caddy and the Bay St. Louis and Waveland communities of Hancock County, Mississippi.

Location

The study shoreline areas are located in Hancock County, the eastern-most coastal county in Mississippi, between Bay St. Louis and Bayou Cadet. It is located on Mississippi Sound about 95 miles west of Mobile, Alabama and about 50 miles east of New Orleans, Louisiana.

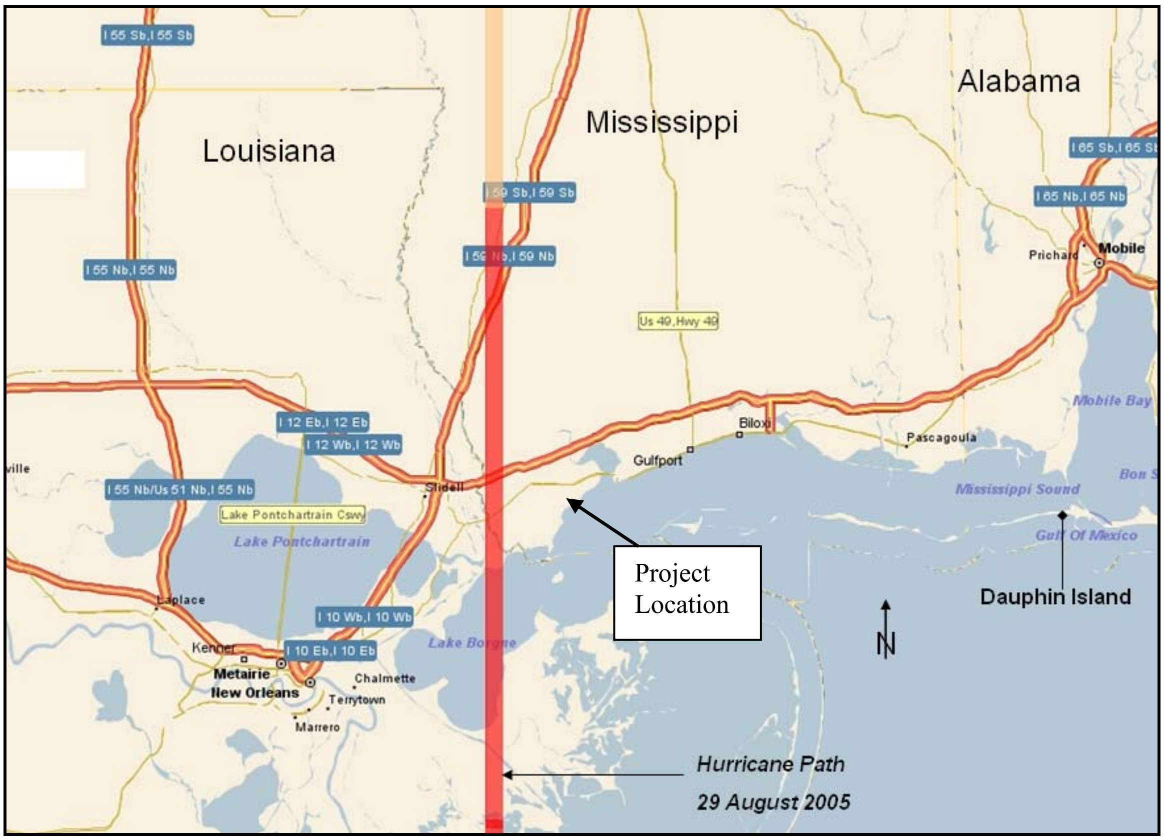


Figure 1. Location Map Showing Path of Hurricane Katrina.

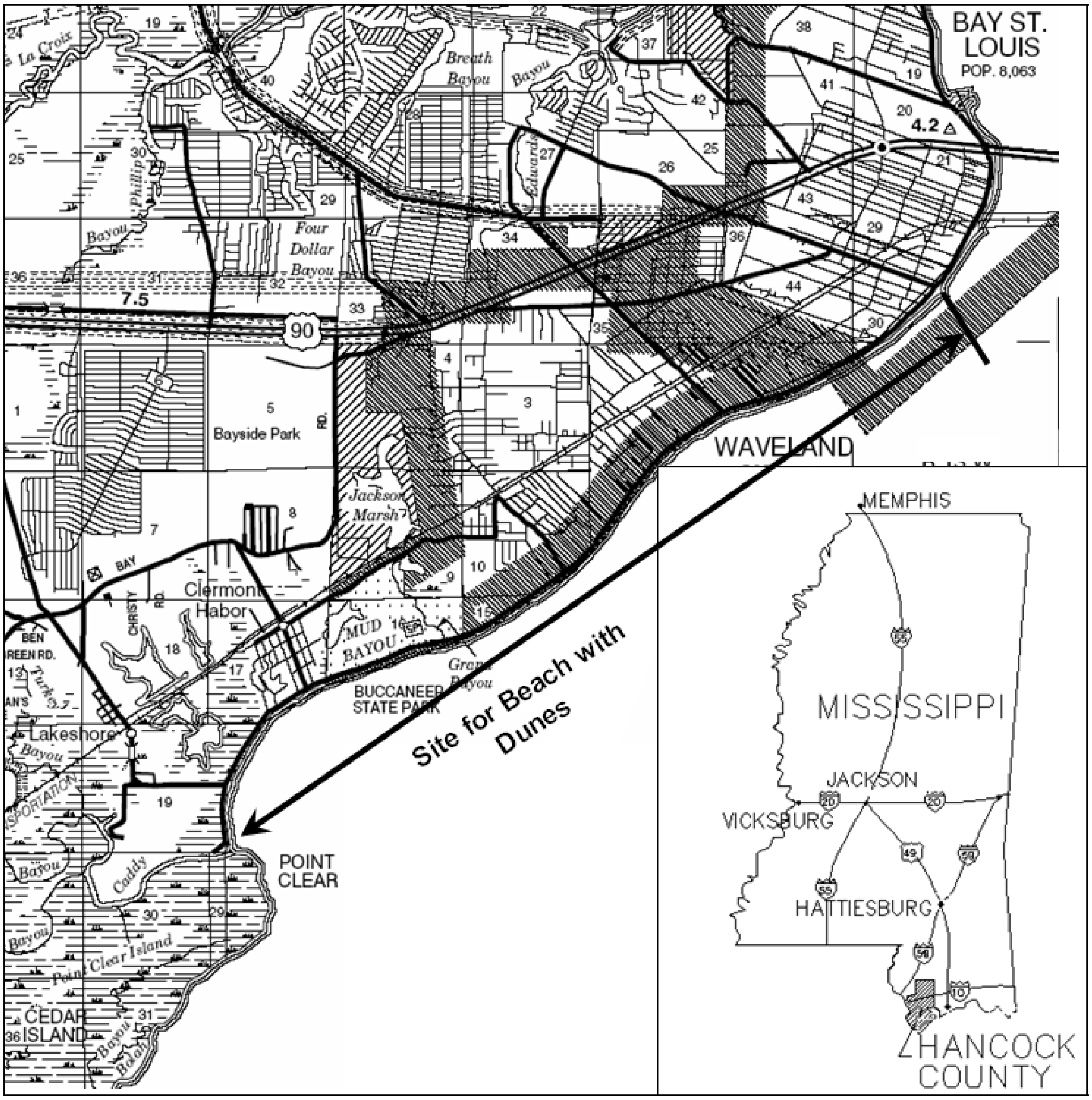


Figure 2. Area Map

The site location is shown on Figures 2, 3, and 4. The Hancock County shoreline running between Bayou Caddy and Waveland is fronted by South Beach Boulevard, which is protected by a concrete seawall and existing beach. The project sites are seaward of Beach Boulevard existing seawalls some 50 feet, creating a 2 foot high sand berm with 1 vertical to 3 horizontal side slopes and supplemented by sand fencing and plantings.

Existing Conditions

The existing Mississippi Sound shoreline in the area is protected by a concrete stepped-face structure about 8 miles long. The seawall was constructed by local interests at various times between 1915 and 1928. Hydrographic and topographic survey data was obtained in the area by the Mobile District under contract in September, 2003. The top elevation of the seawall varies between +3.8 to +5.0 feet National Geodetic Vertical Datum (NGVD). A sand beach was pumped into place along about six miles of this seawall in early 1967 as part of the emergency repair and protection following Hurricane Betsy (September 1965). There is another beach extending for about a mile south of the U.S. Highway 90 Bridge crossing the mouth of St. Louis Bay that was placed by the Mississippi Highway Department during the bridge construction. An additional one-mile-long segment of beach with dunes was constructed in the summer of 2005 from Cadet Bayou eastward. Figure 3 shows the shoreline between Bayou Caddy (Cadet Bayou) and the Washington Street pier. Figures 6 and 7 show typical storm related damage at the beachfront.

South Beach Boulevard is the main thoroughfare along the entire length of the existing seawall. Historical as well as current wave attack against the shoreline of Hancock County has caused migration of soil through or under the seawall and scour of soil below the seawall in various locations, resulting in damages to South Beach Boulevard and other infrastructure. Sections of the highway have collapsed from time to time, disrupting and damaging utilities, and causing hazards and delays for residents and vehicular traffic. Hancock County has frequently repaired the seawall and road because of the loss of material from beneath the highway. Damaged utilities which have required repairs include water, sewer, natural gas, electric power, and electronic communications. The Mobile District has constructed a number of new seawall segments along various reaches of the existing seawall to alleviate this soil migration and scour problems in the study area under Sections 14 and 103 authorities. Seawall alternatives are addressed in the Clermont Harbor and Downtown Bay St. Louis plans.

The seawall is penetrated in a number of locations by sixteen open drainage channels. Typically, the components of these drainage channels at their crossings of South Beach Boulevard include concrete headwalls, concrete box culverts beneath the boulevard, and channel extension guide-walls extending out into Mississippi Sound. Many of these were severely damaged by hurricane Katrina. Typical damages included breaching of the extension guidewalls, failure of the guidewalls, and destruction of the outlet end of the box culverts. Figures 4 and 5 show Hurricane Katrina damage at one site along Beach Boulevard.

Several tidal marshes exist on the landward side of the roadway on the southwestern end of Hancock County around the Waveland area. The existence of these expansive and contiguous tidal marshlands are maintained through tidal conduits (outfalls) built into the existing seawall at regular intervals. Many of the tidal conduits supporting these marsh areas are in a state of severe deterioration. It is also believed that the much of the tidal flow between Mississippi Sound and the marshes have been critically restricted from sedimentation as a result of Hurricane Katrina. The existence of these valuable marshlands is dependent upon the continuation of the tidal exchange provided by the outfalls. The overall health of the marshes is likely constrained by the limited water exchange allowed by the tidal conduit system. Reconstruction and rehabilitation of these systems in a manner that would increase tidal flow and re-establish pre-storm interior drainage capacity is addressed in the Jackson Marsh plan.

Coastal and Hydraulic Data

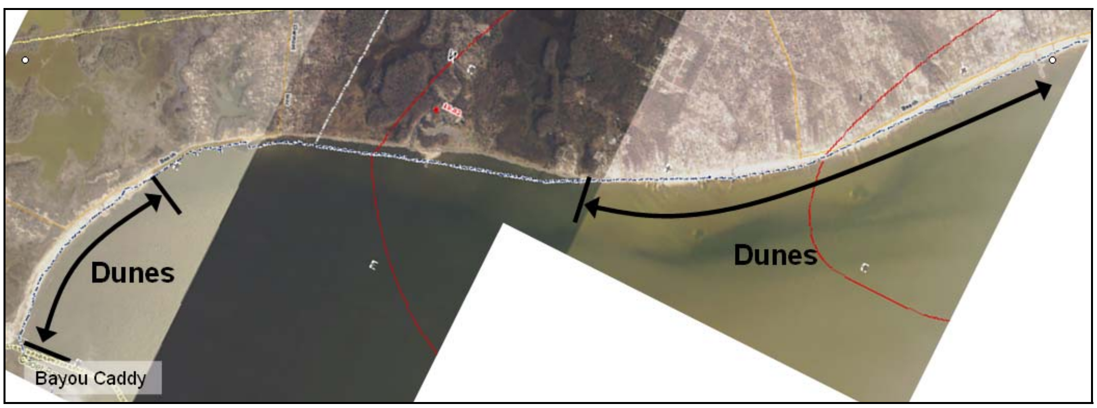
The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

Mississippi Sound is a shallow coastal lagoon extending 80 miles along the coast of the Gulf of Mexico from Mobile Bay, Alabama westward to Lake Borgne, Louisiana. The average depth in the sound is 10 feet, and 99 percent of the sound is less than 29 feet deep.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in St. Louis Bay is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The study area has been impacted by several tropical storms and hurricanes, most recently from Hurricane Katrina in 2005. Post –Katrina recovery of high water marks in the area suggest storm surges on the order of 20 to 25 feet or more. Frequency estimates of historic storm tide elevations are shown in Table 1, suggesting surges from Katrina far exceeded the 100-year surge elevation.

Table 1.
Storm Tide Frequency (feet, NGVD)

2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
3.7	5.1	6.6	9.1	11.7	15.1



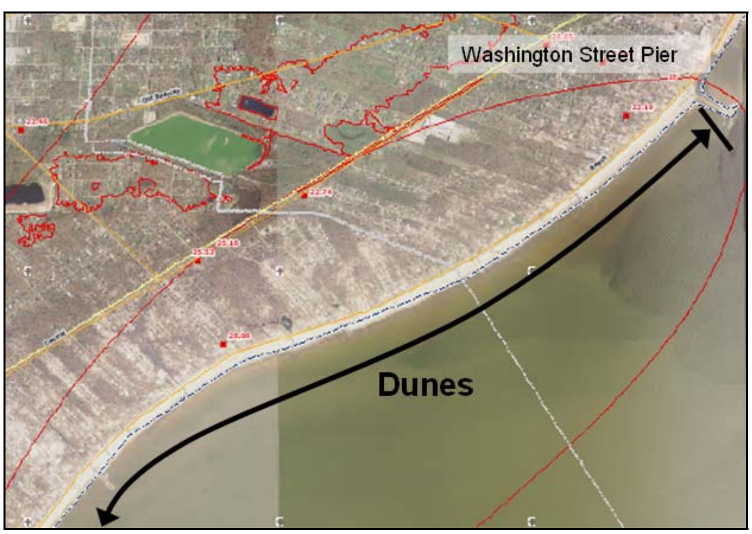


Figure 3. Shoreline from Bayou Caddy (top) to the Washington St. Pier (bottom).

Figure 4. Damaged Pathway, Drainage Channel Outlet, and Outlet Bridge, Near Waveland. Beach Road and Seawall Beyond.

Figure 5. Channel Outlet. Outlet is Breached, Extension Walls are Damaged, and Outlet is Choked with Sand.

Figure 6. Beach and Boardwalk Damage, Looking Southward Near Bayou Caddy.

Figure 7. Beach Erosion along South Beach Boulevard.

Geotechnical Data

The project is located along Beach Blvd. within Hancock county running from Bayou Caddy on the west end through Waveland to Washington Street to the east. The beach road is established at El. 5.0 +, with the beach extending some 150 feet to the water's edge. Typical profiles for this plan can be seen herein. Materials used for the dune construction will have 90% passing the #40 sieve and only 10% will pass the #200 sieve. The sand fill shall not have noticeable amounts of shell and/or gravel. The sand will be trucked to the sites form upland sources within 10 miles of the work area, dumped and reshaped in place.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

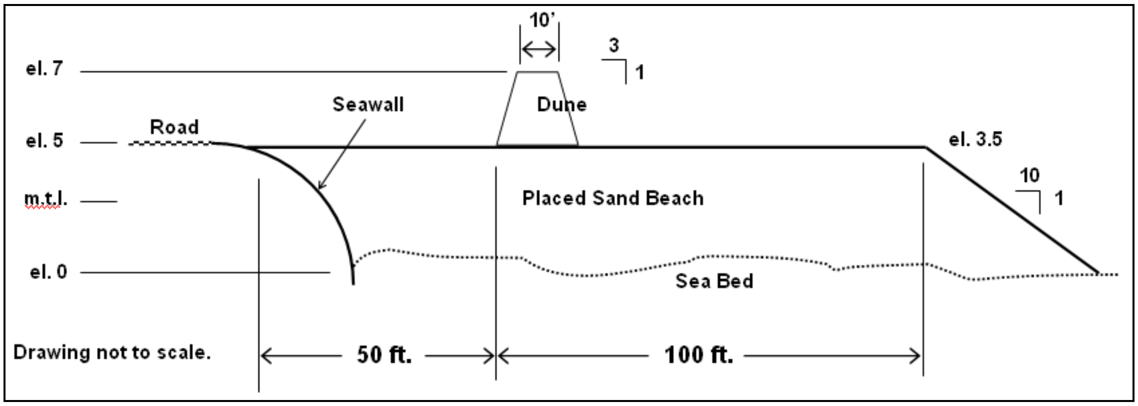
Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.



Typical section, elevations may vary with specific locations.

Alternative Plans

Two plans were evaluated for enhanced beach protection at the study site. All involved providing a dune atop the existing beach. One alternative will be to place the dune material alone and the other alternative will be to place the dune material and add stabilizing fencing and dune vegetation. The finished stable dune will be 2 feet high to approximately Elevation 7.0 with a crest width of 10 feet and side slopes of one vertical to three horizontal. The material will come from the established upland borrow areas within 10 miles of the work area. The plantings will have a density of 1 plant per 4 square feet and the fence will include the entire linear length of the project. The dune alone project will require replacement within 10 years and the dune with plantings and fence will require replacement within 15 years.

Construction Procedures and Water Control Plan

The construction plan will be to install the new dune 50 feet seaward of the existing seawall at the edge of South Beach Road. Construction surveys will be necessary to lay out the design beach template and to confirm as-built grading meets design intents.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

The “dune alone” alternative will require replacement within 10 years and the “dune with plantings and fence” alternative will require replacement within 15 years. Both alternatives will require removal of wind-blown sand from Beach Road by street sweeping equipment, and transferal of wind-blown sand from the lee of the dunes to the front. It is estimated that relocation of sand due to ‘normal’ wind and weather will be required twice annually with a total estimated annual amount to be relocated of no more than 0.25 cubic feet of sand per foot of beach (approximately 300 cubic yards) (reference 1) for the “dune alone” alternative, the “dune with plantings and fence” alternative requiring perhaps 70% of this effort.. Severe storms, such as hurricanes, could severely damage the project regardless of the presence or absence of fencing and vegetation and require replacement of the dunes. The base of the dune is assumed to be at Elevation 4.0. If the still-water elevation at the base of the dune is the elevation at which storm surge, with additional wave action, would begin to erode the dune, an approximately 2-year recurrence interval surge corresponds to this elevation based on frequency analysis of annual maximum water surface elevations at Biloxi.

Cost Estimates

Estimated costs for the alternative plans are shown in Table 2. Quantity estimates are based on drawings and rudimentary field measurements. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 2.
Estimated Costs

ALTERNATIVES	QUANTITY	UNIT	ESTIMATED COST
Beach Dune	43,800	CY	
Total		LS	$1,270,000
Annual O & M		LS	$40,000
Dune	43,800	CY	
Fencing	37,000	LF	
Planting	19	ACRE	
Total		LS	$1,770,000
O & M		LS	$40,000

Schedule and Design for Construction

A typical schedule for preparation of P&S through construction is shown in Table 3 below.

Table 3.
Typical Schedule for P&S

Draft P&S	6 weeks after start
ITR/BCOE review	2 weeks after draft P&S
Final P&S/RTA	3 weeks after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	6 months after NTP

HANCOCK COUNTY STREAMS

General

The hurricanes of 2005 caused damage to drainage ways by blowing trees and other debris into these areas and by deposition of sediment in many areas of Hancock County, MS. There were many canals and drainage ways for low-lying areas near the coast that were affected. This document provides information regarding damage to the drainage ways of the developments or areas near Cowan Bayou and Hancock County Drainage Canals. Rough order-of-magnitude cost estimates for restoring the capacity of these water courses is also presented.

Location

A general location map of the study areas is shown below.

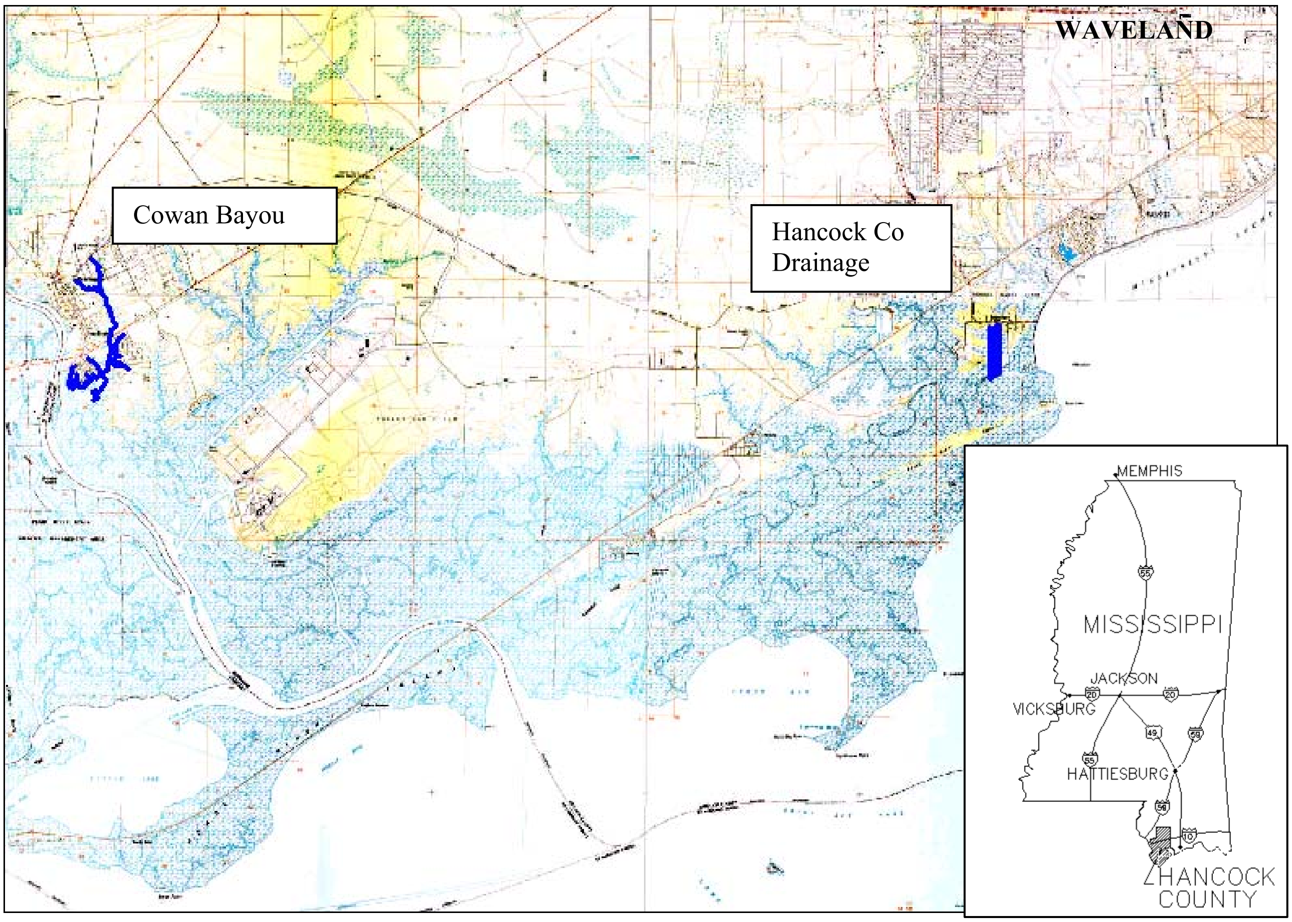


Figure 1. Location Map

Hancock County Streams - Cowan Bayou

This area consists of the drainage at Whites Road, the subdivisions of Oak Harbor and Belle Isle, and the drainage ways connecting these areas. A map of these areas is shown below.

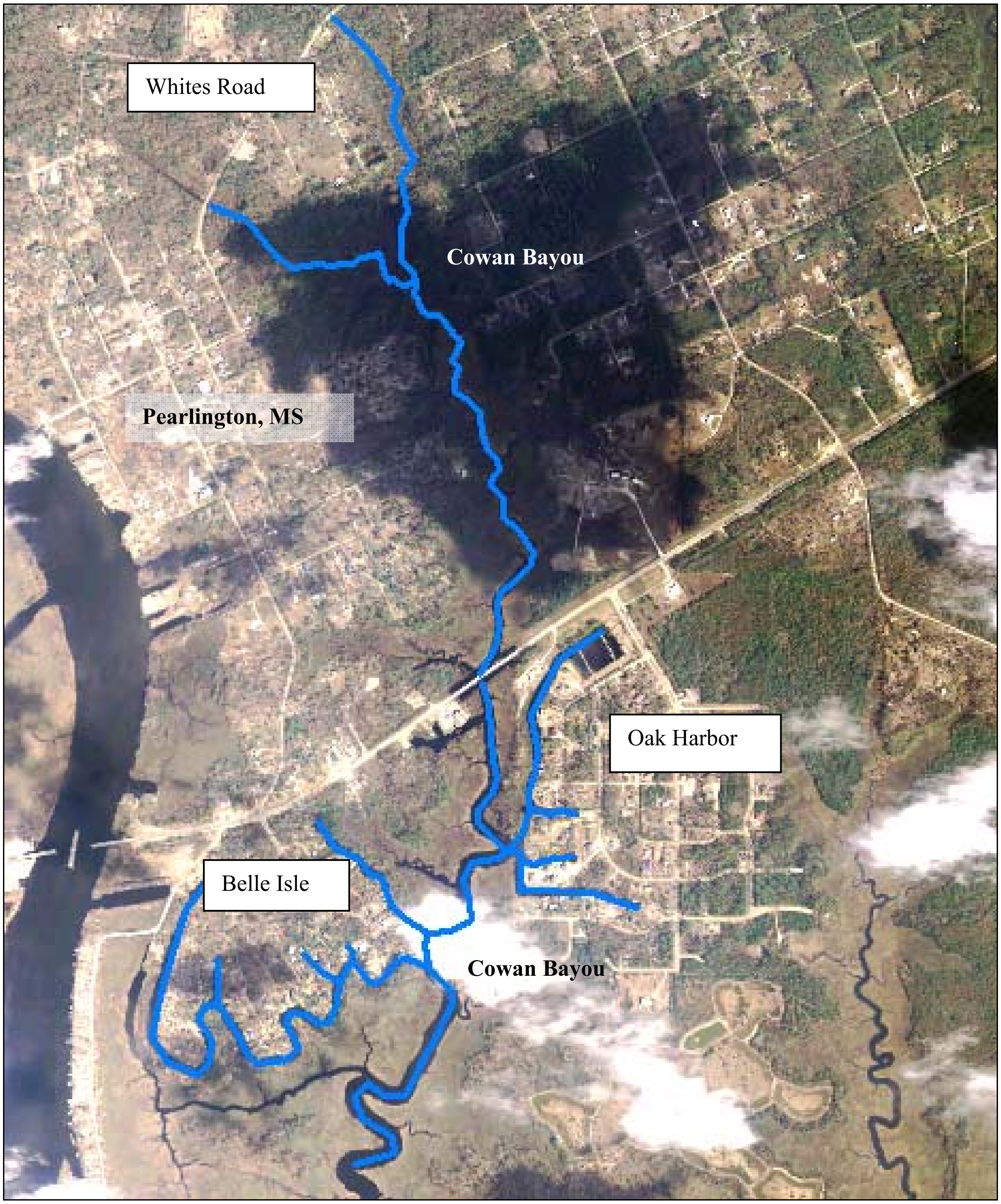


Figure 2. Hancock County Streams - Cowan Bayou

Existing Conditions

The drainage canals in these subdivisions vary in width from approximately 15 ft. – 75 ft. with an average of approximately 45 ft wide. The Cowan Bayou canals total approximately 4.7 miles in length. Although it could not be verified, an engineer representing Hancock County states that the canals shoaled approximately 2 ft from the 2005 hurricanes, from an elevation of -4 ft NGVD to -2 ft NGVD. Photographs of the shoaling along Whites Road are shown below.

Coastal/Hydraulics

High water marks by FEMA indicate water reached elevations near 20 ft NGVD on Cowan Bayou at Pearlington, MS.

Additional data is provided in a report to FEMA by URS Group, Inc., titled "Hurricane Katrina Rapid Response Mississippi Coastal and Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report," 16 January 2006, as well as in a report by FEMA titled "Draft Report, Hurricane Katrina Flood Frequency Analysis," dated September 2005. Results are summarized below.

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b) (Tables 1 and 2 and Figure 6). The flood frequency analysis only represents conditions at and near the gage.

Figure 3. Hancock County Streams - Cowan Bayou - Drainage at Whites Road

Figure 4. Hancock County Streams - Cowan Bayou - Oak Harbor

Figure 5. Hancock County Streams - Cowan Bayou - Belle Isle

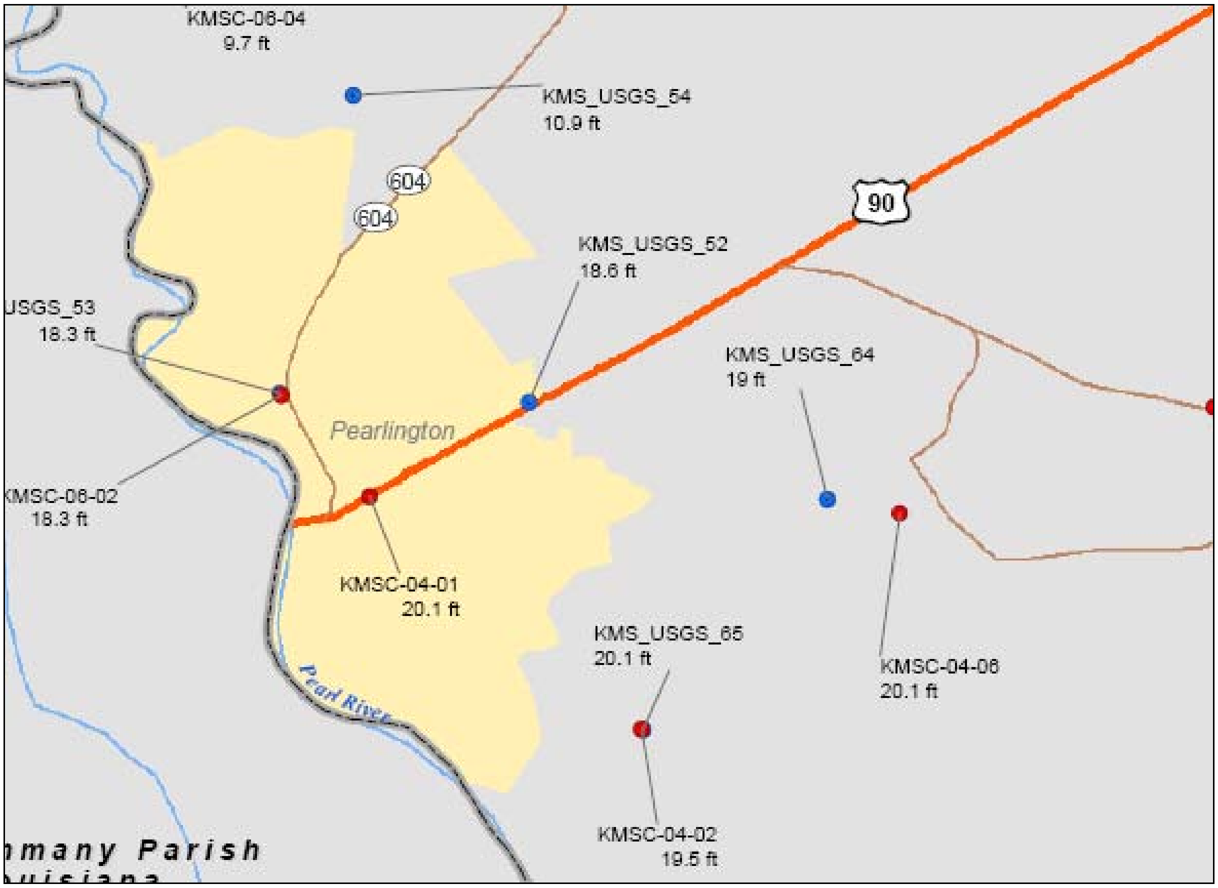


Figure 6. High Water Marks at Pearlington

Table 1.
Selected Tidal Gage Stations from NOAA

Station ID	Name	Latitude	Longitude	Begin Year	End Year
8729840	Pensacola, Pensacola Bay, FL	30.40 N	87.21 W	1924	2005
8735180	Dauphin Island, Mobile Bay, AL	30.25 N	88.08 W	1967	2005
8747766	Waveland, Mississippi Sound, MS	30.28 N	89.37 W	1979	2005
8761724	Grand Isle, East Point, LA	29.26 N	89.96 W	1972	2005

Table 2.
Selected Tidal Gages from USGS/USACE

Name	Latitude	Longitude	Begin Year	End Year
Back Bay Biloxi at Biloxi, MS	30.40 N	88.84 W	1882	1998
Pascagoula River at Pascagoula, MS	30.37 N	88.56 W	1940	1998

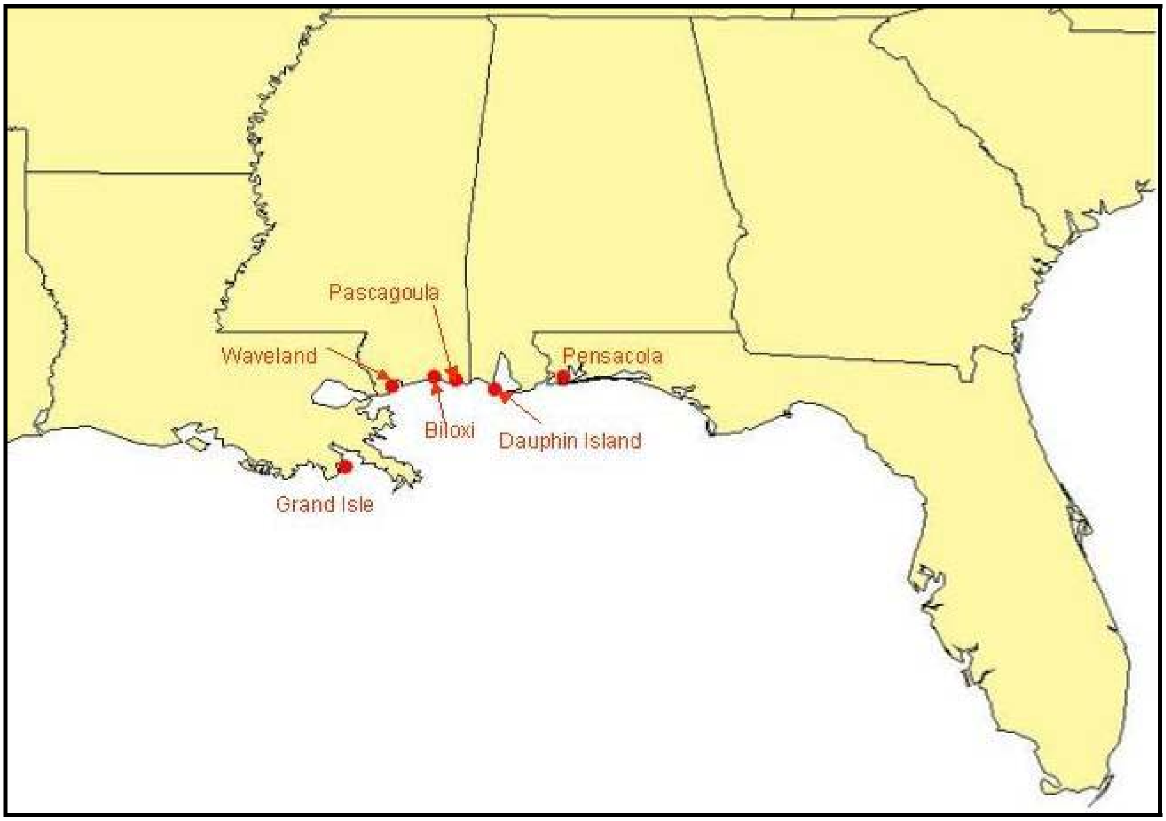


Figure 7. Tidal Gage Locations in the Area Impacted by Hurricane Katrina

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.
- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.
- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.

- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 3. As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 3.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations
Location Katrina Elevation

Location	Katrina Elevation (ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
Dauphin Island, AL	5.81	20
Pensacola, FL	6.07	50

A stage-frequency curve developed by the Corps of Engineers for the Biloxi gage is shown below. The gage shows stage 24 to have a return frequency of 100 years compared to the FEMA table which shows a return interval of >10,000 years.

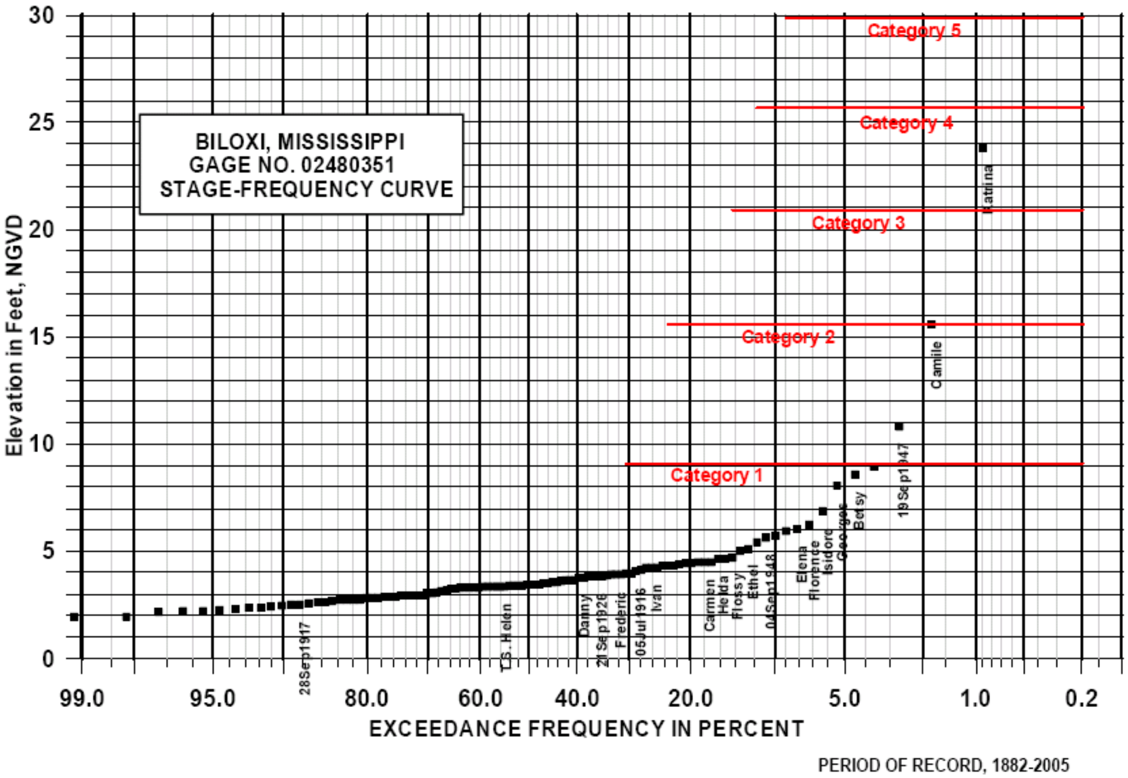


Figure 8. Stage-Frequency Curve

Geotechnical

Geotechnical subsurface investigation has not been conducted for this project and subsurface conditions at the site are unknown. Subsurface conditions are assumed to be similar to those at the closest available geotechnical borings. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Hancock County. The closest geotechnical boring to this site is the Mississippi Department of Geology boring identified as HK29. This boring is located approximately 2500 feet southeast of the site. Sample descriptions and grain size data for the upper 10 feet of this boring are summarized in the table below.

Table 4.
Mississippi Department of Geology Boring HK29

Mississippi Department of Geology Boring HK29 (upper 10 feet):				
Depth	**Description**	**% Gravel**	**% Sand**	**% Silt/Clay**
3' 6"– 3' 11"	Fine Sandy Mud	0.0	49.3	27.1 / 23.6
4' 6"– 4' 9"	Clayey Fine Sand	0.0	60.8	12.0 / 27.2
--- 6' 6"---				
9'1" – 9'6"	Fine Sand	0.0	97.2	2.8

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

Alternative 1: Sediment Removal (2 ft)

This alternative is a short term alternative that would consist of removing approximately 2 ft of sediment over an average width of 45 ft and length of 4.7 miles, as shown in Figure 2. There appears to be a minor amount of debris in the canals which would also have to be removed to facilitate removal of the sediment.

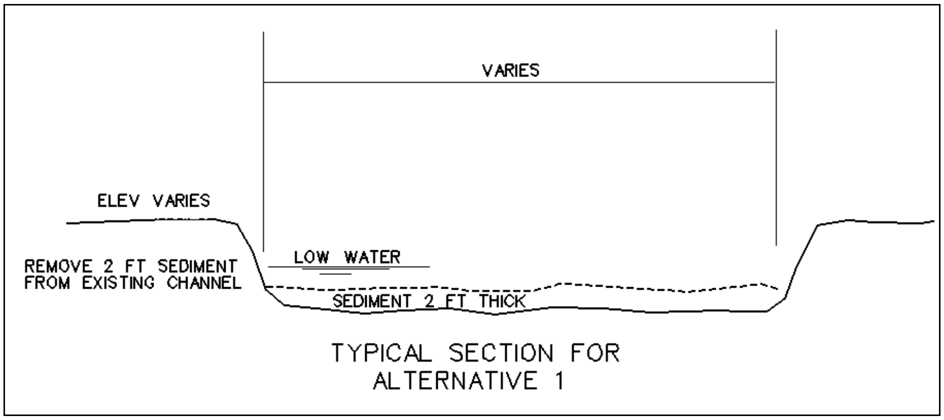


Figure 9. Alternative 1: Sediment Removal

The work could reduce the rainfall flooding to some degree, although tidal water extends throughout the canals below Highway 90. Flooding at Whites Road from high water on the Pearl River would not be reduced. The work in the reach of the bayou above Highway 90 would probably have more impact that work below Highway 90.

Alternative 2: Sediment Removal (1 ft)

This alternative is the same as Alternative 1 except that only 1 foot of sediment would be removed. No additional drawing is provided. This alternative would result in smaller reductions in the water surface that Alternative 1.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection and periodic hydrographic bottom surveys at a few selected locations in the lower reach below Highway 90. These will be plotted using the same reference to monitor changes in the bottom elevations. Maintenance will be the responsibility of Jackson County. Shoaling is expected to be minimal except in the event of a rare hurricane event.

Hancock County Streams – Hancock County Drainage Canals

This area consists of some drainage ways in Hancock County Drainage Canals, and the drainage ways connecting these areas. A map of the area is shown below.

Figure 10. Hancock County Streams – Hancock County Drainage Canals

Existing Conditions

The drainage canals in this area are approximately 100 ft. wide and 300 ft apart. The canals total approximately 1.9 miles in length. An engineer representing Hancock County states that the canals shoaled approximately 2 ft from the 2005 hurricanes. Several larger boats were in the canal at the time which would typically draw approximately 6 ft. The upper end of the eastern canal was clogged with sediment and debris. USGS quad sheets indicate that the elevation of the subdivisions is less than 5 ft. above National Geodetic Vertical Datum (NGVD). Photos in the subdivision are shown below.

Figure 11. Hancock County Streams – Hancock County Drainage Canals

Figure 12. Hancock County Streams – Hancock County Drainage Canals

Coastal/Hydraulics

High water contours by FEMA indicate water reached elevations near 23 ft NGVD at Heron Bay. A photo shows the contour below.

Figure 13. Hancock County Streams – Hancock County Drainage Canals - Inundation Contours

Other storm data is presented in the Cowan Bayou paragraphs above.

Geotechnical

Geotechnical subsurface investigation has not been conducted for this project and subsurface conditions at the site are unknown. Subsurface conditions are assumed to be similar to those at the closest available geotechnical borings. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Hancock County. The closest available geotechnical borings are the USACE Mobile District Bayou Caddy project borings identified as 1 and 2 and the Mississippi Department of Geology boring identified as HK8. Borings 1 and 2 are located approximately 100 to 200 feet east of the easternmost channel at the project. Boring HK8 is located approximately 1000 feet west of the site. Sample descriptions and grain size data for the upper 10 to 12 feet of these borings are summarized in the table below.

Table 5.
USACE Mobile District Boring 1 of Bayou Caddy project (upper 10 feet)

Top Depth	Bottom Depth	Description
0'	3'	Tan Poorly Graded Sand (SP) w/ Tr. Roots
3'	4.5"	Brown Poorly Graded Sand (SP)
4.5"	10.5"	Dk. Gray Sandy Fat Clay (CH) w/ Organic Material

Table 6.
USACE Mobile District Boring 2 of Bayou Caddy project (upper 12 feet)

Top Depth	Bottom Depth	Description
0'	1.5"	Brown Clayey Silt (ML) w/ Organic Material
1.5"	6'	Gray Clayey Silt (ML) w/ Organic Material
7.5"	9'	Gray Clayey Silt (MH) w/ Sand & Organic Material
9'	12'	Gray Clayey Silt (MH) w/ Organic Material

Table 7.
Mississippi Department of Geology Boring HK8 (upper 11 feet)

Depth	Description	% Gravel	% Sand	%Silt/Clay
0' 10" – 1' 0"	Silty Fine Sand	0.0	86.7	9.9 / 3.4
6' 6" – 6' 8"	Muddy Fine Sand	0.0	69.8	18.7 / 11.5
--- 8' 8"---				
10' 8"	Fine Sandy Mud	0.0	20.9	40.9 / 30.1

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

Alternative 1: Sediment Removal (2 ft)

This alternative is a short term alternative that would consist of removing approximately 2 ft of sediment over an average width of 100 ft and length of 1.9 miles, as shown in Figure 10. There appears to be a minor amount of debris in the canals which would also have to be removed to facilitate removal of the sediment.

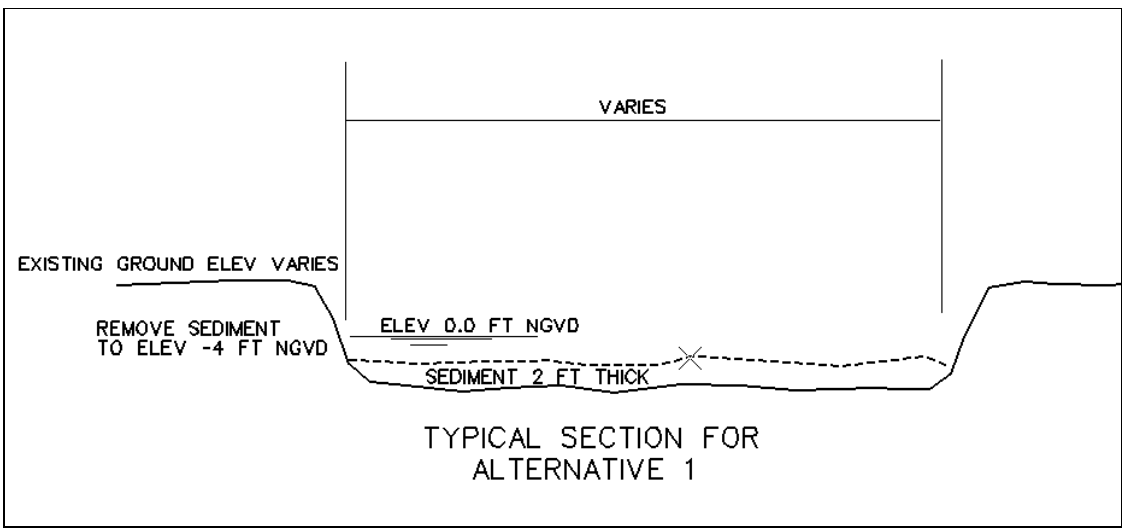


Figure 14. Alternative 1. Sediment Removal

The work could reduce the rainfall flooding to a minimal degree, although tidal water extends throughout the canals.

Alternative 2: Sediment Removal (1ft)

This alternative is the same as Alternative 1 except that only 1 foot of sediment would be removed. No additional drawing is provided. This alternative would result in smaller reductions in the water surface that Alternative 1.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If a marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection and periodic hydrographic bottom. These will be plotted using the same reference to monitor changes in the bottom elevations. Maintenance will be the responsibility of Jackson County. Shoaling is expected to be minimal except in the event of a rare hurricane event.

Table 8.
Hancock County Streams - Alternative 1. Sediment Removal (2ft).
Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Hancock County Streams	ITEM NO.	1	DATE	27-Jul-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Cowan Bayou and Hancock County Canals***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	hancock communities5-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Cowan Bayou and Hancock County Canals 2 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
excavation	157,000	cy	20.00	3,140,000
Misc. Site Items	1	ls	allow	80,000
	Total Direct Construction Cost			$3,320,000
	Indirect Cost	@	15%	498,000
				3,818,000
	Profit	@	9%	343,620
				4,161,620
	Bond	@	1.5%	62,424
09 Account, Channels & Canals		Current Contract Cost, Oct 06		$4,224,044
	01 Account, Lands & Damage (PCA)		LS	862,500
				5,086,544
	30 Account, Plan, Engr.& Design		8%	406,924
				5,493,468
	31 Account, Constr. Management		6%	329,608
				5,823,076
	CONTINGENCY		20%	992,115
				6,815,191
				$6,815,191
				rounded
	TOTAL PROJECT COST, FY-07			**$6,820,000**

Alternative 1: Sediment Removal (2ft) - Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 9.
Hancock County Streams - Cowan Bayou - Alternative 1. Sediment Removal (2ft).
Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Hancock County Communites	ITEM NO.	1	DATE	22-Apr-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Cowan Bayou Removal 2 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m hancock communities4-22.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Cowan Bayou Removal 2 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	82,700	cy	20.00	1,654,000
Misc. Site Items	1	ls	allow	40,000
Total Direct Construction Cost				$1,744,000
Indirect Cost	@		**15%**	261,600
				2,005,600
Profit	@		**9%**	180,504
				2,186,104
Bond	@		**1.5%**	32,792
Current Contract Cost, Oct 06				$2,218,896
01 Account, Lands & Damage (PCA)			**LS**	0
				2,218,896
30 Account, Plan, Engr.& Design			**10%**	221,890
				2,440,785
31 Account, Constr. Management			**6%**	146,447
				2,587,232
CONTINGENCY			**20%**	517,446
				3,104,679
ESCALATION, FY-07			**1%**	31,047
				$3,135,725
				rounded
TOTAL PROJECT COST, FY-07				**$3,140,000**

Alternative 1: Sediment Removal (2 ft) - Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 10.
Hancock County Streams - Alternative 1. Sediment Removal (2ft).
Maintenance Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Coastal MS Study, Hancock County Communites | ITEM NO. 1 | DATE 22-Apr-06
LOCATION: Hancock County MS. | SHEET NO. 2 | OF 3
PREPARED: Parmer | CHECKED: Ellsworth
WORK ITEM: ***Hancock County Drainage Canal Removal*** | BASIS of ESTIMATE: Info furnished per PDT Team
FILE NAME: o-m hancock communities 4-22.xls

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Hancock County Drainage Canal Removal 2 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	74,300	cy	20.00	1,486,000
Misc. Site Items	1	ls	allow	40,000
Total Direct Construction Cost				$1,576,000
Indirect Cost	@		15%	236,400
				1,812,400
Profit	@		9%	163,116
				1,975,516
Bond	@		1.5%	29,633
Current Contract Cost, Oct 06				$2,005,149
01 Account, Lands & Damage (PCA)			LS	0
				2,005,149
30 Account, Plan, Engr.& Design			10%	200,515
				2,205,664
31 Account, Constr. Management			6%	132,340
				2,338,003
CONTINGENCY			20%	467,601
				2,805,604
ESCALATION, FY-07			1%	28,056
				$2,833,660
				rounded
TOTAL PROJECT COST, FY-07				**$2,830,000**

Table 11.
Hancock County Streams - Hancock County Cowan Bayou and Hancock County Canals Sediment Removal (1ft).
Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Hancock County Streams	ITEM NO.	1	DATE	27-Jul-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Cowan Bayou and Hancock County Canals***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	hancock communities6-26.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Cowan Bayou and Hancock County Canals 1 Ft.					
Mobilization, Preparatory Work, Demobilization		1	job	allow	100,000
excavation		78,600	cy	20.00	1,572,000
Misc. Site Items		1	ls	allow	80,000
	Total Direct Construction Cost				$1,752,000
	Indirect Cost	@		15%	262,800
					2,014,800
	Profit	@		9%	181,332
					2,196,132
	Bond	@		1.5%	32,942
09 Account, Channels & Canals		Current Contract Cost, Oct 06			$2,229,074
	01 Account, Lands & Damage (PCA)			LS	862,500
					3,091,574
	30 Account, Plan, Engr.& Design			8%	247,326
					3,338,900
	31 Account, Constr. Management			6%	200,334
					3,539,234
	CONTINGENCY			20%	535,347
					4,074,581
					$4,074,581
					rounded
	TOTAL PROJECT COST, FY-07				**$4,070,000**

Alternative 2: Sediment Removal (1 ft) - Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 12.
Hancock County Streams - Cowan Bayou - Alternative 2. Sediment Removal (1ft).
Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Hancock County Communites	ITEM NO.	1	DATE	22-Apr-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Cowan Bayou Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m hancock communities4-22.xls		

D E S C R I P T I O N	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Cowan Bayou Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	41,400	cy	20.00	828,000
Misc. Site Items	1	ls	allow	40,000
Total Direct Construction Cost				$918,000
Indirect Cost	@		**15%**	137,700
				1,055,700
Profit	@		**9%**	95,013
				1,150,713
Bond	@		**1.5%**	17,261
Current Contract Cost, Oct 06				$1,167,974
01 Account, Lands & Damage (PCA)			**LS**	0
				1,167,974
30 Account, Plan, Engr.& Design			**10%**	116,797
				1,284,771
31 Account, Constr. Management			**6%**	77,086
				1,361,857
CONTINGENCY			**20%**	272,371
				1,634,229
ESCALATION, FY-07			**1%**	16,342
				$1,650,571
				rounded
TOTAL PROJECT COST, FY-07				**$1,650,000**

Alternative 2: Sediment Removal (1 ft) - Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 13.
Hancock County Streams - Hancock County Drainage Canals - Alternative 2. Sediment Removal (1ft).
Maintenance Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Coastal MS Study, Hancock County Communites — ITEM NO. 1 — DATE 22-Apr-06
LOCATION: Hancock County MS. — SHEET NO. 2 — OF 3
PREPARED: Parmer — CHECKED: Ellsworth
WORK ITEM: ***Hancock County Drainage Canal Removal*** — BASIS of ESTIMATE: Info furnished per PDT Team
FILE NAME: o-m hancock communities 4-22.xls

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Hancock County Drainage Canal Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	37,200	cy	20.00	744,000
Misc. Site Items	1	ls	allow	40,000
Total Direct Construction Cost				$834,000
Indirect Cost	@		15%	125,100
				959,100
Profit	@		9%	86,319
				1,045,419
Bond	@		1.5%	15,681
Current Contract Cost, Oct 06				$1,061,100
01 Account, Lands & Damage (PCA)			LS	0
				1,061,100
30 Account, Plan, Engr.& Design			10%	106,110
				1,167,210
31 Account, Constr. Management			6%	70,033
				1,237,243
CONTINGENCY			20%	247,449
				1,484,692
ESCALATION, FY-07			1%	14,847
				$1,499,538
				rounded
TOTAL PROJECT COST, FY-07				**$1,500,000**

A design and construction schedule is shown below. The schedule assumes adequate surveys will be obtained.

Table 14.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report, FEMA (URS Group, Inc.), 16 January 2006.

Draft Report, Hurricane Katrina Flood Frequency Analysis, FEMA, September 2005.

NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b).

JACKSON MARSH

General

The purpose of this document is to provide engineering information for planning and design of environmental restoration measures and interior drainage infrastructure for areas damaged by Hurricane Katrina near the Bay St. Louis and Waveland communities of Hancock County, Mississippi.

Location

The study shoreline areas are located in Hancock County, the western-most coastal county in Mississippi, between Bay St. Louis and Bayou Cadet. It is located on Mississippi Sound about 95 miles west of Mobile, Alabama and about 50 miles east of New Orleans, Louisiana.

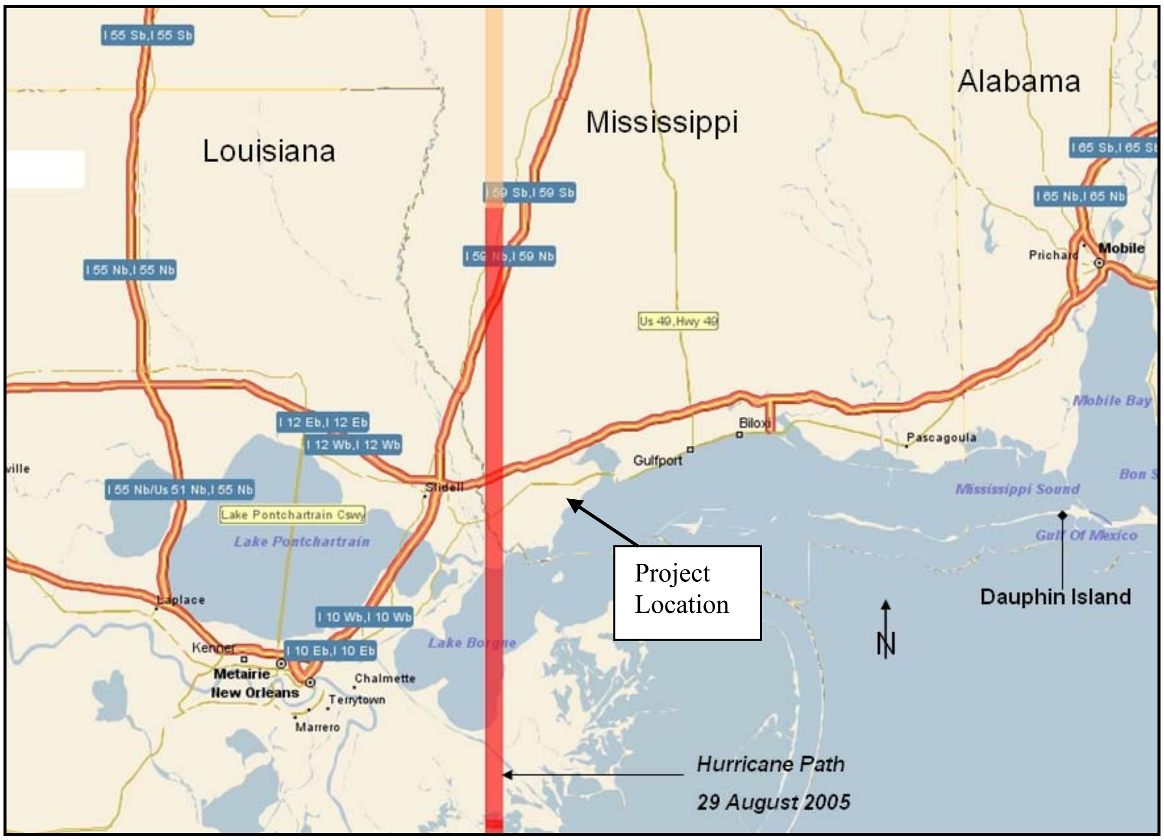


Figure 1. Location Map Showing Path of Hurricane Katrina.

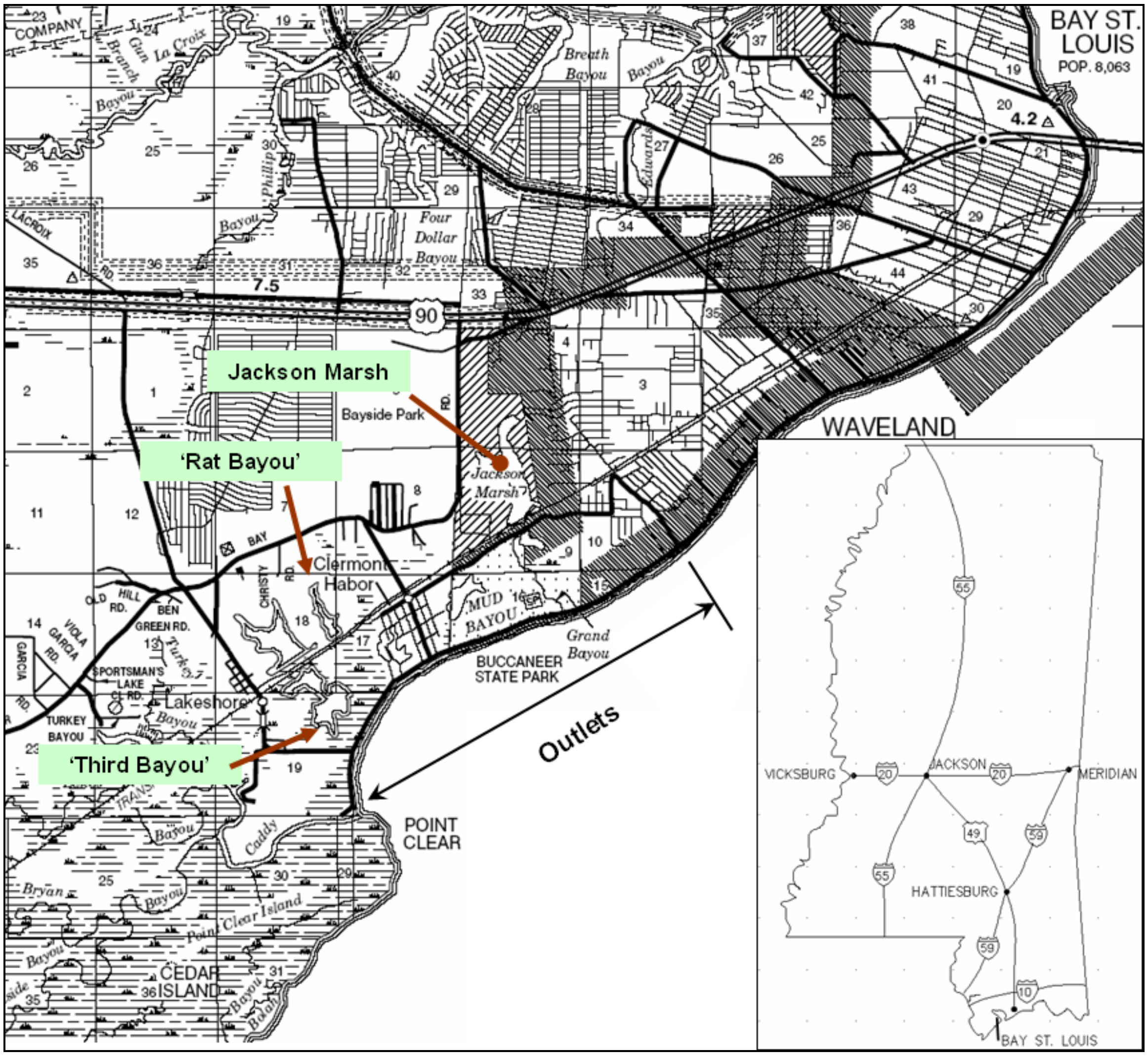


Figure 2. Area Map.

The site location is shown on Figures 2, 3, and 4. The Hancock County shoreline running between Bayou Caddy and Waveland is fronted by Beach Boulevard, which is protected by a concrete seawall and existing beach. The project sites are seaward of Beach Boulevard at the culvert outlets, replacing existing damaged guide walls that extend some 150 feet to the water's edge.

Existing Conditions

The existing Mississippi Sound shoreline in the area is protected by a concrete stepped-face structure about 8 miles long. The seawall was constructed by local interests at various times between 1915 and 1928. Hydrographic and topographic survey data was obtained by the Mobile District under contract in September, 2003. The top elevation of the seawall varies between +3.8 to +5.0 feet National Geodetic Vertical Datum (NGVD). A sand beach was pumped into place on the along about six miles of this seawall in early 1967 as part of the emergency repair and protection following Hurricane Betsy (September 1965). There is another beach extending for about a mile south of the U.S. Highway 90 Bridge crossing the mouth of St. Louis Bay that was placed by the

Mississippi Highway Department during the bridge construction. An additional one-mile-long segment of beach was constructed in the summer of 2005 from Cadet Bayou eastward.

South Beach Boulevard is the main thoroughfare along the entire length of the existing seawall. Historical as well as current wave attack against the shoreline of Hancock County has caused migration of soil through or under the seawall and scour of soil below the seawall in various locations, resulting in damages to South Beach Boulevard and other infrastructure. Sections of the highway have collapsed from time to time, disrupting and damaging utilities, and causing hazards and delays for residents and vehicular traffic. Hancock County has frequently repaired the seawall and road because of the loss of material from beneath the highway. Damaged utilities which have required repairs include water, sewer, natural gas, electric power, and electronic communications. The Mobile District has constructed a number of new seawall segments along various reaches of the existing seawall to alleviate this soil migration and scour problems in the study area under Sections 14 and 103 authorities.

The seawall is penetrated in a number of locations by open drainage channels. Typically, the components of these drainage channels at their crossings of South Beach Boulevard include concrete headwalls, concrete box culverts beneath the boulevard, and channel extension guide-walls extending out into Mississippi Sound. Many of these were severely damaged by hurricane Katrina. Typical damages included breaching of the extension guidewalls, failure of the guidewalls, and destruction of the outlet end of the box culverts.

There are 16 outlets along Beach Blvd with 12 identified outlets that the guide walls require replacement. That means that there are 24 walls, each 155 feet in length and having a pile length of 15 feet. This should give a total of 55, 800 square feet of piling in place. The pile section should be of sufficient stiffness to no require tiebacks. There will be an average of 5 feet in unsupported length and 10 feet of embedment.

Several tidal marshes exist on the landward side of the roadway on the southwestern end of Hancock County around the Waveland area. The existence of these expansive and contiguous tidal marshlands are maintained through tidal conduits (outfalls) built into the existing seawall at regular intervals. Many of the tidal conduits supporting these marsh areas are in a state of severe deterioration. It is also believed that the much of the tidal flow between Mississippi Sound and the marshes have been critically restricted from sedimentation as a result of Hurricane Katrina. The existence of these valuable marshlands is dependent upon the continuation of the tidal exchange provided by the outfalls. Without the tidal exchange, the marshes would drastically deteriorate and cease to function as a tidal salt marsh. In the short term, clearing and/or reconstruction of tidal outfalls would maintain a minimum tidal flow necessary to sustain salt marshes providing vital stabilization. The overall health of the marshes is likely constrained by the limited water exchange allowed by the tidal conduit system. Reconstruction in a manner that would increase tidal flow may also result in the expansion and restoration of marsh areas that may have been restricted due to the present tidal exchange allowed by the old seawall and tidal conduits. Restoring a greater tidal flow will provide for the restoration, protection, stabilization, and continued existence of the present ecological resources.

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

Mississippi Sound is a shallow coastal lagoon extending 80 miles along the coast of the Gulf of Mexico from Mobile Bay, Alabama westward to Lake Borgne, Louisiana. The average depth in the sound is 10 feet, and 99 percent of the sound is less than 29 feet deep.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in St. Louis Bay is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The Wolf and Jordan Rivers discharge fresh water into opposite sides of the upper portion of St. Louis Bay, with average flows of about 830 and 710 cfs, respectively. The study area has been impacted by several tropical storms and hurricanes, most recently from Hurricane Katrina in 2005. Post-Katrina recovery of high water marks in the area suggest storm surges on the order of 20 to 25 feet or more. Frequency estimates of historic storm tide elevations are shown in Table 2, suggesting surges from Katrina far exceeded the 100-year surge elevation.

Table 2.
Storm Tide Frequency (feet, NGVD)

2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
3.7	5.1	6.6	9.1	11.7	15.1

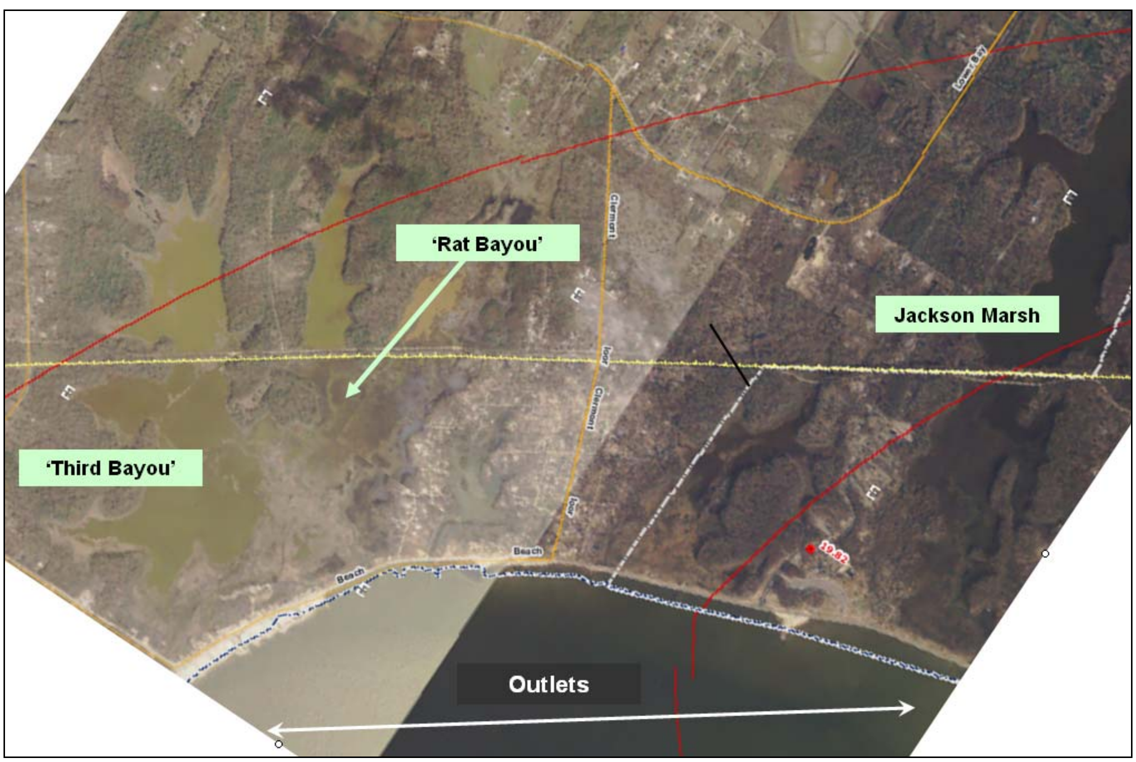


Figure 3. Shoreline West From 'Third Bayou' to Jackson Mark. Hurricane Katrina High Water Marks in Feet NAVD 88 Datum Shown in Red.

Figure 4. Damaged pathway, drainage channel outlet, and outlet bridge, near Waveland.

Figure 5. Close-up of outlet featured in Figure 5. Channel is choked with sand, channel confinement structure destroyed.

Figure 6. Channel outlet. Outlet is breached, extension walls are damaged, and outlet is choked with sand.

Figure 7. Landward entrance of outlet channel drainage culvert.

Figure 8. Looking upstream from culvert entrance.

Geotechnical Data

The project is located along Beach Blvd. within Hancock county running from Bayou Caddy on the west end through Waveland to the east. The twelve outlets provided drainage and tidal exchange between the Mississippi Sound and the fresh water marshes located inland from the beach. The beach road is established at El. 5.0 +, with the beach extending some 150 feet to the water's edge. The outfalls require training walls to maintain the channel integrity form the outlet to the beach water. The existing training walls are in a state of failure and will choke off these outlets if they fail completely. The marshes will be jeopardized quickly if the water exchange is not maintained. The walls will run from the edge of the outlet headwalls to the water's edge. The top of the wall will be placed at El. 5.0 with about 5.0 feet of unsupported length and embedded some 10 feet. The wall should be capable of supporting the backfill without the aid of tiebacks since erosion of the backfill is likely during a storm event. The existing beach subgrade for the support of the walls can be assumed to be poorly graded sands and silty sands from El. 5.0 to El. -15. Medium to dense poorly graded sands and silty sands can be expected below El. 5.0 with more silty sands and possible organic content beyond El. -10.0, becoming less silty beyond El. -15.

The alternative solutions provide for various types of sheetpiling to be driven from the edge of the concrete outlet wall to the beach water contact. The design of the sheetpiles should be based on soils having an in place density of 110 PCF, a cohesion of 300 PSF and an angle of internal friction of 25 degrees. The soils will assume to be saturated below El. 3.0 NGVD. The new walls can be access from the beach on each side. Lateral earth pressure coefficients can be derived from the soil values provided but the wall penetration should be on the order of 2.0 times the unsupported length for any section of wall.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternative Plans

The immediate need is to replace those retaining walls that are in a high state of deterioration and in danger of failing. Those walls that have been recently replaced are basically sound and will only require channel excavations to clear the channel and restore tidal flow. Any reconditioning of the marshes themselves will be studied under the long term analysis.

- Alternative 1. Replace existing training walls at 12 outlet structures with new aluminum sheetpile walls. The total wall length is 155 feet, pile length of 15 feet, pile embedded 10 feet. Pile to have a moment capacity of 10,000 ft-#/ft of wall or greater. Further investigation is needed to find the best product for corrosion and abrasion resistance. Excavate 1,000 CY of sand materials from within the channel and deposit it behind the new walls.
- Alternative 2. Install 155 LF of 55,800 square feet of new sheetpile walls using vinyl sheets at 12 locations. Excavate 1,000 CY of sand materials from within the channel and deposit it behind the new walls.

- Alternative 3. Replace existing training walls at 12 outlet structures with new composite sheetpile walls. The total wall length is 155 feet, pile length of 15 feet, pile embedded 10 feet. Pile to have a moment capacity of 10,590 ft-#/ft of wall or greater, similar to "Creative Pultrusions, INC. SuperLoc 1560". Excavate 1,000 CY of sand materials from within the channel and deposit it behind the new walls.

Construction Procedures and Water Control Plan

The construction plan will be to install the new wall immediately in front of the existing wall and cut the sheets at least 3 feet below the finish grade behind the wall. Each channel outlet will require removal of sand materials and deposited behind the new walls.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Annual maintenance of the repaired system will be required with the outlet channels being cleaned and the replacement of the end pieces after each major event. We estimate that the vinyl systems will require replacement every 12 years and the aluminum system will be replaced every 15 years. The channels will require some 200 cy to be removed every year from storm events.

Cost Estimates

Estimated costs for the alternative plans are shown in Table 3. Quantity estimates are based on drawings and rudimentary field measurements. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 3.
Estimated Costs

Alternatives	Quantity	Unit	Estimated Cost
Aluminum Sheetpiles	55,800	SF	
Unclassified Excavation	1,000	CY	
TOTAL			$4,520,000.
O&M		LS	$40,000.
Vinyl/Composite Sheetpile	55,800	SF	
Unclassified Excavation	1,000	CY	
TOTAL			$3,030,000.
O&M		LS	$40,000.

Schedule and Design for Construction

A typical schedule for preparation of P&S through construction is shown in Table 4 below.

Table 4.
Typical Schedule for P&S

Draft P&S	6 weeks after start
ITR/BCOE review	2 weeks after draft P&S
Final P&S/RTA	3 weeks after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	3 months after NTP

CLERMONT HARBOR

General

The purpose of this document is to provide engineering information for planning and design analysis as related to shore protection at Clermont Harbor in Hancock County, Mississippi.

Location

The study area is located in Hancock County, the westernmost coastal county in Mississippi. It is located on Mississippi Sound about 95 miles west of Mobile, Alabama. and about 50 miles east of New Orleans, Louisiana. The study area extends along a paved road (South Beach Boulevard) for about 2,000 feet from a point approximately 1 mile from the western terminus of South Beach Boulevard. The study area is bordered by Mississippi Sound. The shoreline and associated infrastructure of the study area is afforded some protection by an existing seawall, and South Beach Boulevard runs parallel along the seawall for its entire length. A map of the study area is shown as Figure 1.

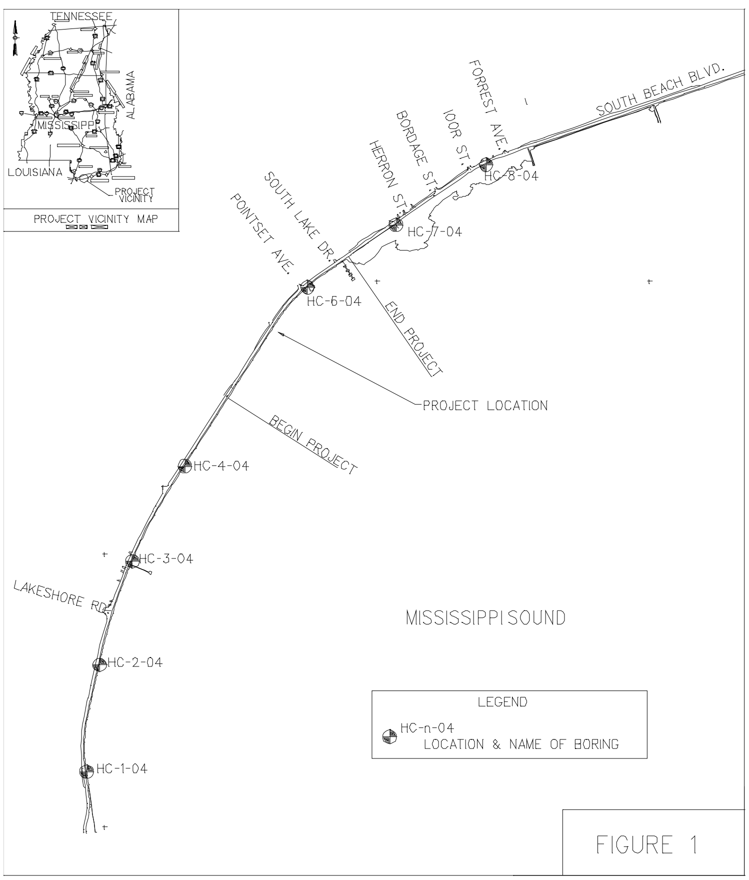


Figure 1. Study Area

Existing Conditions

The existing seawall fronting Mississippi Sound is a concrete stepped-face structure about 8 miles long. Figure 2 is an illustration of existing conditions. The seawall was constructed by local interests at various times between 1915 and 1928. Hydrographic and topographic survey data were obtained by the Mobile District under contract in September, 2003. The top elevation of the seawall varies between +3.8 and +5.0 feet National Geodetic Vertical Datum (NGVD). A sand beach was pumped into place on the Mississippi Sound side along about six miles of this seawall in early 1967 as part of the emergency repair and protection following Hurricane Betsy (September 1965). There is another beach extending for about a mile south of the U.S. Highway 90 bridge crossing the mouth of St. Louis Bay that was placed by the Mississippi Highway Department during the bridge construction. An additional one-mile-long segment of beach was constructed in the summer of 2005 from Cadet Bayou eastward.

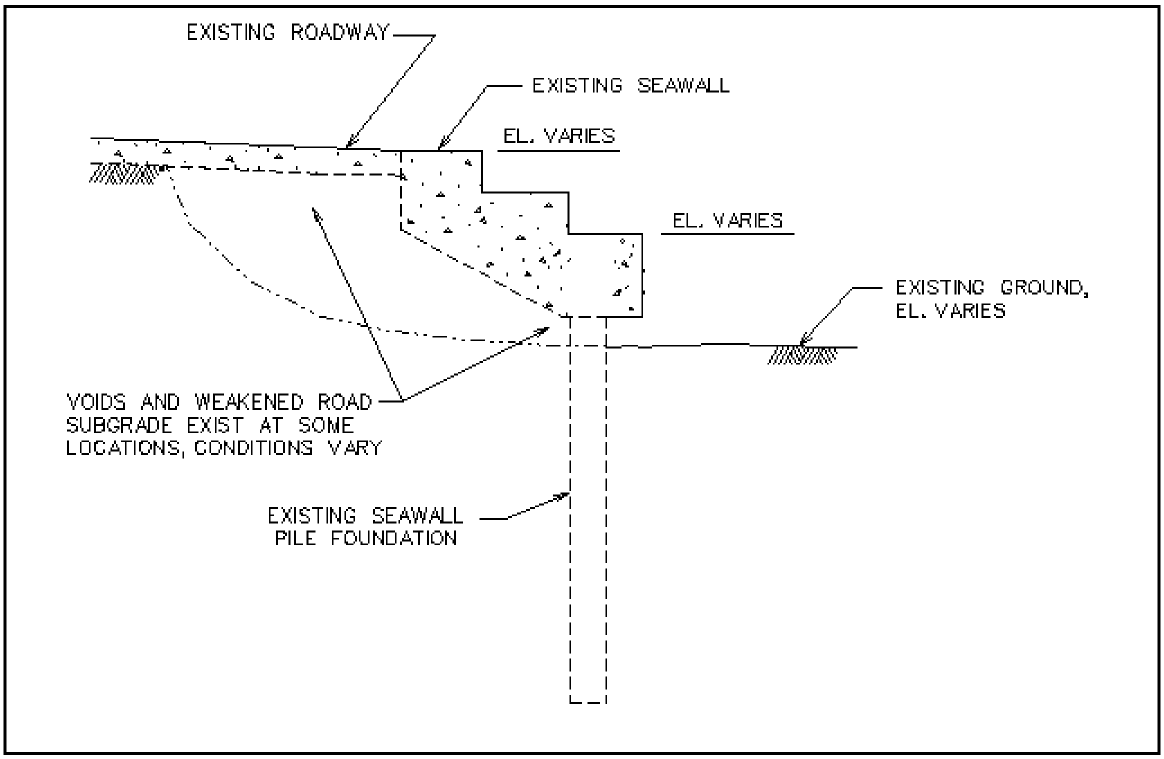


Figure 2. Existing Conditions

South Beach Boulevard is the main thoroughfare along the entire length of the existing seawall. Historical as well as current wave attack against the shoreline of Hancock County have caused migration of soil through or under the seawall and scour of soil below the seawall in various locations, resulting in damages to South Beach Boulevard and other infrastructure. Sections of the highway have collapsed from time to time, disrupting and damaging utilities, and causing hazards and delays for residents and vehicular traffic. Hancock County has frequently repaired the road because of the loss of material from beneath the highway. Damaged utilities which have required repairs include water, sewer, natural gas, electric power, and electronic communications. The Mobile District has constructed a number of projects, consisting of sealing of the seaward face with sheet piling bulkheads, along various reaches of the existing seawall to alleviate these soil migration and scour problems in the study area under Sections 14 and 103 authorities. The configuration of these rehabilitation projects is similar to that shown in Figure 3.

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

Mississippi Sound is a shallow coastal lagoon extending 80 miles along the coast of the Gulf of Mexico from Mobile Bay, Alabama, westward to Lake Borgne, Louisiana. The average depth in the sound is 10 feet, and 99 percent of the sound is less than 29 feet deep. Circulation patterns within the vicinity of the study area are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in St. Louis Bay is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. Freshwater discharge into Mississippi Sound comes primarily from the Pearl River and averages approximately 12,800 cubic feet per second (cfs). Wave heights in Mississippi Sound exceed 5 feet more than 20 percent of the time in winter, but only 5 percent of the time in summer. The study area has been impacted by several tropical storms and hurricanes, most recently from Hurricane Katrina in 2005. Frequency estimates of storm tide elevations are shown in Table 1.

Table 1.
Storm Tide Frequency (Feet, Ngvd)

2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
3.7	5.1	6.6	9.1	11.7	15.1

Geotechnical Data

Subsurface investigations on the seaward side of and close to the existing seawall were conducted in April 2004 by the Mobile District. The boring locations are shown in Figure 1. Predominately sand dredged soil was being placed as beach fill just seaward of the seawall in the southern portion of the study area while the subsurface investigation was being conducted. A rock jetty was recently (2004) constructed near the southern end of the project.

One of the borings (HC-06-04) made in the April 2004 investigation was located within the project limits and another two borings (HC-04-04 and HC-07-04) are located nearby. Two borings (HC-06-04 and HC-07-04) were drilled to a depth of 15 feet and one boring (HC-04-04) was drilled to 30 feet. Splitspoon samples were taken on 1.5-foot intervals to a depth of 15 feet and on 3-foot centers where drilled to depths greater than 15 feet. Standard Penetration Tests (SPT) were made during drilling. Splitspoon samples were visually classified in accordance with the Unified Soils Classification System and placed in jars. Surface water depths were recorded at the borings located in water. Groundwater depth measurements at the boring locations on land were not initially recorded because of the drilling method used, but groundwater was encountered at shallow depths. The boreholes would have collapsed had drilling been delayed to obtain groundwater readings in these borings. The groundwater depths shown on the drilling logs were estimated at these locations. Ground surface elevation at each boring was measured relative to the top step of the seawall at each boring. No laboratory testing was made. The ground surface just seaward of the seawall was probed over the length of the investigated area. Concrete rubble was found in several areas by the probing.

Soils encountered at and near the project shallower than approximately El. -15 feet MLLW were predominately classified as poorly graded sands (SP) and silty sand (SM). Highly plastic clays (CH), clayey sand (SC), silty sand (SM), and poorly graded sands (SP) were found at deeper locations in boring HC-04-04. A zone of very soft soil about 14 feet thick was encountered 2 of the 3 borings. The very soft zone extended downward from ground surface at boring HC-04-04 and downward from 3 feet below ground surface at boring HC-06-04. Rock was not encountered in the borings at this project site. Rock boulders and cobbles, not identified in the borings or in the survey of Aug 2003, were present at a recently constructed rock jetty when observed during the 2004 investigation. Groundwater levels at the borings vary approximately as the water levels in the Mississippi Sound where the borings are located. Concrete rubble was encountered at one boring (HC-07-04). Concrete rubble identified by observation and probing is present at some other locations at the project site.

HTRW

HTRW issues for this project are addressed in a separate addendum to this report.

Alternative Plans

Three plans were evaluated for shore protection at the study site.

- Steel sheet piling. This alternative would consist of the installation of continuous interlocked steel sheet piling along the face of the lower-most step of the existing stepped seawall for the entire project length of approximately 2,000 feet. The sheet pile bulkhead would be anchored to the seawall face using steel rock anchors; the void behind the bulkhead would be backfilled with gravel and sealed at the top with a reinforced concrete cap.
- Vinyl sheet piling. The vinyl sheet pile alternative is essentially the same as the steel alternative except for the sheet pile material. There would some different considerations for material thickness and anchorage spacing, but otherwise the plans would be very similar. Figure 3 depicts the arrangement for both the steel and vinyl sheet pile alternatives.
- Stone revetment. The revetment alternative would employ graded riprap in a dike configuration to provide protection of the existing seawall from wave action. The stone would be placed adjacent to the seawall and underlain with filter fabric to prevent migration of foundation material from behind the wall. Figure 4 is a cross section of the stone revetment alternative.

The plan selected for recommendation is the vinyl sheet pile plan described above.

Construction Procedures and Water Control Plan

Construction of the recommended plan will entail installation of sheet piling along the face of the existing seawall, then anchorage of the piling to the bottom step of the wall, backfilling of the void behind the sheet piling and placement of a cast-in-place concrete cap atop the sheet piling. The construction can be accomplished “in the dry” and thus will not require control of groundwater or surface runoff.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operation and Maintenance

Requirements for operation and maintenance will be minimal, as much of the project will be located below water and the portion above water will consist of concrete. Periodic inspection should be conducted for signs of cracking or spalling of the concrete cap and damage to the sheetpile bulkhead from waterborne debris. It is expected that continual deterioration of the concrete cap and occasional damage to the sheetpile bulkhead will result in the necessity to repair concrete spalls approximately 5 times a year and repair damaged sheetpiling sheets at a rate of 5 each year.

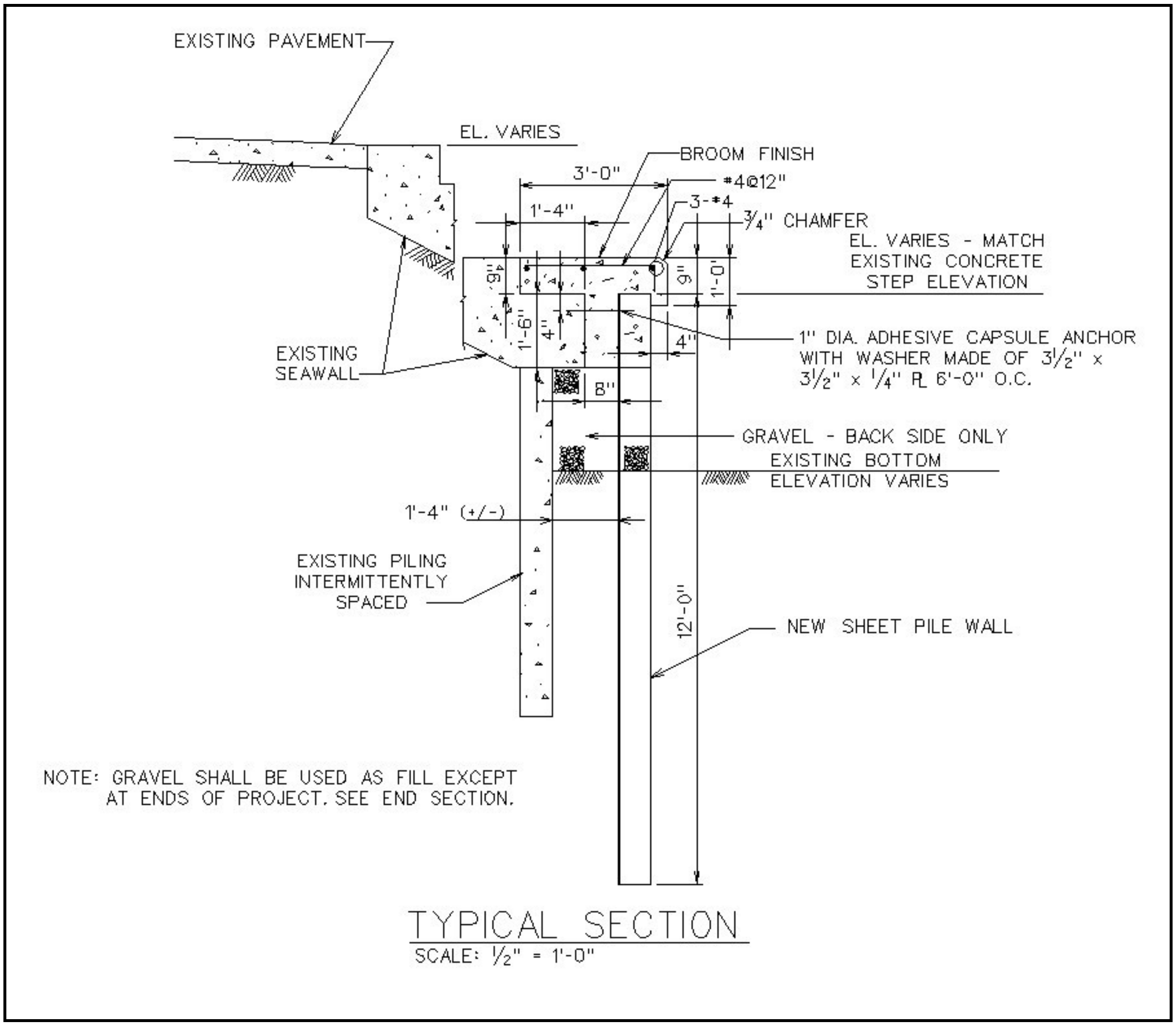


Figure 3. Arrangement for Steel and Vinyl Sheetpile Alternatives

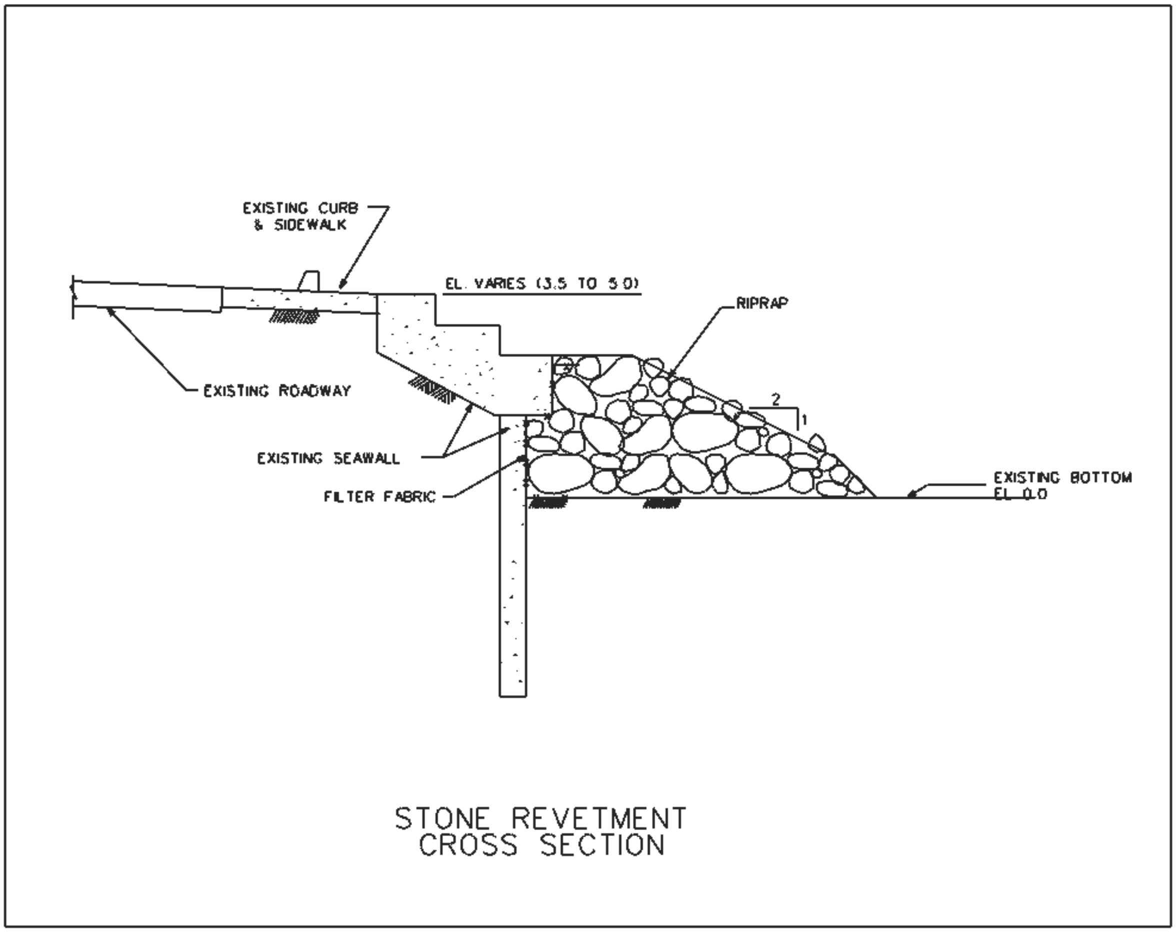


Figure 4. Cross Section of the Stone Revetment Alternative

Cost Estimates

Estimated costs for each alternative plan are shown in Table 2. Quantity estimates are based on topographic surveys as previously discussed. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 2.
Estimated Costs

Alternatives	Quantity	Unit	Estimated Cost
Riprap Revetment	1	LS	$560,000
Vinyl Sheet pile	1	LS	$1,350,000
Steel Sheet pile	1	LS	$1,680,000

Schedule for Design and Construction

A typical schedule for preparation of P&S through construction is shown in Table 3 below.

Table 3.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

U.S. Army Corps of Engineers, "Design of Coastal Revetments, Seawalls, and Bulkheads," Engineer Manual No. 1110-2-1614.

U.S. Army Corps of Engineers, "Engineering and Design for Civil Works Projects," Engineer Regulation No. 1110-2-1150.

U.S. Army Corps of Engineers, "Hydraulic Design for Coastal Shore Protection Projects," Engineer Regulation No. 1110-2-1407.

ADDENDUM 1 – LOGS OF BORINGS AND TEST DATA

GENERAL NOTES:

1. GROUNDWATER DEPTHS OR ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER ENCOUNTERED ON THE DATES SHOWN. ABSENCE OF GROUNDWATER DATA ON CERTAIN BORINGS IMPLIES THAT NO DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THE LOCATIONS. GROUNDWATER ELEVATIONS VARY AND SEEPAGE ABOVE THE DEPTHS OR ELEVATIONS SHOWN CAN BE EXPECTED AT ANY TIME.

2. WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL MINOR VARIATIONS IN CHARACTERISTICS OF THE SUBSURFACE MATERIALS ARE ANTICIPATED AND, IF ENCOUNTERED, SUCH VARIATIONS WILL NOT BE CONSIDERED AS DIFFERING MATERIALLY FROM THE DESCRIPTION SHOWN WITH THE LOGS OR PROFILES.

3. SOILS ARE CLASSIFIED IN ACCORDANCE WITH THE UNIFIED SOIL CLASSIFICATION SYSTEM, TECHNICAL MEMORANDUM NO. 3-357 DATED APRIL 1960 FOR CIVIL PROJECTS AND MILITARY STANDARD 619B DATED 12 JUNE 1968 FOR MILITARY PROJECTS.

4. DRIVING RESISTANCES (BLOW COUNTS OR N VALUES) ARE DETERMINED WITH A STANDARD SPLIT SPOON SAMPLER (1-3/8" I.D.) AND A 140-LB DRIVING HAMMER WITH A 30" DROP UNLESS OTHERWISE NOTED ON THE BORING LOGS. N VALUES SHOWN NUMERICALLY ON THE LOGS ARE THE SUM OF BLOWS FOR THE LOWER TWO OF THREE 0.5-FOOT DRIVES THAT MAKE UP THE 1.5-FOOT STANDARD PENETRATION TEST, EXCEPT WHEN REFUSAL OCCURS. REFUSAL OF THE SPLITSPOON IS DEFINED AS 50 BLOWS IN LESS THAN A 0.5-FOOT DRIVE. REFUSAL IS SHOWN ON THE LOGS AS INDICATED IN THE FOLLOWING EXAMPLES:

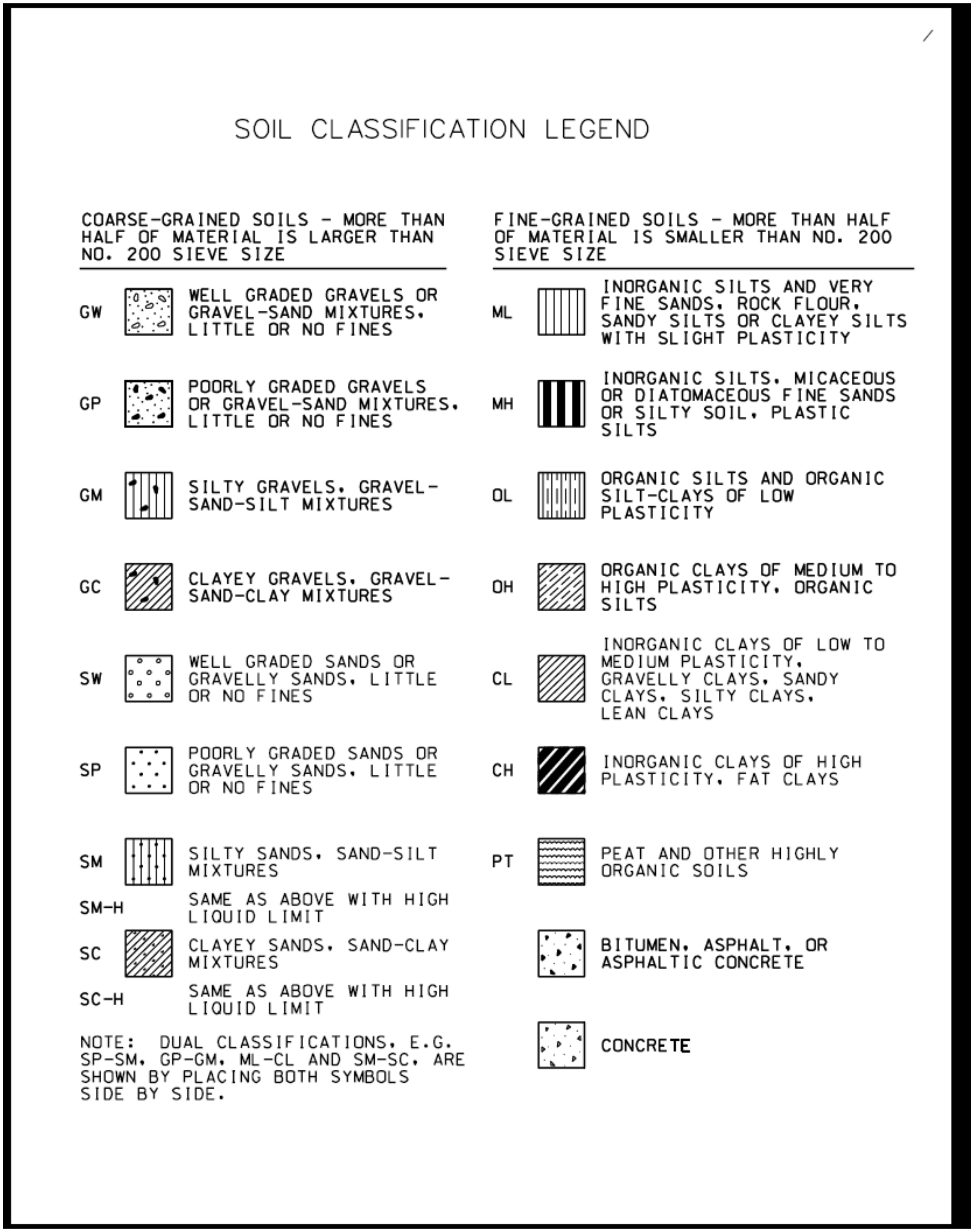
50/0.3' – INDICATES 50 BLOWS (REFUSAL) AFTER 0.3' PENETRATION IN THE FIRST DRIVE.

20, 50/0.2' – INDICATES 20 BLOWS IN THE FIRST DRIVE AND REFUSAL AFTER 0.2' PENETRATION IN THE SECOND DRIVE.

20, 85/0.8' – INDICATES 20 BLOWS IN THE FIRST DRIVE, 35 BLOWS IN THE SECOND DRIVE AND REFUSAL (50 BLOWS) AFTER 0.3' PENETRATION IN THE THIRD DRIVE.

5. "MAX SIZE" OF GRAVEL OR ROCK FRAGMENTS SHOWN ON THE BORING LOGS REPRESENTS THE MAXIMUM SIZE OF MATERIAL RECOVERED IN THE DRIVE SAMPLER AND/OR CORE BARREL OR OBSERVED FROM AUGERING UNLESS OTHERWISE NOTED. NOTE THAT THE MAXIMUM LOGGED SIZE OF GRAVEL OR ROCK FRAGMENTS IS LIKELY TO BE SMALLER THAN THE MAXIMUM SIZE OF THE IN-PLACE MATERIAL, ESPECIALLY WHEN THE MAXIMUM LOGGED SIZE IS MORE THAN APPROXIMATELY ONE-HALF THE DIAMETER OF THE DRIVE SAMPLER OR CORE BARREL, OR MORE THAN ONE-THIRD THE DIAMETER OF THE AUGER.

6. CLASSIFICATIONS SHOWN IN COLUMN D OF THE BORING LOG FORM ARE THE DRILLING INSPECTOR'S FIELD VISUAL CLASSIFICATION OF SAMPLES UNLESS OTHERWISE INDICATED ON THE LOG. WHEN AVAILABLE, LABORATORY CLASSIFICATIONS OF SAMPLES ARE SHOWN IN COLUMN G (REMARKS COLUMN) UNLESS OTHERWISE INDICATED.


SOIL CLASSIFICATION LEGEND
COARSE-GRAINED SOILS - MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE
GW
WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GP
POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GM
SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
GC
CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
SW
WELL GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
SP
POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
SM
SILTY SANDS, SAND-SILT MIXTURES
SM-H
SAME AS ABOVE WITH HIGH LIQUID LIMIT
SC
CLAYEY SANDS, SAND-CLAY MIXTURES
SC-H
SAME AS ABOVE WITH HIGH LIQUID LIMIT
NOTE: DUAL CLASSIFICATIONS, E.G. SP-SM, GP-GM, ML-CL AND SM-SC, ARE SHOWN BY PLACING BOTH SYMBOLS SIDE BY SIDE.
FINE-GRAINED SOILS - MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE
ML
INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SANDY SILTS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
MH
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTY SOIL, PLASTIC SILTS
OL
ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
OH
ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
CL
INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
CH
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
PT
PEAT AND OTHER HIGHLY ORGANIC SOILS
BITUMEN, ASPHALT, OR ASPHALTIC CONCRETE
CONCRETE

ABBREVIATIONS

@	AT
ACCUM	ACCUMULATED
ALT	ALTERNATING
ANG	ANGULAR
APPROX.	APPROXIMATE (LY)
ARGIL	ARGILLACEOUS
AUG	AUGER
AVG	AVERAGE
B.A.	BASE OF ALLUVIUM
B.I.	BREAKAGE INTERVAL
B.O.H.	BOTTOM OF HOLE
BBL	BARREL
BDD	BED (ED) (DING)
BDR	BEDROCK
BENT.	BENTONITIC
BGE	BEIGE
BKY	BLOCKY
BL	BLACK (ISH)
BLD	BOULDER
BR	BROWN (ISH)
BREC.	BRECCIATED
BRK	BROKEN, BREAKAGE
C.D.	CORRECTED DEPTH
CAL	CALCITE, CALCAREOUS
CARB	CARBONACEOUS
CAV	CAVITY
CBL	COBBLE
CEM	CEMENT
CHT	CHERT
CIRCLE.	CIRCULATION
CLY	CLAYEY
CMT'D	CEMENTED
CNTR (S)	CONCENTRATION (S)
COMP	COMPACT
CONC	CONCRETE
CONCR	CONCRETIONS
CONGL	CONGLOMERATE
CONT.	CONTINUED
CR'D	CRUSHED
CRM	CRUMBLY
CSE	COARSE
CTD	COATED
D.	DENSE
d.	DEPTH
D.A.	DRILL ACTION
D.T.	DRILL TIME
D.W.L.	DRILL WATER LOSS
D.W.R.	DRILL WATER RETURN
DECOM	DECOMPOSED
DIAG	DIAGONAL
DIS.	DISSEMINATED
DK	DARK
DOL.	DOLOMITE, DOLOMITIC
DRL	DRILLING
DSTG	DISINTEGRATE (D)
EL	ELEVATION
ENC	ENCOUNTERED
EST	ESTIMATE (D)
EXCL	EXCLUDING
EXTR	EXTREMELY
F.	FINE (LY)
F.R.	FLUID RETURN
F/T	FISHTAILED
FE	IRON
FERR	FERRUGINOUS
FIS	FISSILE
FLD	FILLED
FM	FORMATION

ABBREVIATIONS

FOLIA.	FOLIATION
FOS	FOSSIL (IFEROUS)
FRAC	FRACTURE
FRAG	FRAGMENT (S)
G.W.	GROUNDWATER
GEN.	GENERALLY
GLAU	GLAUCONITE (ITIC)
GR	GRAY (ISH)
GRA	GRAIN (ED)
GRAD	GRADATIONAL
GRN	GREEN (ISH)
GRT	GROUT
GVL	GRAVEL (LY)
GYP	GYPSUM
H/A	HIGH ANGLE
H/B	HAMMER BREAK
HD	HARD
HI	HIGH (LY)
HLD	HEALED
HMR	HAMMER
HOR	HORIZONTAL
HYD	HYDRAULIC
INCL	INCLUDING (ED)
INDT	INDURATED
INIT	INITIAL (LY)
INTBDD	INTERBED (DED)
INTLAM	INTERLAMINATED
IRR	IRREGULAR (LY)
JT'S	JOINT'S
JTD	JOINTED
L.C.	LOSE CORE
L.D.W.	LOST DRILL WATER
L/A	LOW ANGLE
LAB.	LABOR
LAM	LAMINATED, LAMINA (NAE)
LAY.	LAYER
LEA	LEACHED
LGE	LARGE
LIG	LIGNITIC
LIT	LITTLE
LL	LIQUID LIMIT
LN. (S)	LENSE (S)
LO	LOOSE
LS	LIMESTONE
LT	LIGHT
MAS	MASSIVE
MAX	MAXIMUM
MECH	MECHANICAL
MED	MEDIUM
MIC	MICACEOUS
MIN	MINIMUM
MINR	MINERALIZED (IZATION)
MIX.	MIXTURE
MOD	MODERATE (D)
MOT	MOTTLED (ING)
MST	MOIST
MTL	MATERIAL
MTX	MATRIX
N/A	NOT APPLICABLE
N/E	NOT ENCOUNTERED
N/R	NO RECOVERY
NOD.	NODULE
NUM	NUMEROUS
OB	OVERBURDEN (UNCLASSIFIED)
OBS	OBSERVED
OCC	OCCASIONAL (LY)
OOL	OOLITE, OOLITIC
OP	OPEN (ED)
OR	ORANGE

ABBREVIATIONS

ORG	ORGANIC
P.S.I.	POUNDS/SQ. IN.
P.T.	PRESSURE TEST
PART.	PARTIALLY
PCS	PIECES
PERTRO	PETROLEUM, PETROLIFEROUS
PHOS	PHOSPHATE (PHOROUS)
PI	PLASTICITY INDEX
PIT	PIT (TED) (TING)
PKT (S)	POCKET (S)
PL	PLASTIC LIMIT
PLA	PLATY
PLAS	PLASTIC
PLN	PLANE
PNK	PINK
PR	POORLY
PRED	PREDOMINATED
PRESS	PRESSURE
PROB	PROBABLE (ABILITY)
PTC	PARTICLES
PTG	PARTING
PUR	PURPLE
QTZ	QUARTZ
QTZE	QUARTZITE
R.Q.D.	ROCK QUALITY DESIGNATION
RBL	RUBBLE
RD	RED (DISH)
REC	RECOVERY
RECEM	RECEMENTED
RND	ROUND (ED)
RTS	ROOTS
S/S	SPLIT
SAP	SAPROLITE
SAT	SATURATED
SCAT.	SCATTEREDLY
SCH (S)	SCHIST (OS)
SD	SAND
SDY	SANDY
SH	SHALE
SI	SILT
SIS	SILTSTONE
SIY	SILTY
SL	SLIGHT (LY)
SLCES	SILICEOUS
SLICK.	SLICKENSIDE
SML	SMALL
SO	SOFT
SOL	SOLUTION (ED) (ING)
SPG	SPECIFIC GRAVITY
SPT	STANDARD PENETRATION TEST
SPT	STANDARD SPLITSPOON
SS	SANDSTONE
ST	STRAIN (ED) (ING)
STF	STIFF
STR	STRUCTURE
STRG	STRINGER
STYL	STYLOLITE (OLITIC)
SUR	SURFACED
T.F.R.	TOP OF FIRM ROCK
T.O.R.	TOP OF ROCK
T.S.R.	TOP OF SOUND ROCK
TEXT.	TEXTURE
THK	THICK
THN	THIN
TI	TIGHT
TN	TAN (NISH)
TR	TRACE
TRP	TRIPOLI

ABBREVIATIONS

UD	UNDISTUBED
UL	UNACCOUNTABLE LOSS
UNACC	UNACCOUNTABLE
UNWEA	UNWEATHERED
V/	VERY
VERT	VERTICAL
VGY	VUGGY
W.C.	WATER CONTENT
W.L.	WATER LEVEL
W/	WITH
W/H	WEIGHT OF HAMMER
W/R	WEIGHT OF ROD
WD	WOOD
WEA	WEATHERED
WG	WEIGH
WHT	WHITE
X-BDD	CROSS-BEDDED
XL	CRYSTAL
XLN	CYRSTALLINE
YEL	YELLOW

Proj. No.: MSE00126 Alt. Proj. No.: CH02R774 Hole No. HC-04-04

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MDO	SHEET 1 OF 2 SHEETS

1. PROJECT HANCOCK CO. SEAWALL COASTAL	10. SIZE AND TYPE OF BIT SPT
2. LOCATION (Coordinates or Station) N 273306' E 798238'	11. ELEVATION DATUM NGVD, FEET COORDINATE DATUM MSE STATE PLANE, NAD83, FEET
3. DRILLING AGENCY MDO	12. MANUFACTORER'S DESIGNATION OF DRILL SWAMP BUGGY ON BARGE
4. HOLE NO. (As shown on drawing title and file number) HC-04-04	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN DISTURBED 14 UNDISTURBED 0
5. NAME OF DRILLER R. REEVES	14. TOTAL NUMBER CORE BOXES 0
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED DEG. FROM VERTICAL	15. ELEVATION GROUNDWATER EL. 0.9'
	16. DATE HOLE STARTED 6 APR 2004 COMPLETED 6 APR 2004
7. THICKNESS OF OVERBURDEN –	17. ELEVATION TOP OF HOLE EL. -1.8'
8. DEPTH DRILLED INTO ROCK –	18. TOTAL CORE RECOVERY FOR BORING
9. TOTAL DEPTH OF HOLE 30.0'	19. SIGNATURE OF INSPECTOR W. SHARP DRAFTED RLN CHECKED RLN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
-1.8	0.0						
			TAN POORLY GRADED SAND (SP)		1	W.L. d.-2.7' OFFSET 10' EAST OF SEAWALL	WOH
			TAN POORLY GRADED SAND (SP)		2		WOH
	3.0		TAN POORLY GRADED SAND (SP)		3		WOH
-7.8	6.0		BLACK POORLY GRADED SAND (SP)		4		1
-9.3	7.5		GRAY & TAN CLAYEY SAND (SC)		5		0
	9.0		TAN SILTY SAND (SM)		6		WOH
			TAN SILTY SAND (SM)		7		WOH
	12.0		TAN SILTY SAND (SM)		8		WOH
-15.3	13.5		TAN SILTY SAND (SM) W/ GRAY SILTY CLAY (CH) LAYERS		9		WOH
-16.8	15.0		GRAY SILTY CLAY (CH)		10		6
-18.3	16.5		GRAY SILTY SAND (SM)				
-19.8	18.0		GRAY SILTY CLAY (CH)		12		10
-21.3	19.5		NO SAMPLE				
-22.8	21.0		GRAY SILTY CLAY (CH)		13		22

ENG FORM 1836 (Facsimile) PROJECT HANCOCK CO. SEAWALL COASTAL HOLE NO. HC-04-04

DRILLING LOG (Cont Sheet) | ELEVATION TOP OF HOLE: EL. -1.8' | Hole No. HC-04-04

PROJECT: COASTAL HANCOCK CO. SEAWALL | INSTALLATION: MDO | SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
-22.8	21.0				13		
-24.3	22.5		NO SAMPLE				
-25.8	24.0		GRAY SILTY CLAY (CH) W/ SILTY SAND (SM) LAYERS		14		43
-27.3	25.5		NO SAMPLE				
-28.8	27.0		GRAY & ORANGE CLAYEY SAND (SC) & ORGANICS		15		30
-30.3	28.5		NO SAMPLE			1/2 SACK OF TIGERSOL SALTWATER DRILLING MUD. BACKFILLED WITH SAND	
-31.8	30.0		TAN SILTY SAND (SM) & ORGANICS		16	B.O.H.	55

ENG FORM 1836-A (Facsimile) | PROJECT: HANCOCK CO. SEAWALL COASTAL | HOLE NO. HC-04-04

Proj. No.: MSE00126 Alt. Proj. No.: CH02R774 Hole No. HC-06-04

DRILLING LOG — DIVISION: SOUTH ATLANTIC	INSTALLATION: MDO — SHEET 1 OF 1 SHEETS
1. PROJECT: HANCOCK CO. SEAWALL COASTAL	10. SIZE AND TYPE OF BIT: SPT
2. LOCATION (Coordinates or Station): N 274960' E 799388'	11. ELEVATION DATUM: NGVD, FEET COORDINATE DATUM: MSE STATE PLANE, NAD83, FEET
3. DRILLING AGENCY: MDO	12. MANUFACTORER'S DESIGNATION OF DRILL: SWAMP BUGGY
4. HOLE NO. (As shown on drawing title and file number): HC-06-04	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 10, UNDISTURBED 0
5. NAME OF DRILLER: J. STAFFERD	14. TOTAL NUMBER CORE BOXES: 0
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED DEG. FROM VERTICAL	15. ELEVATION GROUNDWATER: NOT ENCOUNTERED
7. THICKNESS OF OVERBURDEN: –	16. DATE HOLE: STARTED 14 APR 2004, COMPLETED 14 APR 2004
8. DEPTH DRILLED INTO ROCK: –	17. ELEVATION TOP OF HOLE: EL. 2.5'
9. TOTAL DEPTH OF HOLE: 15.0'	18. TOTAL CORE RECOVERY FOR BORING
	19. SIGNATURE OF INSPECTOR: W. SHARP — DRAFTED RLN, CHECKED RLN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
2.5	0.0						
			WHITE SILTY SAND (SM)			OFFSET 8' SOUTHEAST OF SEAWALL	3
0.2	2.3		WHITE SILTY SAND (SM)				10
-0.3	2.8		ORGANIC SILT (OH)				
-2.0	4.5		BLACK SILTY SAND (SM)				1
-3.5	6.0		BLACK SILT (ML)				0
			DARK GRAY SILTY CLAY (CH)				0
-6.5	9.0		DARK GRAY SILTY CLAY (CH) W/ SAND LENSES				0
			GRAY SILTY SAND (SM)				0
	12.0		GRAY SILTY SAND (SM)				0
			GRAY SILTY SAND (SM)			GROUNDWATER NOT RECORDED	0
-12.5	15.0		GRAY SILTY SAND (SM)			BACKFILLED WITH SAND B.O.H.	4

ENG FORM 1836 (Facsimile) PROJECT: HANCOCK CO. SEAWALL COASTAL HOLE NO. HC-06-04

DOWNTOWN BAY ST. LOUIS

General

The purpose of this document is to provide engineering information for planning and design analysis as related to shore protection at Downtown Bay St. Louis in Hancock County, Mississippi.

Location

The study area is located in Hancock County, the westernmost coastal county in Mississippi, on Mississippi Sound about 95 miles west of Mobile, Alabama and about 50 miles east of New Orleans, Louisiana. The area extends along a paved road (South Beach Boulevard) in the city of Bay St. Louis for about 1 mile south from U S Highway 90. The study area is bordered on the east by the Mississippi Sound. The shoreline and associated infrastructure of the study area is afforded some protection by existing seawalls and bulkheads. Figure 1 is a map of the study area.

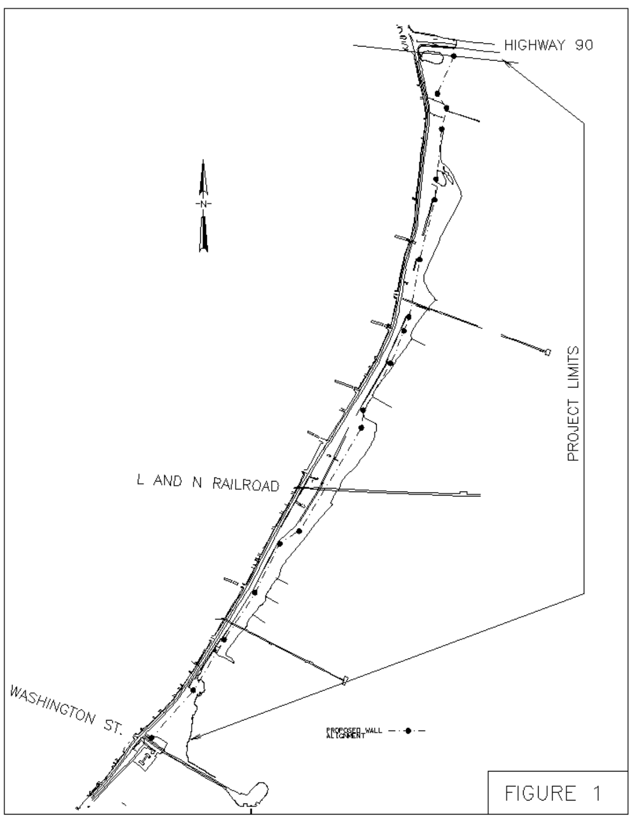


Figure 1. Project Area

Existing Conditions

Existing protective structures fronting the project area include an interlocking concrete paver revetment with a top elevation of about 7 feet at the north end, adjoined by a timber bulkhead with top elevation of about 10 feet to the south, then a section of stepped concrete seawall with top elevation of about 10 feet, and a length of vertical-faced concrete seawall with a top elevation of 10 feet. Hydrographic and topographic survey data were obtained by the Mobile District under contract in September 2003.

South Beach Boulevard was the main thoroughfare along the entire length of the project area. Wave attack from the surge elevation of Hurricane Katrina destroyed South Beach Boulevard and the commercial and residential structures on both sides of the boulevard (see Figures 2 through 4). Utilities located beneath the pavement and adjacent to the street were also lost, including water, sewer, natural gas, electric power, and electronic communications.

Figure 2. South Beach Blvd at Main Street (Shoreward)

Figure 3. South Beach Blvd at Main Street (Northward)

Figure 4. South Beach Blvd Utilities

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

Mississippi Sound is a shallow coastal lagoon extending 80 miles along the coast of the Gulf of Mexico from Mobile Bay, Alabama westward to Lake Borgne, Louisiana. The average depth in the sound is 10 feet, and 99 percent of the sound is less than 29 feet deep.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in St. Louis Bay is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The Wolf and Jordan Rivers discharge fresh water into opposite sides of the upper portion of St. Louis Bay, with average flows of about 830 and 710 cfs, respectively. The study area has been impacted by several tropical storms and hurricanes, most recently from Hurricane Katrina in 2005. Frequency estimates of storm tide elevations are shown in Table 1.

Table 1.
Storm Tide Frequency (feet, NGVD)

2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
3.7	5.1	6.6	9.1	11.7	15.1

Geotechnical Data

No subsurface investigations have been made at the proposed seawall alignment as of April 2006. Subsurface investigation is planned for final design.

Eight splitspooned borings and two offset borings were drilled March 1993 by the USACE Mobile District for a previous Hancock County Section 14 seawall project. Three of the 8 splitspooned borings (HCS-6-93, HCS-7-93 and HCS-8-93) and the two offset borings (HCS-6A-93 and HCS-7A-93) from this investigation are located near the proposed seawall for this project. Subsurface conditions assumed for this report are partially based on conditions indicated by these three splitspooned borings and two offset borings. Logs of these borings are included in Addendum 1. These borings were located on the seaward road shoulder of South Beach Boulevard. The splitspooned borings were drilled to depths varying from 44 to 48 feet and to bottom elevations varying from -23 to -24. Splitspoon samples were taken on irregular intervals above elevation 0 and on 1.5-foot intervals to the bottom elevations. Standard Penetration Tests (SPT) were made during drilling. Splitspoon samples were visually classified in accordance with the Unified Soils Classification System and placed in jars. Undisturbed Shelby tube samples were taken at the two offset borings. Ground surface elevation was measured at each boring location. Laboratory testing including laboratory visual classification, sieve analysis, Atterberg limits, moisture content, and unconsolidated undrained (Q) triaxial compression tests was performed on selected samples. Laboratory testing for the relevant 5 borings is included in Addendum 1.

Soils encountered at the borings were classified mostly as poorly graded sand (SP), but also included inorganic silt (ML), clayey sand (SC), and organic silt (OL). Some of soil classified in the field as inorganic silt (ML) was classified in the laboratory as highly plastic clay (CH). The relative density of the soil varied from very soft / very loose (SPT N=2) to very hard / very dense (SPT N=100+). Soils at approximately elevations -10 to -20 feet were indicated to be mostly soft to medium ML, SC, and OL soils. The soil overlying and underlying this layer to the elevation investigated is predominately dense to hard poorly graded sand (SP). Possibly the relative density of some or all of the hard to very hard sand encountered at the borings is related to foundation grouting or other construction at the road and may not be present at the proposed pile wall. Due to the existing ground surface slope, much of the dense to hard poorly graded sand (SP) that exists at shallow depths at the boring locations likely does not exist at the proposed wall location. Rock was not encountered in any borings. Except during non-steady state conditions during and following rainstorms and overtopping storm surges, groundwater levels at the site will vary approximately as the water levels in Mississippi Sound near where the project is located.

For preliminary foundation design for this report, the soil at the proposed seawall was assumed have the following idealized properties:

Table 2.
Assumed Idealized Soil Properties

Strata El., ft	Soil	SPT N, blows per foot
+2 to -10	Sand	10
-10 to -20	Silt	4
below -20	Sand	30

The assumed subsurface conditions are unsuited for a shallow foundation for all except small seawall heights. For this reason and to minimize risk of seawall failure due to scour beneath the seawall, a seawall structure founded on piles is recommended. A continuous sheetpile wall at the seaward side of the structure is recommended to prevent scour of soil beneath and behind the seawall.

HTRW

HTRW issues for this project are addressed in a separate addendum to this report.

Alternative Plans

Two plans were evaluated for shore protection at the study site.

- Alternative 1: Inverted Reinforced Concrete T-wall on Concrete Pile Foundation. This alternative would consist of the installation of a deep pile foundation with a concrete pile cap which would serve as the base of a steel-reinforced inverted Tee seawall with a maximum top elevation of 20.0. A cross section of this wall configuration is shown in Figure 5.

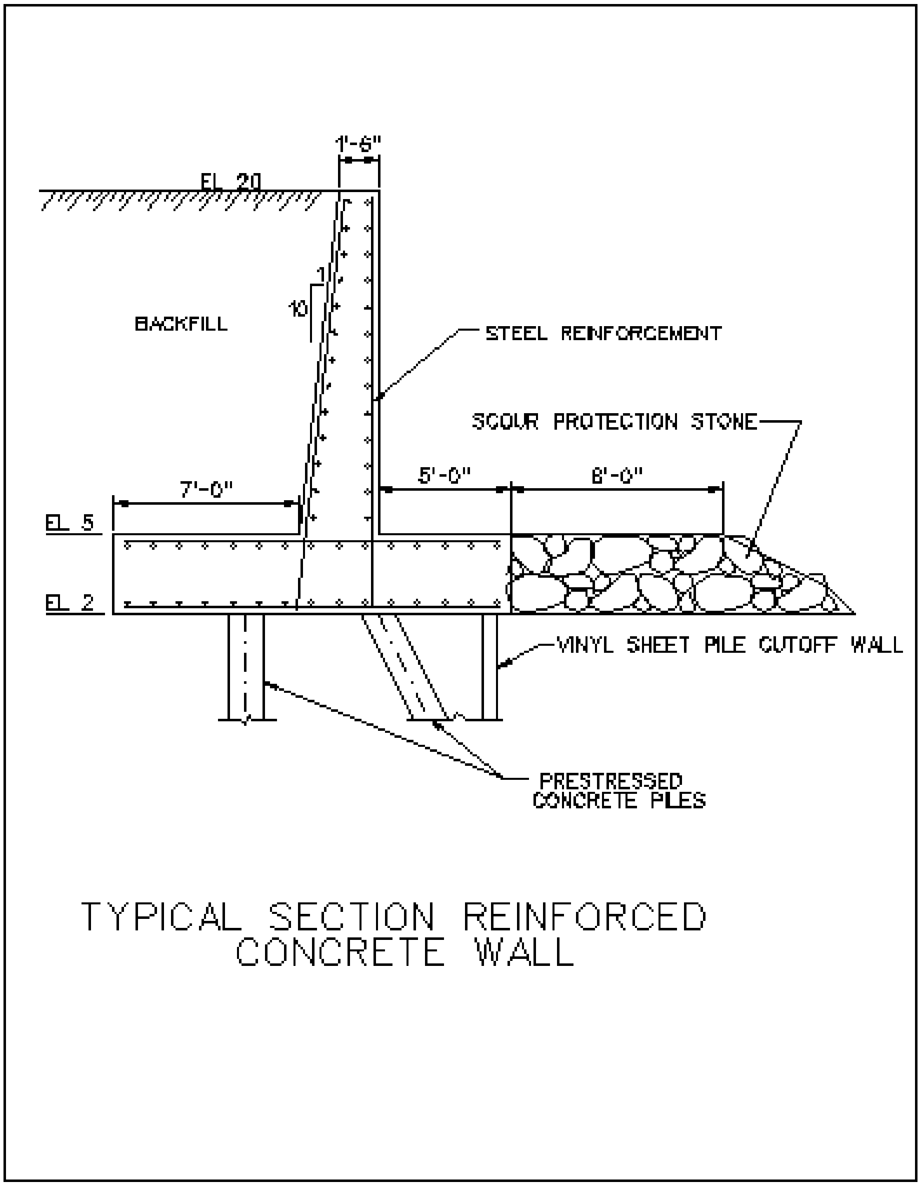


Figure 5. Cross Section of Reinforced Concrete Wall

- Alternative 2: Gravity Concrete Seawall on Concrete Pile Foundation. This alternative would employ unreinforced mass concrete in lieu of steel-reinforced moment-resisting base and stem. Adoption of this alternative would require a greater quantity of concrete but would eliminate the necessity of reinforcing steel in the wall. Figure 6 depicts the configuration of this wall.

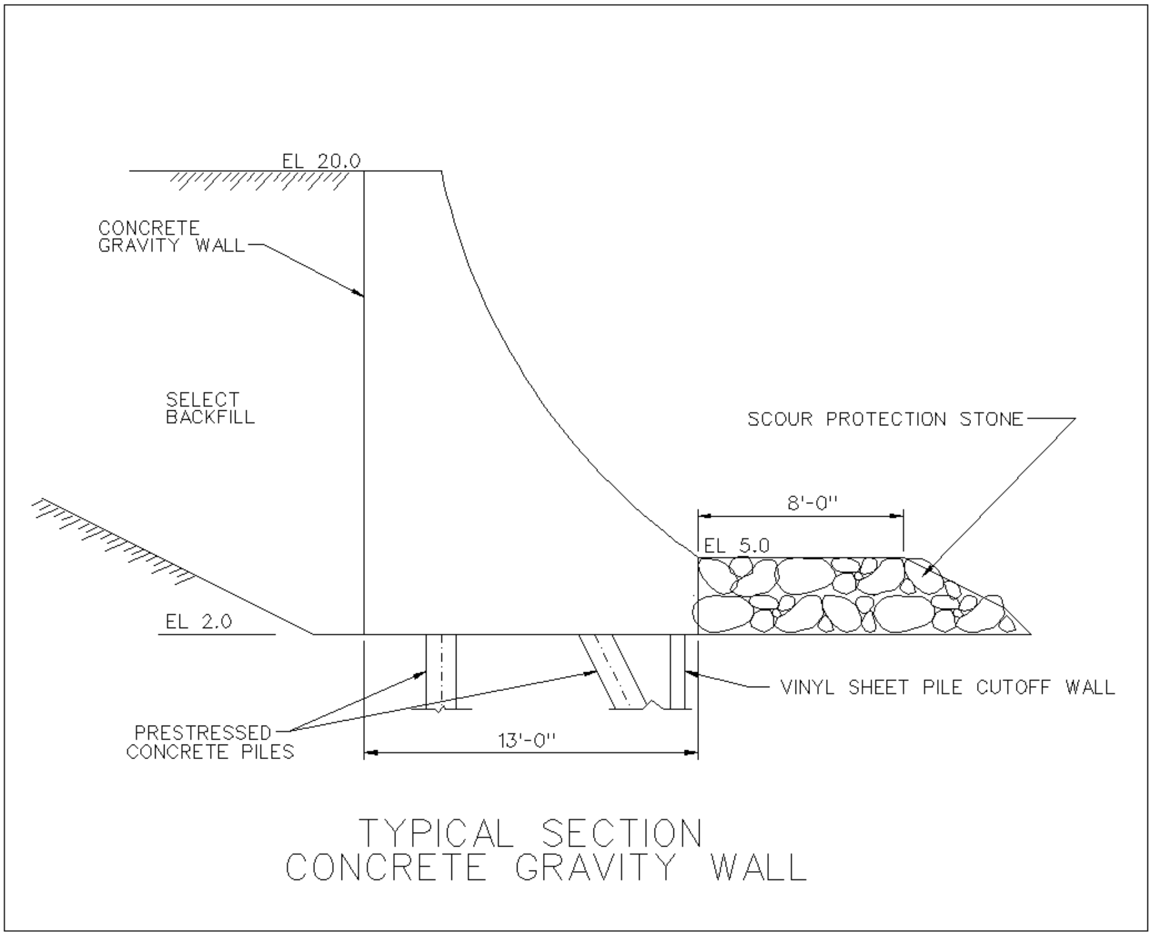


Figure 6. Cross Section of Concrete Gravity Wall

The plan selected for recommendation is the gravity concrete seawall on concrete pile foundation.

Construction Procedure and Water Control Plan

Construction of the recommended plan will begin with installation of the concrete foundation piling and the sheet pile cutoff wall, followed by placement of the gravity concrete seawall. It is anticipated that conditions will allow all construction to be accomplished “in the dry” and require no special control measures for groundwater or surface runoff.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operation and Maintenance

Operation and maintenance activities for this project will be minimal but will include periodic visual inspection of the concrete surfaces for cracking or spalling, monitoring of the backfill drainage system for effectiveness, and periodic replacement of components of the drainage system and displaced scour protection stone. Scour stone replacement is expected to amount to approximately 20 percent of the originally placed quantity every 10 years.

Cost Estimates

Estimated costs for each alternative plan are shown in Table 3. Quantity estimates are based on topographic surveys as previously discussed. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 3.
Estimated Costs

Alternatives	Quantity	Unit	Estimated Cost
Concrete T-Wall	1	LS	$29,400,000
Concrete Gravity Wall	1	LS	$29,140,000

Schedule for Design and Construction

A typical schedule for preparation of P&S through construction is shown in Table 4 below.

Table 4.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	15 months after NTP

References

U.S. Army Corps of Engineers, “Design of Coastal Revetments, Seawalls, and Bulkheads”, Engineer Manual No. 1110-2-1614.

U.S. Army Corps of Engineers, “Engineering and Design for Civil Works Projects”, Engineer Regulation No. 1110-2-1150.

U.S. Army Corps of Engineers, “Hydraulic Design for Coastal Shore Protection Projects”, Engineer Regulation No. 1110-2-1407.

COWAND POINT

General

The purpose of this document is to provide engineering information for planning and design analysis as related to shore protection at Cowand Point in Hancock County, Mississippi.

Location

The study area is located in Hancock County, the westernmost coastal county in Mississippi. It is located on Mississippi Sound about 95 miles west of Mobile, Alabama and about 50 miles east of New Orleans, Louisiana. The study area extends along a paved road (North Beach Boulevard) in the city of Bay St. Louis for about 3 miles north from U S Highway 90. The study area is bordered on the east by St. Louis Bay. The shoreline and associated infrastructure of the study area is afforded some protection by an existing seawall, and North Beach Boulevard runs parallel along the seawall for its entire length. Figure 1 is a map of the study area.

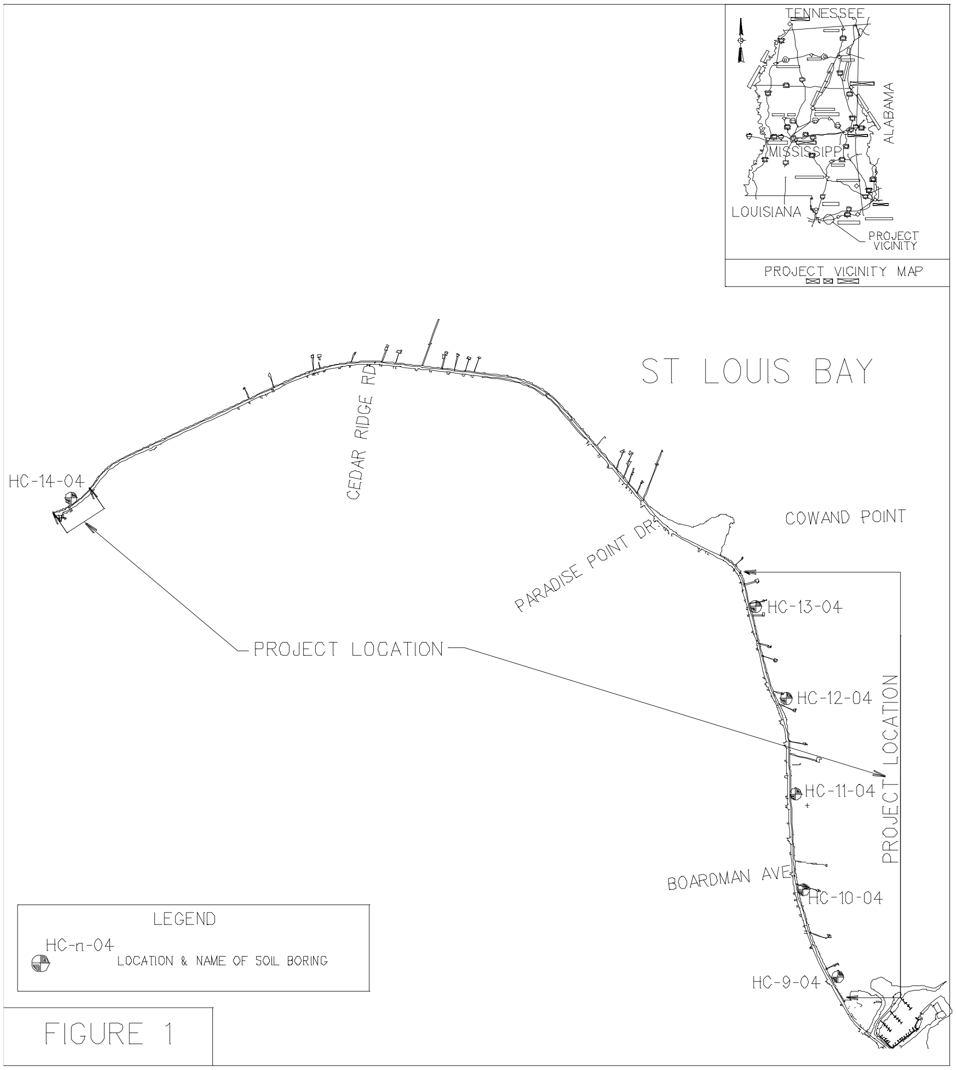


Figure 1. Project Locations

Existing Conditions

The existing seawall fronting St. Louis Bay is a concrete stepped-face structure about 3 miles long. Figure 2 is an illustration of existing conditions. The seawall was constructed by local interests at various times between 1915 and 1928. Hydrographic and topographic survey data was obtained by the Mobile District under contract in September, 2003. The top elevation of the seawall varies between +2.5 and +8.0 feet National Geodetic Vertical Datum (NGVD).

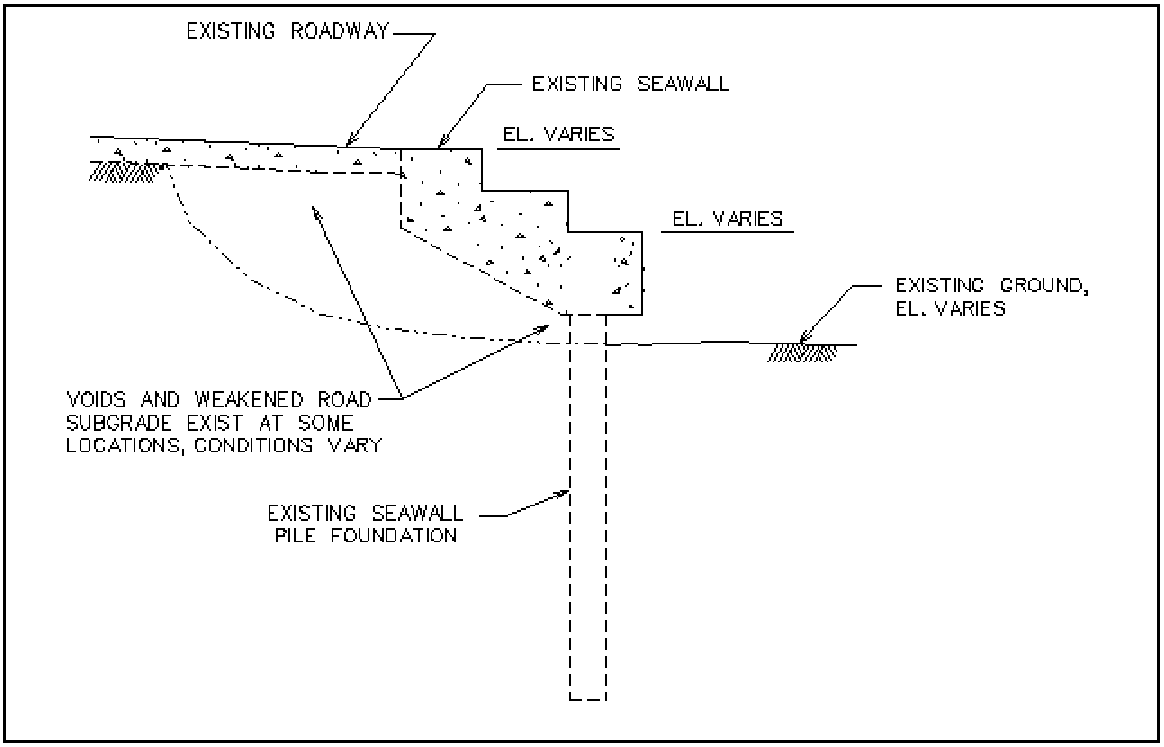


Figure 2. Cross Section of Existing Conditions

North Beach Boulevard is the main thoroughfare along the entire length of the existing seawall. Historical as well as current wave attack against the shoreline of Hancock County have caused migration of soil through or under the seawall and scour of soil below the seawall in various locations, resulting in damages to North Beach Boulevard and other infrastructure. Sections of the roadway have collapsed from time to time, disrupting and damaging utilities, and causing hazards and delays for residents and vehicular traffic. Hancock County has frequently repaired the seawall and road because of the loss of material from beneath the highway. Damaged utilities which have required repairs include water, sewer, natural gas, electric power, and electronic communications. The Mobile District has constructed a number of projects, consisting of sealing of the seaward face with sheet piling bulkheads, along various reaches of the existing seawall to alleviate these soil migration and scour problems in the study area under Sections 14 authority. The locations of these new seawall segments are shown in Figure 1. The configuration of these rehabilitation projects is similar to that shown in Figure 3.

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. Average temperatures are 82 degrees Fahrenheit for the summer months and 53 degrees Fahrenheit for the winter months. The average annual rainfall is about 60 inches, and is fairly evenly distributed throughout the year. Precipitation records also indicate July as the wettest month, while October is the driest.

St. Louis Bay is a shallow basin connected to Mississippi Sound on the south by a tidal pass approximately 1.9 miles in width. Depths in the bay average about 4 to 5 feet.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides, winds, and freshwater discharges. The mean diurnal tide range in St. Louis Bay is 1.6 feet, and the extreme (except during storms) is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The Wolf and Jordan Rivers discharge fresh water into opposite sides of the upper portion of St. Louis Bay, with average flows of about 830 and 710 cfs, respectively. The study area has been impacted by several tropical storms and hurricanes, most recently from Hurricane Katrina in 2005. Frequency estimates of storm tide elevations are shown in Table 1.

Table 1.
Storm Tide Frequency (feet, NGVD)

2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
3.7	5.1	6.6	9.1	11.7	15.1

Geotechnical Data

Subsurface investigations on the seaward side of and close to the existing seawall were conducted in April 2004 by the Mobile District.

Six of the 13 borings made in the April 2004 investigation were located at or near the project. The boring locations are shown in Figure 1. These borings, identified as HC-09-04 to HC-13-04, were drilled to depths ranging from 15 to 30 feet. Splitspoon samples were taken on 1.5-foot intervals to a depth of at least 15 feet and usually were taken on 3-foot centers where they were drilled to depths greater than 15 feet. Standard Penetration Tests (SPT) were made during drilling. Splitspoon samples were visually classified in accordance with the Unified Soils Classification System and placed in jars. Surface water depths were recorded at the borings located in water. Groundwater depth measurements at the boring locations on land were not initially recorded because of the drilling method used, but groundwater was encountered at shallow depths. The boreholes would have collapsed had drilling been delayed to obtain groundwater readings in these borings. The groundwater depths shown on the drilling logs were estimated at these locations. Ground surface elevation at each boring was measured relative to the top step of the seawall at each boring. No laboratory testing was made. The ground surface just seaward of the seawall was probed over the length of the investigated area. Concrete slabs and rubble were found in several areas.

Soils encountered at the project were predominately classified as poorly graded sands (SP) and silty sand (SM), clays (CL and CH), and clayey sand (SC). Zones of varying thicknesses of very soft soils were encountered in all except one boring (HC-11-04). The thickness of the very soft soil is at least 15 feet and possibly more at boring HC-12-04. Soil in most areas of the site generally consist of a layer of very soft or very loose soil underlain by generally loose to moderately compact sand or sandy soil. Rock was not encountered in any borings except as an anomalous 1-inch thick layer in one boring (HC-13-04), underlain by soil. Groundwater levels at the borings vary approximately as the water levels in Bay St. Louis where the borings are located. Concrete slabs and concrete rubble (not identified in borings) are present at the surface at some locations at the project site. Their locations near what appear to be filled grout holes at the road suggest the concrete slabs were formed when pressure grouting was performed to fill voids under the road.

HTRW

HTRW issues for this project are addressed in a separate addendum to this report.

Alternative Plans

Three plans were evaluated for shore protection at the study site.

- Steel sheet piling. This alternative would consist of the installation of continuous interlocked steel sheet piling along the face of the lower-most step of the existing stepped seawall for the entire project length of approximately 5,000 feet. The sheet pile bulkhead would be anchored to the seawall face using steel rock anchors; the void behind the bulkhead would be backfilled with gravel and sealed at the top with a reinforced concrete cap.
- Vinyl sheet piling. The vinyl sheet pile alternative is essentially the same as the steel alternative except for the sheet pile material. There would some different considerations for material thickness and anchorage spacing, but otherwise the plans would be very similar. Figure 3 depicts the arrangement for both the steel and vinyl sheet pile alternatives.
- Stone revetment. The revetment alternative would employ graded riprap in a dike configuration to provide protection of the existing seawall from wave action. The stone would be placed adjacent to the seawall and underlain with filter fabric to prevent migration of foundation material from behind the wall.

The plan selected for recommendation is the vinyl sheet pile plan described above.

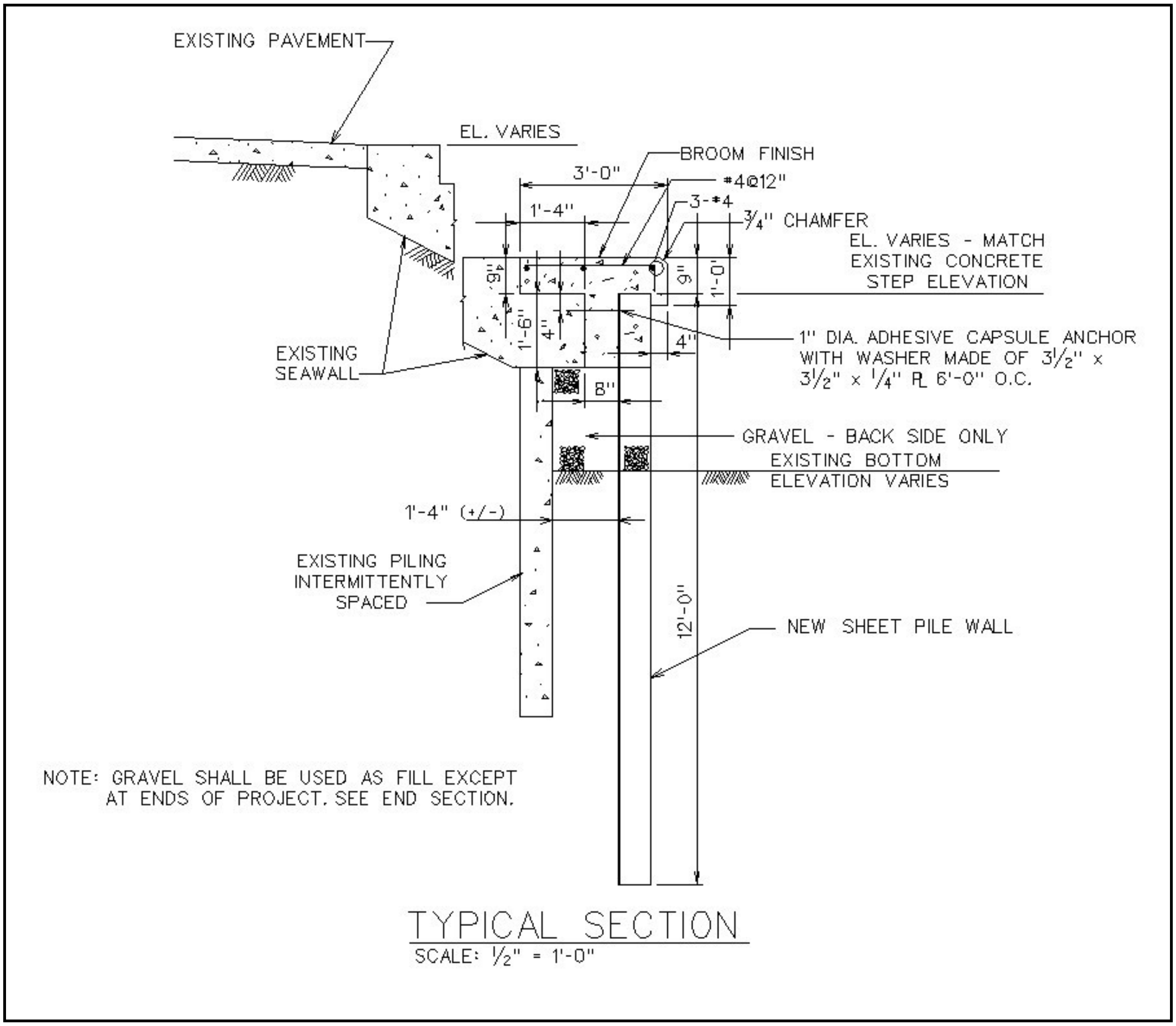


Figure 3. Cross Section for Sheet piling Alternatives

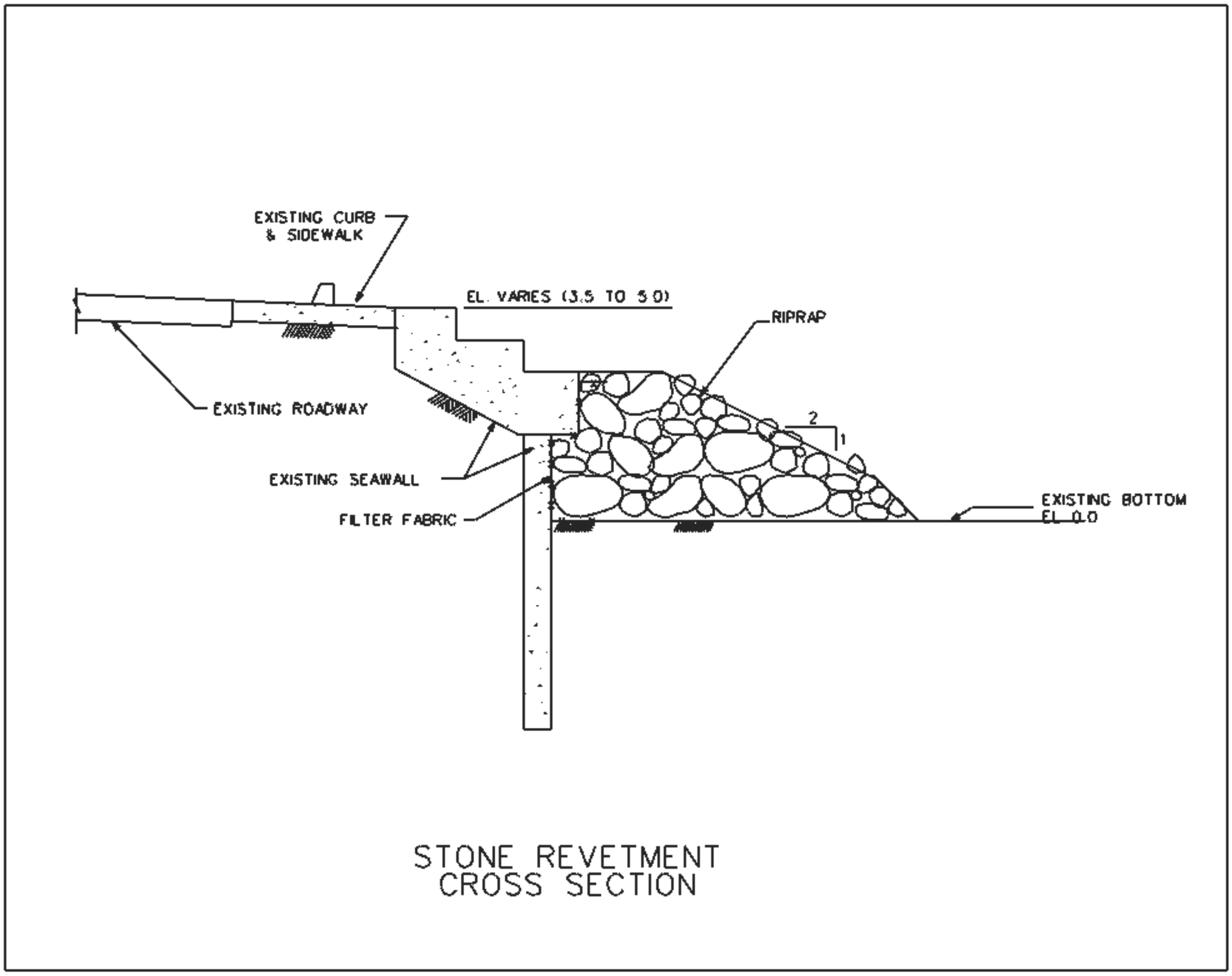


Figure 4. Cross Section of Stone Revetment

Construction Procedures and Water Control Plan

Construction of the recommended plan will entail installation of sheet piling along the face of the existing seawall, then anchorage of the piling to the bottom step of the wall, backfilling of the void behind the sheet piling and placement of a cast-in-place concrete cap atop the sheet piling. The construction can be accomplished “in the dry” and thus will not require control of groundwater or surface runoff.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operation and Maintenance

Requirements for operation and maintenance will be minimal, as much of the project will be located below water and the portion above water will consist of concrete. Periodic inspection should be conducted for signs of cracking or spalling of the concrete cap and damage to the sheet pile bulkhead from waterborne debris. It is expected that continual deterioration of the concrete cap and occasional damage to the sheet pile bulkhead will result in the necessity to repair concrete spalls approximately 10 times a year and repair damaged sheet piling sheets at a rate of 15 each year.

Cost Estimates

Estimated costs for each alternative plan are shown in Table 2. Quantity estimates are based on topographic surveys as previously discussed. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 2.
Estimated Costs

Alternatives	Quantity	Unit	Estimated Cost
Riprap Revetment	1	LS	$1,980,000
Vinyl Sheet pile	1	LS	$3,820,000
Steel Sheet pile	1	LS	$4,900,000

Schedule for Design and Construction

A typical schedule from preparation of P&S through construction is shown in Table 3 below.

Table 3.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

U.S. Army Corps of Engineers, “Design of Coastal Revetments, Seawalls, and Bulkheads,” Engineer Manual No. 1110-2-1614.

U.S. Army Corps of Engineers, “Engineering and Design for Civil Works Projects,” Engineer Regulation No. 1110-2-1150.

U.S. Army Corps of Engineers, “Hydraulic Design for Coastal Shore Protection Projects,” Engineer Regulation No. 1110-2-1407.

ADDENDUM 1 - LOGS OF BORINGS AND TEST DATA

GENERAL NOTES:

1. GROUNDWATER DEPTHS OR ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER ENCOUNTERED ON THE DATES SHOWN. ABSENCE OF GROUNDWATER DATA ON CERTAIN BORINGS IMPLIES THAT NO DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THE LOCATIONS. GROUNDWATER ELEVATIONS VARY AND SEEPAGE ABOVE THE DEPTHS OR ELEVATIONS SHOWN CAN BE EXPECTED AT ANY TIME.

2. WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS AND FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL MINOR VARIATIONS IN CHARACTERISTICS OF THE SUBSURFACE MATERIALS ARE ANTICIPATED AND, IF ENCOUNTERED, SUCH VARIATIONS WILL NOT BE CONSIDERED AS DIFFERING MATERIALLY FROM THE DESCRIPTION SHOWN WITH THE LOGS OR PROFILES.

3. SOILS ARE CLASSIFIED IN ACCORDANCE WITH THE UNIFIED SOIL CLASSIFICATION SYSTEM, TECHNICAL MEMORANDUM NO. 3-357 DATED APRIL 1960 FOR CIVIL PROJECTS AND MILITARY STANDARD 619B DATED 12 JUNE 1968 FOR MILITARY PROJECTS.

4. DRIVING RESISTANCES (BLOW COUNTS OR N VALUES) ARE DETERMINED WITH A STANDARD SPLIT SPOON SAMPLER (1-3/8" I.D.) AND A 140-LB DRIVING HAMMER WITH A 30" DROP UNLESS OTHERWISE NOTED ON THE BORING LOGS. N VALUES SHOWN NUMERICALLY ON THE LOGS ARE THE SUM OF BLOWS FOR THE LOWER TWO OF THREE 0.5-FOOT DRIVES THAT MAKE UP THE 1.5-FOOT STANDARD PENETRATION TEST, EXCEPT WHEN REFUSAL OCCURS. REFUSAL OF THE SPLITSPOON IS DEFINED AS 50 BLOWS IN LESS THAN A 0.5-FOOT DRIVE. REFUSAL IS SHOWN ON THE LOGS AS INDICATED IN THE FOLLOWING EXAMPLES:

50/0.3' - INDICATES 50 BLOWS (REFUSAL) AFTER 0.3' PENETRATION IN THE FIRST DRIVE.

20, 50/0.2' - INDICATES 20 BLOWS IN THE FIRST DRIVE AND REFUSAL AFTER 0.2' PENETRATION IN THE SECOND DRIVE.

20, 85/0.8' - INDICATES 20 BLOWS IN THE FIRST DRIVE, 35 BLOWS IN THE SECOND DRIVE AND REFUSAL (50 BLOWS) AFTER 0.3' PENETRATION IN THE THIRD DRIVE.

5. "MAX SIZE" OF GRAVEL OR ROCK FRAGMENTS SHOWN ON THE BORING LOGS REPRESENTS THE MAXIMUM SIZE OF MATERIAL RECOVERED IN THE DRIVE SAMPLER AND/OR CORE BARREL OR OBSERVED FROM AUGERING UNLESS OTHERWISE NOTED. NOTE THAT THE MAXIMUM LOGGED SIZE OF GRAVEL OR ROCK FRAGMENTS IS LIKELY TO BE SMALLER THAN THE MAXIMUM SIZE OF THE IN-PLACE MATERIAL, ESPECIALLY WHEN THE MAXIMUM LOGGED SIZE IS MORE THAN APPROXIMATELY ONE-HALF THE DIAMETER OF THE DRIVE SAMPLER OR CORE BARREL, OR MORE THAN ONE-THIRD THE DIAMETER OF THE AUGER.

6. CLASSIFICATIONS SHOWN IN COLUMN D OF THE BORING LOG FORM ARE THE DRILLING INSPECTOR'S FIELD VISUAL CLASSIFICATION OF SAMPLES UNLESS OTHERWISE INDICATED ON THE LOG. WHEN AVAILABLE, LABORATORY CLASSIFICATIONS OF SAMPLES ARE SHOWN IN COLUMN G (REMARKS COLUMN) UNLESS OTHERWISE INDICATED.

SOIL CLASSIFICATION LEGEND

COARSE-GRAINED SOILS - MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE

GW	WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
SW	WELL GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
SM	SILTY SANDS, SAND-SILT MIXTURES
SM-H	SAME AS ABOVE WITH HIGH LIQUID LIMIT
SC	CLAYEY SANDS, SAND-CLAY MIXTURES
SC-H	SAME AS ABOVE WITH HIGH LIQUID LIMIT

NOTE: DUAL CLASSIFICATIONS, E.G. SP-SM, GP-GM, ML-CL AND SM-SC, ARE SHOWN BY PLACING BOTH SYMBOLS SIDE BY SIDE.

FINE-GRAINED SOILS - MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE

ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SANDY SILTS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTY SOIL, PLASTIC SILTS
OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
PT	PEAT AND OTHER HIGHLY ORGANIC SOILS
	BITUMEN, ASPHALT, OR ASPHALTIC CONCRETE
	CONCRETE

ABBREVIATIONS

@	AT
ACCUM	ACCUMULATED
ALT	ALTERNATING
ANG	ANGULAR
APPROX.	APPROXIMATE (LY)
ARGIL	ARGILLACEOUS
AUG	AUGER
AVG	AVERAGE
B.A.	BASE OF ALLUVIUM
B.I.	BREAKAGE INTERVAL
B.O.H.	BOTTOM OF HOLE
BBL	BARREL
BDD	BED (ED) (DING)
BDR	BEDROCK
BENT.	BENTONITIC
BGE	BEIGE
BKY	BLOCKY
BL	BLACK (ISH)
BLD	BOULDER
BR	BROWN (ISH)
BREC.	BRECCIATED
BRK	BROKEN. BREAKAGE
C.D.	CORRECTED DEPTH
CAL	CALCITE. CALCAREOUS
CARB	CARBONACEOUS
CAV	CAVITY
CBL	COBBLE
CEM	CEMENT
CHT	CHERT
CIRCLE.	CIRCULATION
CLY	CLAYEY
CMT'D	CEMENTED
CNTR (S)	CONCENTRATION (S)
COMP	COMPACT
CONC	CONCRETE
CONCR	CONCRETIONS
CONGL	CONGLOMERATE
CONT.	CONTINUED
CR'D	CRUSHED
CRM	CRUMBLY
CSE	COARSE
CTD	COATED
D.	DENSE
d.	DEPTH
D.A.	DRILL ACTION
D.T.	DRILL TIME
D.W.L.	DRILL WATER LOSS
D.W.R.	DRILL WATER RETURN
DECOM	DECOMPOSED
DIAG	DIAGONAL
DIS.	DISSEMINATED
DK	DARK
DOL.	DOLOMITE. DOLOMITIC
DRL	DRILLING
DSTG	DISINTEGRATE (D)
EL	ELEVATION
ENC	ENCOUNTERED
EST	ESTIMATE (D)
EXCL	EXCLUDING
EXTR	EXTREMELY
F.	FINE (LY)
F.R.	FLUID RETURN
F/T	FISHTAILED
FE	IRON
FERR	FERRUGINOUS
FIS	FISSILE
FLD	FILLED
FM	FORMATION

ABBREVIATIONS

FOLIA.	FOLIATION
FOS	FOSSIL (IFEROUS)
FRAC	FRACTURE
FRAG	FRAGMENT (S)
G.W.	GROUNDWATER
GEN.	GENERALLY
GLAU	GLAUCONITE (ITIC)
GR	GRAY (ISH)
GRA	GRAIN (ED)
GRAD	GRADATIONAL
GRN	GREEN (ISH)
GRT	GROUT
GVL	GRAVEL (LY)
GYP	GYPSUM
H/A	HIGH ANGLE
H/B	HAMMER BREAK
HD	HARD
HI	HIGH (LY)
HLD	HEALED
HMR	HAMMER
HOR	HORIZONTAL
HYD	HYDRAULIC
INCL	INCLUDING (ED)
INDT	INDURATED
INIT	INITIAL (LY)
INTBDD	INTERBED (DED)
INTLAM	INTERLAMINATED
IRR	IRREGULAR (LY)
JT'S	JOINT'S
JTD	JOINTED
L.C.	LOSE CORE
L.D.W.	LOST DRILL WATER
L/A	LOW ANGLE
LAB.	LABOR
LAM	LAMINATED. LAMINA (NAE)
LAY.	LAYER
LEA	LEACHED
LGE	LARGE
LIG	LIGNITIC
LIT	LITTLE
LL	LIQUID LIMIT
LN. (S)	LENSE (S)
LO	LOOSE
LS	LIMESTONE
LT	LIGHT
MAS	MASSIVE
MAX	MAXIMUM
MECH	MECHANICAL
MED	MEDIUM
MIC	MICACEOUS
MIN	MINIMUM
MINR	MINERALIZED (IZATION)
MIX.	MIXTURE
MOD	MODERATE (D)
MOT	MOTTLED (ING)
MST	MOIST
MTL	MATERIAL
MTX	MATRIX
N/A	NOT APPLICABLE
N/E	NOT ENCOUNTERED
N/R	NO RECOVERY
NOD.	NODULE
NUM	NUMEROUS
OB	OVERBURDEN (UNCLASSIFIED)
OBS	OBSERVED
OCC	OCCASIONAL (LY)
OOL	OOLITE. OOLITIC
OP	OPEN (ED)
OR	ORANGE

ABBREVIATIONS

ORG	ORGANIC
P.S.I.	POUNDS/SQ. IN.
P.T.	PRESSURE TEST
PART.	PARTIALLY
PCS	PIECES
PERTRO	PETROLEUM, PETROLIFEROUS
PHOS	PHOSPHATE (PHOROUS)
PI	PLASTICITY INDEX
PIT	PIT (TED) (TING)
PKT (S)	POCKET (S)
PL	PLASTIC LIMIT
PLA	PLATY
PLAS	PLASTIC
PLN	PLANE
PNK	PINK
PR	POORLY
PRED	PREDOMINATED
PRESS	PRESSURE
PROB	PROBABLE (ABILITY)
PTC	PARTICLES
PTG	PARTING
PUR	PURPLE
QTZ	QUARTZ
QTZE	QUARTZITE
R.Q.D.	ROCK QUALITY DESIGNATION
RBL	RUBBLE
RD	RED (DISH)
REC	RECOVERY
RECEM	RECEMENTED
RND	ROUND (ED)
RTS	ROOTS
S/S	SPLIT
SAP	SAPROLITE
SAT	SATURATED
SCAT.	SCATTEREDLY
SCH (S)	SCHIST (OS)
SD	SAND
SDY	SANDY
SH	SHALE
SI	SILT
SIS	SILTSTONE
SIY	SILTY
SL	SLIGHT (LY)
SLCES	SILICEOUS
SLICK.	SLICKENSIDE
SML	SMALL
SO	SOFT
SOL	SOLUTION (ED) (ING)
SPG	SPECIFIC GRAVITY
SPT	STANDARD PENETRATION TEST
SPT	STANDARD SPLITSPOON
SS	SANDSTONE
ST	STRAIN (ED) (ING)
STF	STIFF
STR	STRUCTURE
STRG	STRINGER
STYL	STYLOLITE (OLITIC)
SUR	SURFACED
T.F.R.	TOP OF FIRM ROCK
T.O.R.	TOP OF ROCK
T.S.R.	TOP OF SOUND ROCK
TEXT.	TEXTURE
THK	THICK
THN	THIN
TI	TIGHT
TN	TAN (NISH)
TR	TRACE
TRP	TRIPOLI

ABBREVIATIONS

UD	UNDISTUBED
UL	UNACCOUNTABLE LOSS
UNACC	UNACCOUNTABLE
UNWEA	UNWEATHERED
V/	VERY
VERT	VERTICAL
VGY	VUGGY
W.C.	WATER CONTENT
W.L.	WATER LEVEL
W/	WITH
W/H	WEIGHT OF HAMMER
W/R	WEIGHT OF ROD
WD	WOOD
WEA	WEATHERED
WG	WEIGH
WHT	WHITE
X-BDD	CROSS-BEDDED
XL	CRYSTAL
XLN	CYRSTALLINE
YEL	YELLOW

Proj. No.: MSE00126 Alt. Proj. No.: CH02R774 Hole No. HC-04-04

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MDO	SHEET 1 OF 2 SHEETS

1. PROJECT	HANCOCK CO. SEAWALL COASTAL	10. SIZE AND TYPE OF BIT	SPT
2. LOCATION (Coordinates or Station)	N 273306′ E 798238′	11. ELEVATION DATUM	NGVD, FEET
		COORDINATE DATUM	MSE STATE PLANE, NAD83, FEET
3. DRILLING AGENCY	MDO	12. MANUFACTORER'S DESIGNATION OF DRILL	SWAMP BUGGY ON BARGE
4. HOLE NO. (As shown on drawing title and file number)	HC-04-04	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED 14 / UNDISTURBED 0
5. NAME OF DRILLER	R. REEVES	14. TOTAL NUMBER CORE BOXES	0
6. DIRECTION OF HOLE	[X] VERTICAL [ ] INCLINED DEG. FROM VERTICAL	15. ELEVATION GROUNDWATER	EL. 0.9′
		16. DATE HOLE	STARTED 6 APR 2004 / COMPLETED 6 APR 2004
7. THICKNESS OF OVERBURDEN	–	17. ELEVATION TOP OF HOLE	EL. -1.8′
8. DEPTH DRILLED INTO ROCK	–	18. TOTAL CORE RECOVERY FOR BORING	
9. TOTAL DEPTH OF HOLE	30.0′	19. SIGNATURE OF INSPECTOR	W. SHARP / DRAFTED RLN / CHECKED RLN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
-1.8	0.0		TAN POORLY GRADED SAND (SP)		1	W.L. d.-2.7′	WOH
			TAN POORLY GRADED SAND (SP)		2	OFFSET 10′ EAST OF SEAWALL	WOH
	3.0		TAN POORLY GRADED SAND (SP)		3		WOH
-7.8	6.0		BLACK POORLY GRADED SAND (SP)		4		1
-9.3	7.5		GRAY & TAN CLAYEY SAND (SC)		5		0
	9.0		TAN SILTY SAND (SM)		6		WOH
			TAN SILTY SAND (SM)		7		WOH
	12.0		TAN SILTY SAND (SM)		8		WOH
-15.3	13.5		TAN SILTY SAND (SM) W/ GRAY SILTY CLAY (CH) LAYERS		9		WOH
-16.8	15.0		GRAY SILTY CLAY (CH)		10		6
-18.3	16.5		GRAY SILTY SAND (SM)				
-19.8	18.0		GRAY SILTY CLAY (CH)		12		10
-21.3	19.5		NO SAMPLE				
-22.8	21.0		GRAY SILTY CLAY (CH)		13		22

ENG FORM 1836 (Facsimile) PROJECT HANCOCK CO. SEAWALL COASTAL HOLE NO. HC-04-04

DRILLING LOG (Cont Sheet)	ELEVATION TOP OF HOLE EL. -1.8'	Hole No. HC-04-04
PROJECT COASTAL HANCOCK CO. SEAWALL	INSTALLATION MDO	SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
-22.8	21.0				13		
-24.3	22.5		NO SAMPLE				
-25.8	24.0		GRAY SILTY CLAY (CH) W/ SILTY SAND (SM) LAYERS		14		43
-27.3	25.5		NO SAMPLE				
-28.8	27.0		GRAY & ORANGE CLAYEY SAND (SC) & ORGANICS		15		30
-30.3	28.5		NO SAMPLE			1/2 SACK OF TIGERSOL SALTWATER DRILLING MUD.	
-31.8	30.0		TAN SILTY SAND (SM) & ORGANICS		16	BACKFILLED WITH SAND B.O.H.	55

ENG FORM 1836-A (Facsimile)	PROJECT HANCOCK CO. SEAWALL COASTAL	HOLE NO. HC-04-04

Proj. No.: MSE00126 Alt. Proj. No.: CH02R774 Hole No. HC-06-04

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MDO	SHEET 1 OF 1 SHEETS

1. PROJECT: HANCOCK CO. SEAWALL COASTAL	10. SIZE AND TYPE OF BIT: SPT
2. LOCATION (Coordinates or Station): N 274960' E 799388'	11. ELEVATION DATUM: NGVD, FEET; COORDINATE DATUM: MSE STATE PLANE, NAD83, FEET
3. DRILLING AGENCY: MDO	12. MANUFACTORER'S DESIGNATION OF DRILL: SWAMP BUGGY
4. HOLE NO. (As shown on drawing title and file number): HC-06-04	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED 10; UNDISTURBED 0
5. NAME OF DRILLER: J. STAFFERD	14. TOTAL NUMBER CORE BOXES: 0
6. DIRECTION OF HOLE: [X] VERTICAL [ ] INCLINED DEG. FROM VERTICAL	15. ELEVATION GROUNDWATER: NOT ENCOUNTERED
	16. DATE HOLE: STARTED 14 APR 2004; COMPLETED 14 APR 2004
7. THICKNESS OF OVERBURDEN: –	17. ELEVATION TOP OF HOLE: EL. 2.5'
8. DEPTH DRILLED INTO ROCK: –	18. TOTAL CORE RECOVERY FOR BORING
9. TOTAL DEPTH OF HOLE: 15.0'	19. SIGNATURE OF INSPECTOR: W. SHARP; DRAFTED RLN; CHECKED RLN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
2.5	0.0		WHITE SILTY SAND (SM)			OFFSET 8' SOUTHEAST OF SEAWALL	3
0.2	2.3		WHITE SILTY SAND (SM)				10
-0.3	2.8		ORGANIC SILT (OH)				
-2.0	4.5		BLACK SILTY SAND (SM)				1
-3.5	6.0		BLACK SILT (ML)				0
			DARK GRAY SILTY CLAY (CH)				0
-6.5	9.0		DARK GRAY SILTY CLAY (CH) W/ SAND LENSES				0
			GRAY SILTY SAND (SM)				0
	12.0		GRAY SILTY SAND (SM)				0
			GRAY SILTY SAND (SM)			GROUNDWATER NOT RECORDED	0
-12.5	15.0		GRAY SILTY SAND (SM)			BACKFILLED WITH SAND B.O.H.	4

ENG FORM 1836 (Facsimile) PROJECT HANCOCK CO. SEAWALL COASTAL HOLE NO. HC-06-04

Proj. No.: MSE00126 Alt. Proj. No.: CH02R774 Hole No. HC-07-04

DRILLING LOG	DIVISION SOUTH ATLANTIC	INSTALLATION MDO	SHEET 1 OF 1 SHEETS
1. PROJECT	HANCOCK CO. SEAWALL COASTAL	10. SIZE AND TYPE OF BIT	SPT
2. LOCATION (Coordinates or Station)	N 275515' E 800175'	11. ELEVATION DATUM / COORDINATE DATUM	NGVD, FEET / MSE STATE PLANE, NAD83, FEET
3. DRILLING AGENCY	MDO	12. MANUFACTORER'S DESIGNATION OF DRILL	SWAMP BUGGY
4. HOLE NO. (As shown on drawing title and file number)	HC-07-04	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED 10 / UNDISTURBED 0
5. NAME OF DRILLER	J. STAFFERD	14. TOTAL NUMBER CORE BOXES	0
6. DIRECTION OF HOLE	[X] VERTICAL [ ] INCLINED DEG. FROM VERTICAL	15. ELEVATION GROUNDWATER	NOT ENCOUNTERED
7. THICKNESS OF OVERBURDEN	–	16. DATE HOLE	STARTED 9 APR 2004 / COMPLETED 9 APR 2004
8. DEPTH DRILLED INTO ROCK	–	17. ELEVATION TOP OF HOLE	EL. 4.5'
9. TOTAL DEPTH OF HOLE	15.0'	18. TOTAL CORE RECOVERY FOR BORING	
		19. SIGNATURE OF INSPECTOR	W. SHARP / DRAFTED RLN / CHECKED RLN

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY OR W.C. e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	SPT BLOWS/FT
4.5	0.0						
			TAN POORLY GRADED SAND (SP) & ROOTS		1	OFFSET 10' SOUTHEAST OF SEAWALL	3
			WHITE POORLY GRADED SAND (SP)		2		7
	3.0						
0.0	4.5		WHITE POORLY GRADED SAND (SP) & SHELLS		3		8
-1.5	6.0		CONCRETE RUBBLE, NO RECOVERY		4		50+
			GRAY SILTY SAND (SM)		5		1
			GRAY SILTY SAND (SM)		6		5
	9.0						
			GRAY SILTY SAND (SM)		7		7
			GRAY SILTY SAND (SM)		8		11
	12.0						
			GRAY SILTY SAND (SM)		9	GROUNDWATER NOT RECORDED	20
-10.5	15.0		GRAY SILTY SAND (SM)		10	BACKFILLED WITH SAND B.O.H.	14

ENG FORM 1836 (Facsimile) PROJECT HANCOCK CO. SEAWALL COASTAL HOLE NO. HC-07-04

LONG BEACH CANALS

General

Flooding within the Canal 2/3 drainage basin in the City of Long Beach, Harrison County, Mississippi has been a chronic problem for many years. Complex flow patterns and steady urbanization have caused increased flooding along 28th Street and along Canal 2/3 in the City of Long Beach. This report addresses several alternatives and a proposed flooding solution. Rough order-of-magnitude cost estimates for restoring the capacity of these water courses is also presented.

Location

General location maps of the study area are shown in Figures 1, 2 and 3. The project area is within the City of Long Beach and Harrison County, Mississippi, south of Interstate 10. Canal 2/3 provides drainage for the northwest and west portions of Long Beach. The City of Long Beach is located along the Mississippi Sound between the City of Gulfport to the east, the City of Pass Christian to the southwest and Harrison County to the north and west. Canal 2 becomes Canal 3 as it flows in a southwesterly direction to Bayou Portage.

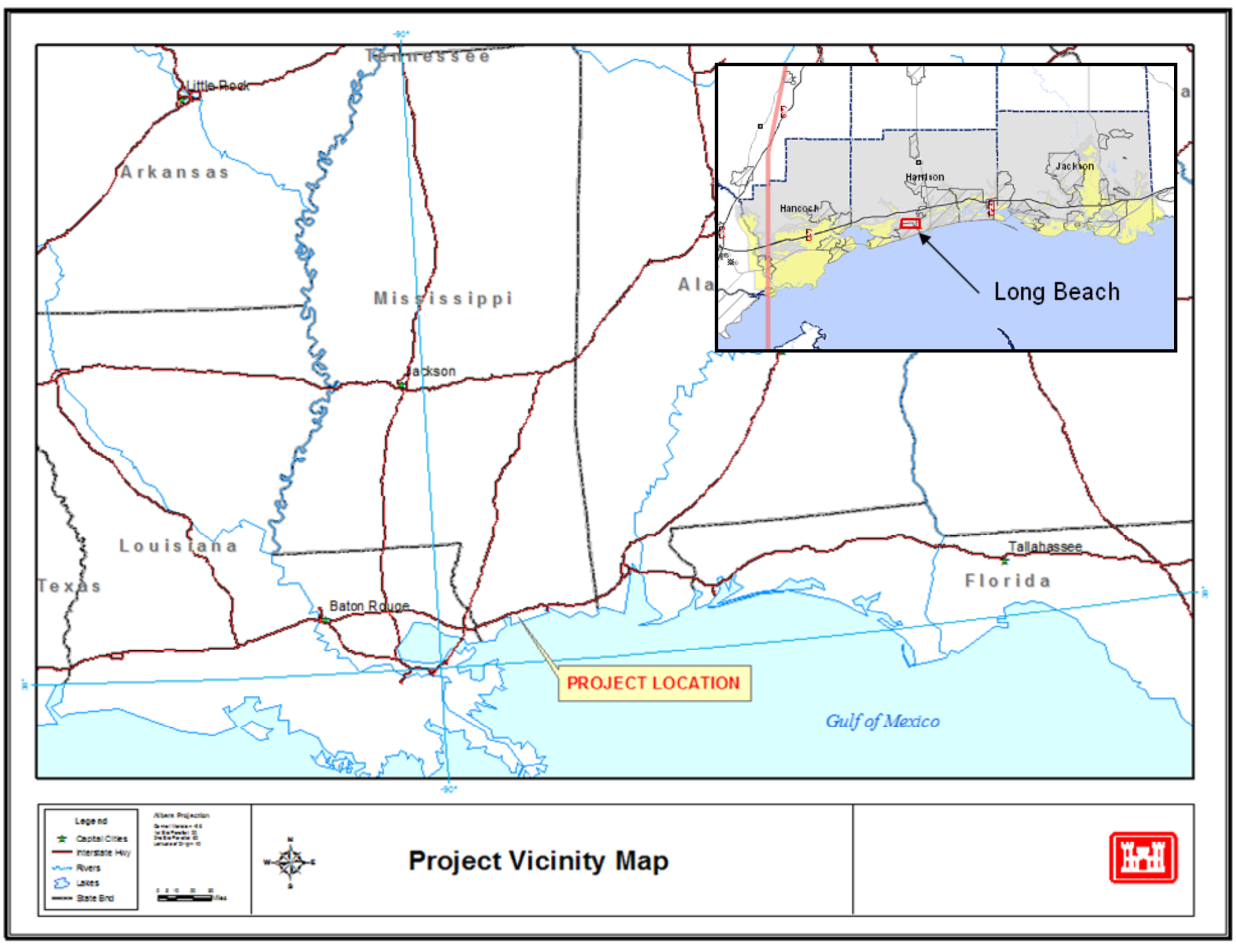


Figure 1. Vicinity Map

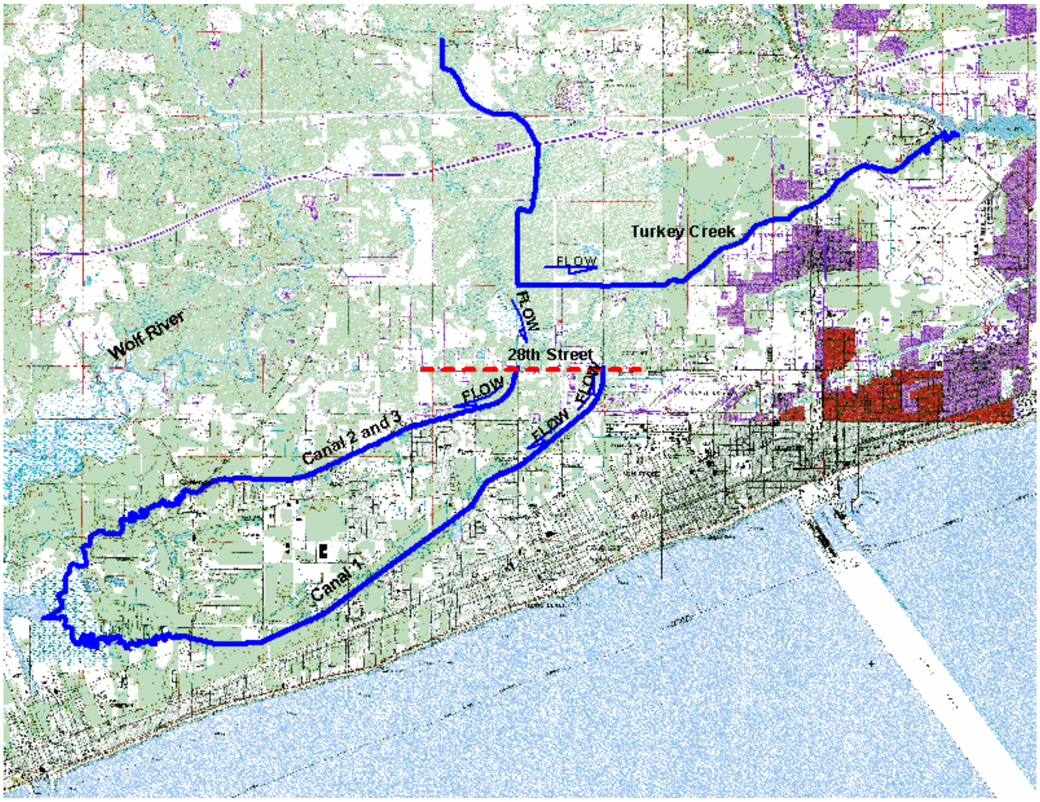


Figure 2. Project Location Map

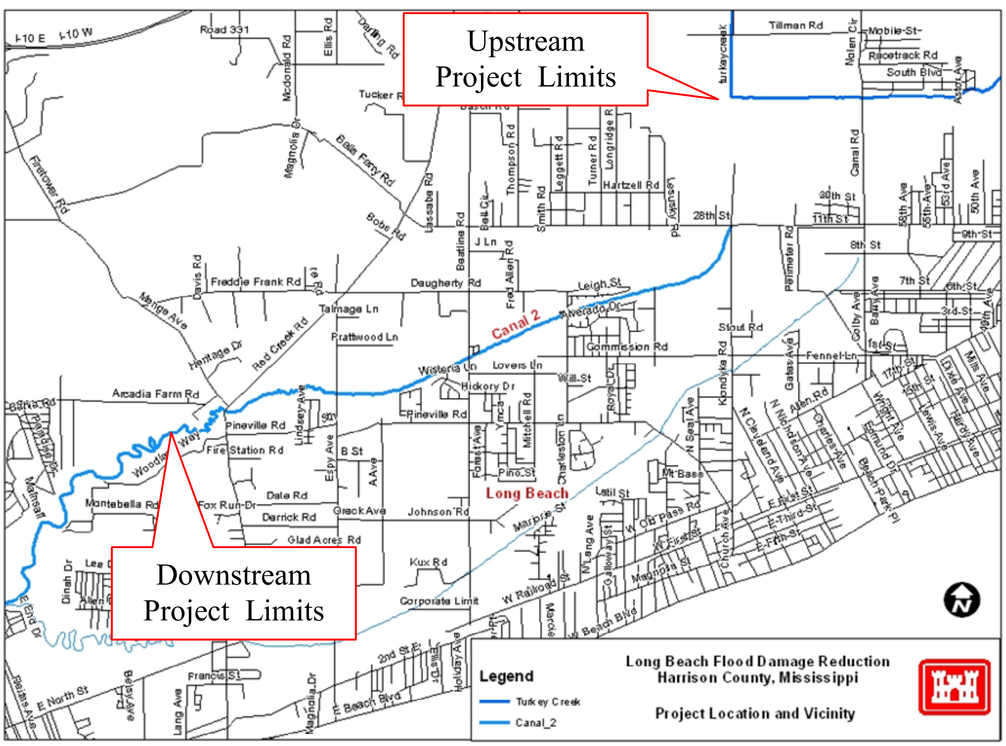


Figure 3. Project Location Map

Existing Condition

Complex flow patterns and steady urbanization have caused increased flooding within the low lying Canal 2/3 floodplain. Flooding occurs when the water in the Turkey Creek channel to the north of Canal 2 in Harrison County overflows the stream banks and flows into the floodplain. During high flow conditions most of the water from the upper basin (the upper 7.7 miles, above 28th Street) of Turkey Creek overflows the streambank to the south. The overbank flows flood 28th Street in Long Beach and flow into Canal 2 that drains to the southwest through the City of Long Beach. A smaller percent of the upper basin flow continues to the east along the main stem of Turkey Creek to Bernard Bayou. The flooding condition of the 100-yr flood event is shown in Figure 4. The water spills out of the Turkey Creek basin: (1) because of the very low elevation of the right over-bank near 28th Street, which carries most of the flood flow, and (2) because of the low elevation of 28th Street.

Once the flow from the Turkey Creek upper basin has spilled across 28th Street, the Canal 2/3 carries the flow. Due to the increased flows from the upper Turkey Creek Basin, the drainage canal does not have the flow carrying capacity to effectively hold the flows within the channel banks. The majority of the flow is carried by Canal 2 to Bayou Portage. During very high flows, the water from Canal 2 spills across the floodplain into Canal 1, which flows downstream to Johnson Bayou and Bayou Portage.

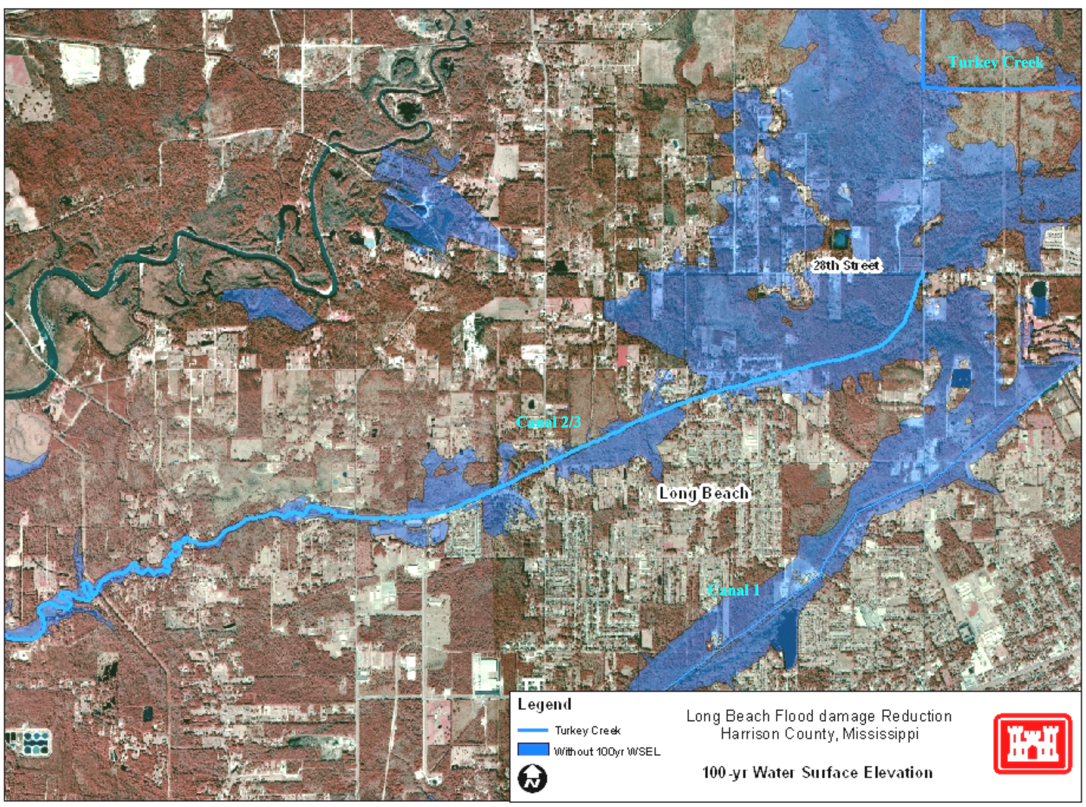


Figure 4. Existing Flood Conditions

Coastal/Hydraulics

Hurricane Katrina reached from elevation 19.04 ft NGVD and 24.07 ft NGVD along Canal 2/3 in Long Beach as shown in Figure 5. The ground elevation along Canal 2/3 is between elevation 2 and 26 ft NGVD.

Additional data is provided in a report to FEMA by URS Group, Inc., titled “Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report”, 16 January 2006, as well as in a report by FEMA titled “Draft Report, Hurricane Katrina Flood Frequency Analysis,” dated September 2005. Results are summarized below.

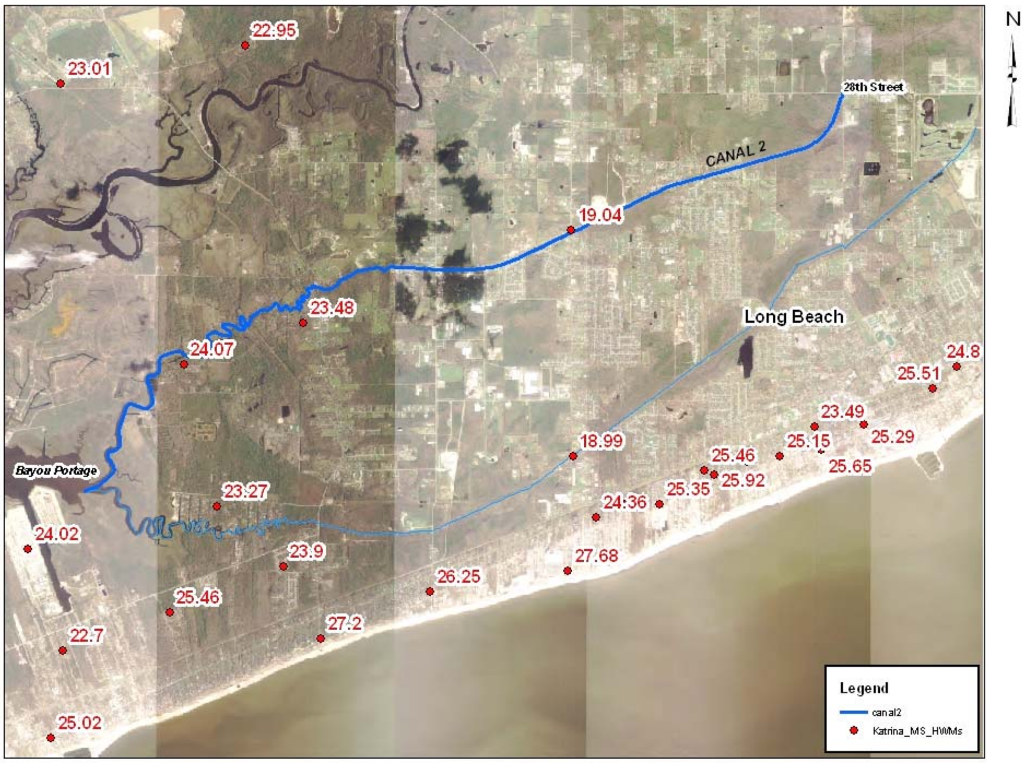


Figure 5. Hurricane Katrina High Water Marks

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b) (Tables 1 and 2 and Figure 6). The flood frequency analysis only represents conditions at and near the gage.

Table 1.
Selected Tidal Gage Stations from NOAA

Station ID	Name	Latitude	Longitude	Begin Year	End Year
8729840	Pensacola, Pensacola Bay, FL	30.40 N	87.21 W	1924	2005
8735180	Dauphin Island, Mobile Bay, AL	30.25 N	88.08 W	1967	2005
8747766	Waveland, Mississippi Sound, MS	30.28 N	89.37 W	1979	2005
8761724	Grand Isle, East Point, LA	29.26 N	89.96 W	1972	2005

Table 2.
Selected Tidal Gages from USGS/USACE

Name	Latitude	Longitude	Begin Year	End Year
Back Bay Biloxi at Biloxi, MS	30.40 N	88.84 W	1882	1998
Pascagoula River at Pascagoula, MS	30.37 N	88.56 W	1940	1998

Figure 4. Tidal Gage Locations in the Area Impacted by Hurricane Katrina

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.

- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.
- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.
- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet.
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet.

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 3 As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 3.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations Location Katrina Elevation

Location	Katrina Elevation (Ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
DauphinIsland, AL	5.81	20
Pensacola, FL	6.07	50

A stage-frequency curve developed by the Corps of Engineers for the Gulfport gage is shown in Figure 5. The gage shows stage 24 to have a return frequency of 100 years compared to the FEMA table which shows a return interval of >10,000 years.

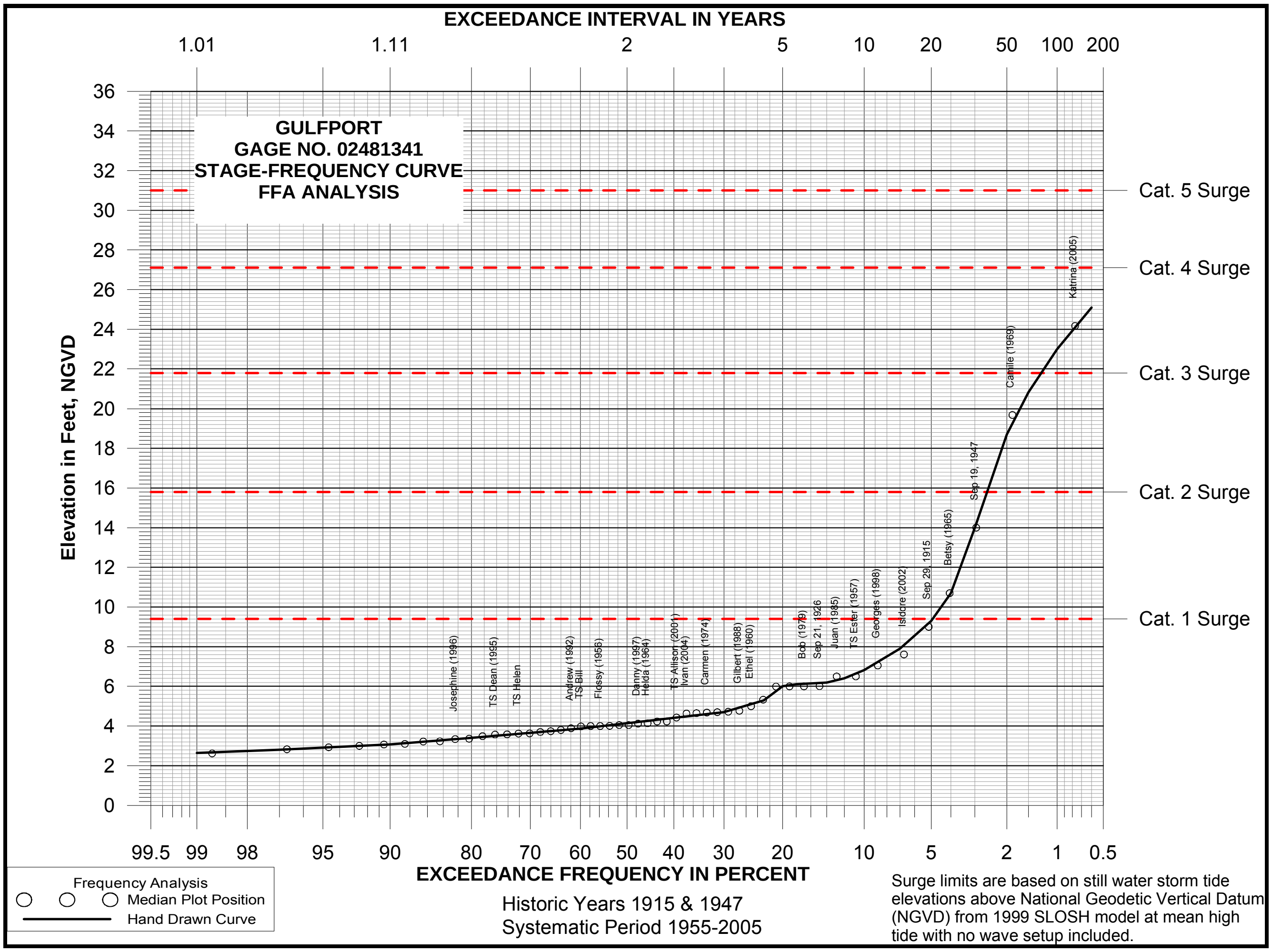


Figure 5. Stage-Frequency

Hydrologic and Hydraulic Analyses

Hydrologic and hydraulic analyses for the alternatives are documented in the Turkey Creek Section 205 Flood Control Feasibility Study, Appendix B, Engineering Analyses report on file in the Mobile District Office. The following information summarizes the analyses for the alternatives considered for the Long Beach canals flood reduction project. Figures 6 and 7 show the watershed sub-basins.

The U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) was selected for modeling the Turkey Creek watershed, which includes the drainage basins for Canals 1 and 2/3. This model includes procedures for converting rainfall to streamflow, as well as capabilities to generate, combine and route hydrographs through various channels, storage facilities, drainage structures and conveyances. The HEC-HMS model allows for the magnitude, relative frequency and duration of certain stream flows to be characterized for existing and future land cover/land use conditions. Flows for alternatives were initiated with Future Condition run-off from the HMS model. These flows were used by the HEC-RAS unsteady flow hydraulic model, and the flow routing was accomplished internally by the HEC-RAS model.

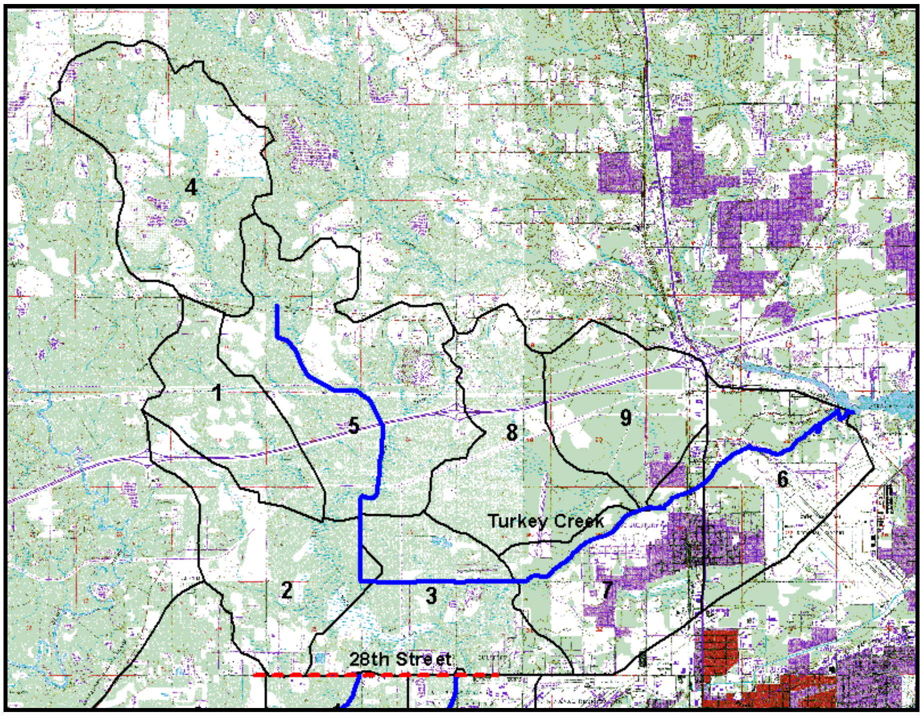


Figure 6. Turkey Creek Sub-Basins

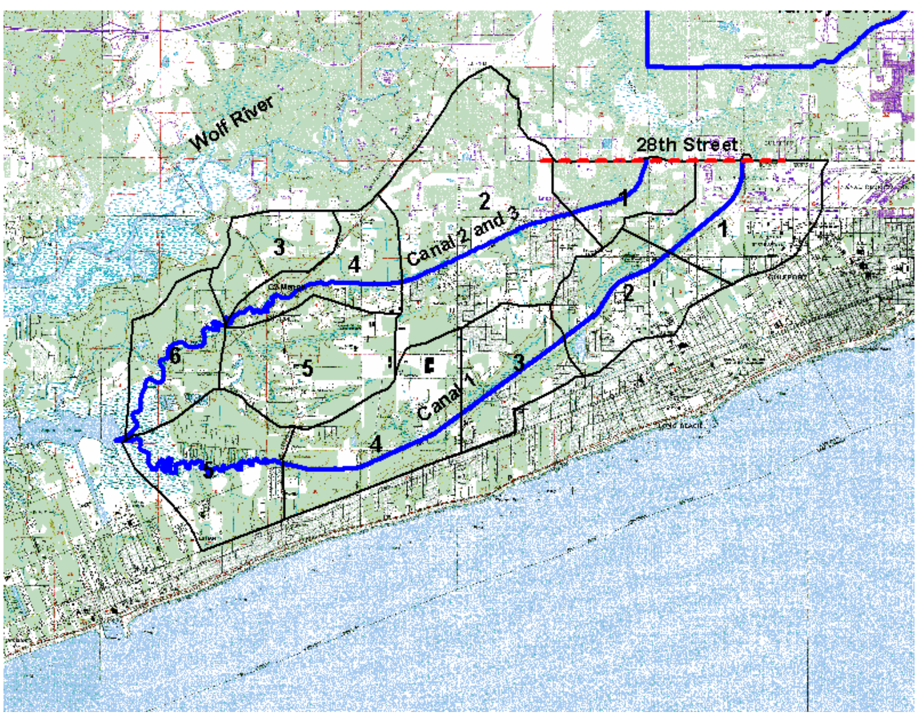


Figure 7. Canals 1 and 2 Sub-Basins

The U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) was selected to simulate the existing and future condition open channel flow and to determine the water surface profiles along Canal 2/3. Once the floodplain cross-sections were cut from the composite topographic surface, they were used to construct an unsteady flow model in HEC-RAS. The unsteady flow model has the capacity to immediately account for variations in flow occurrence by off-channel and in-channel storage areas or losses over 28th Street. The model was constructed to account for these losses and also for a connection between Canals 1 and 2. The loss of water during high floods at 28th Street was modeled by using the ground elevations, mostly along the center of 28th Street, as two weirs; one at Canal Road at Canal 1, near River Station (RS) 29000, and the other at Canal 2, near RS 31000. Coefficients at the weir, in addition to lag times in the HEC-HMS model, were adjusted to match anecdotal information regarding the frequency and depth of flooding along the street. Existing and future without project condition water surface profiles are shown in Figures 8 and 9. The channel modification alternative C2-1 was evaluated generally by modifying the geometry in the HEC-RAS model and re-computing hydraulic parameters in the unsteady model. Future land use coefficients were used in HEC-HMS models to develop the flows used in the alternative evaluations. Although several alternatives were evaluated to reduce flooding, some of them were not modeled because preliminary evaluation revealed they would not be acceptable for economic, environmental, or other reasons.

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. The general surface and subsurface conditions for this work are typical of the surrounding area. The work areas are generally flat with slight slopes to the existing creek. The near surface soils consist of poorly graded sands and silty sands to depths within any influence of this project. Groundwater can be expected within 10 feet of the surface.

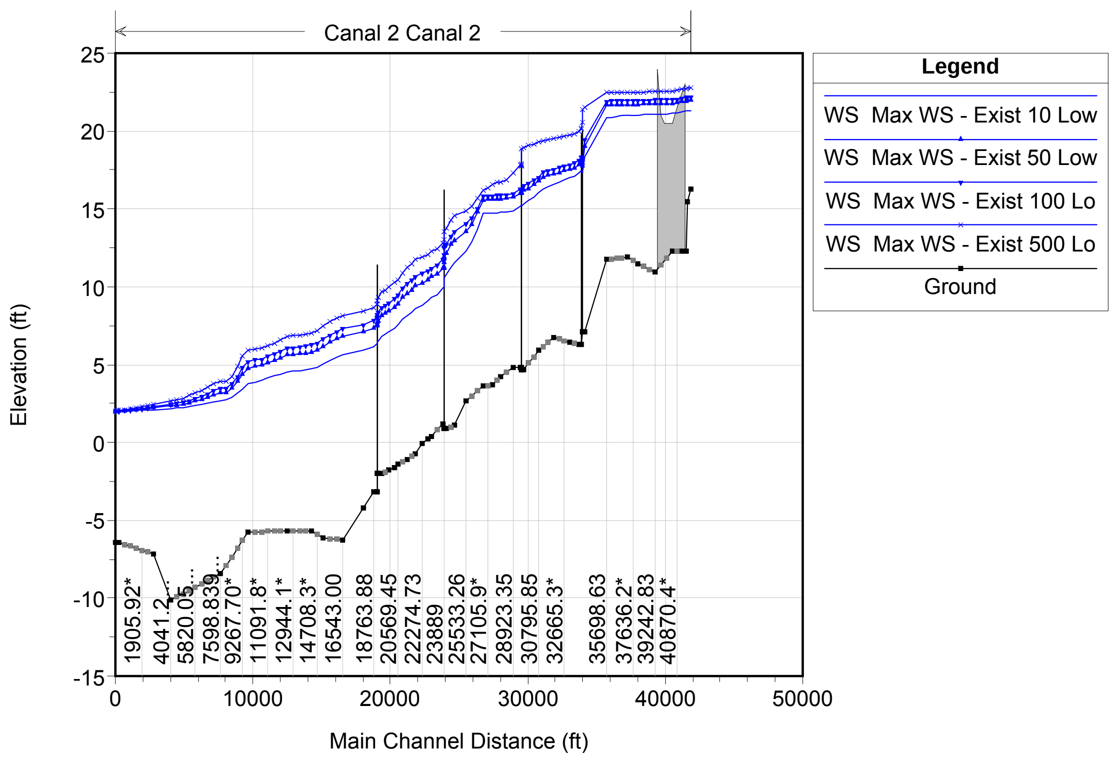


Figure 8. Canal 2 Existing Conditions Water Surface Profiles

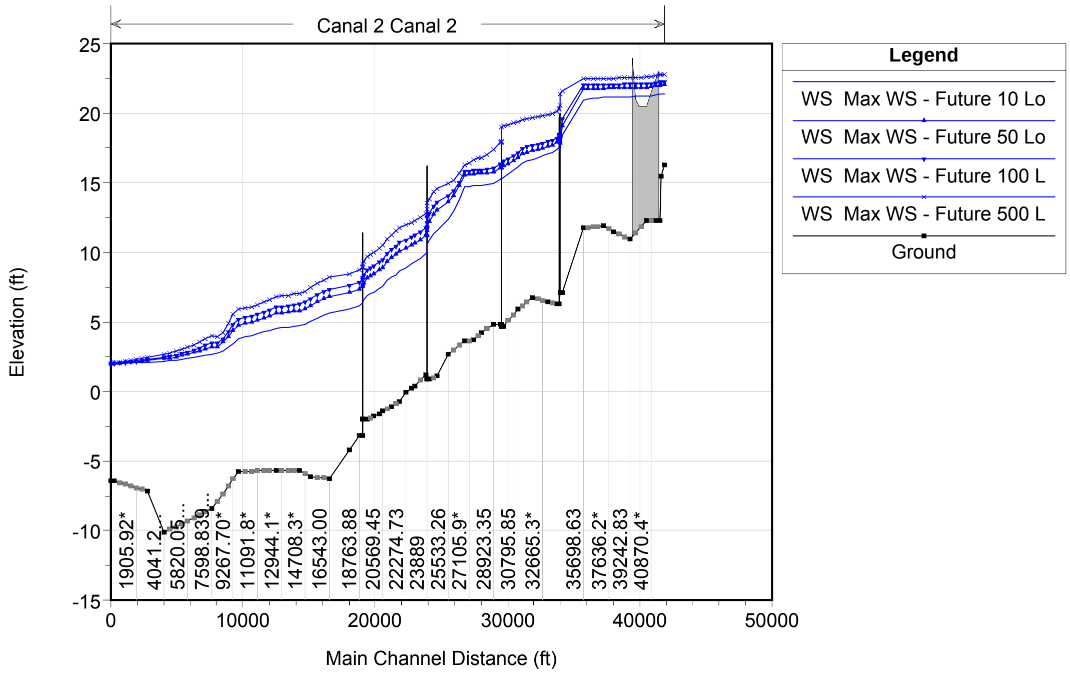


Figure 9. Canal 2 Future Without Project Conditions Water Surface Profiles

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

- ***Alternative 1*— Construct Culvert System from 28th Street to Mississippi Sound.** This alternative consists of increasing the size of the culverts under 28th St to carry the flow presently going over the road. A schematic of this plan showing the location of the culverts is shown in Figure 10. Based on the unsteady flow HEC-RAS model, the future hydrology without-project 100-yr flow is approximately 4300 cfs at the culverts at an elevation 23.8 ft NGVD.

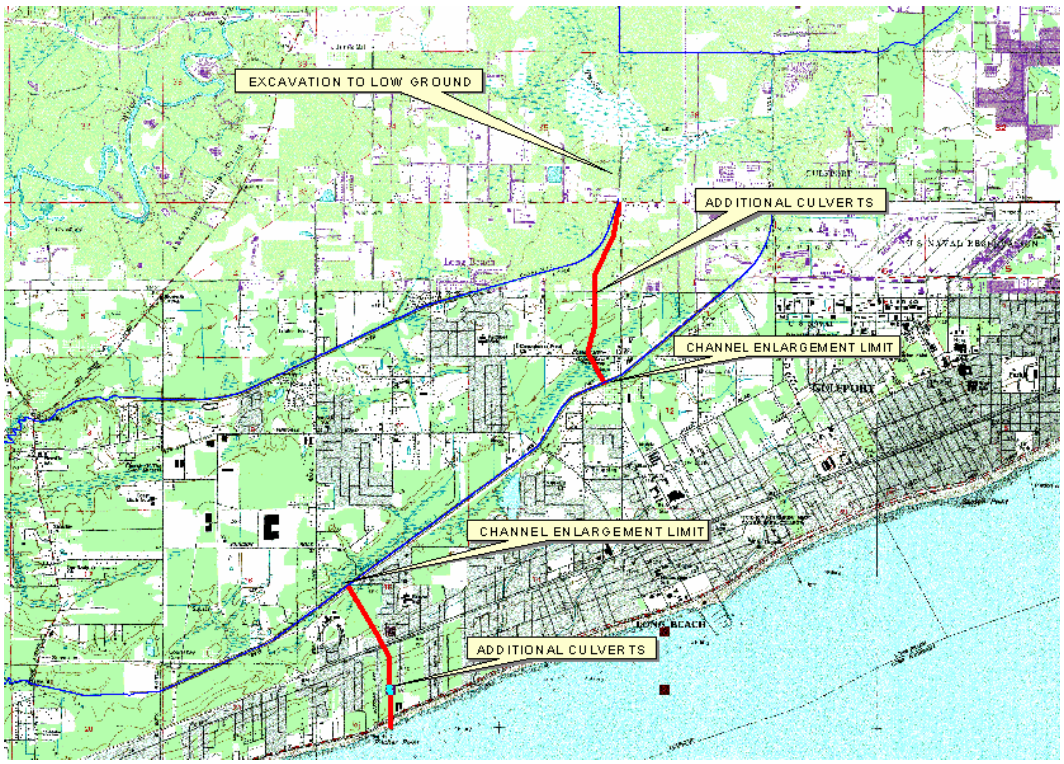


Figure 10. Canal 1 and Culverts Alternative Alignment

The existing elevation of 28th Street is approx el 21.0 ft NGVD. The existing weir flow over 28th Street is approx 3400 cfs, the remaining 870 cfs goes down Turkey Creek. To reduce the 100-yr water surface at 28th Street to elevation 21 to prevent overtopping the road, and to reduce flood damages, provision would have to be made to get the water to the culvert at that elevation. A channel large enough to carry the flow at this elevation would be required upstream of 28th Street. This could be done by constructing a berm across the floodplain perpendicular to the flow adjacent to a large channel (120 ft wide at elevation 13.5 ft NGVD—see depth computation in Table 4) and diverting all or part of the flow in the basin into the culverts. Basically, this could divert the upper Turkey Creek basin into the culverts(4300 cfs). Canal 1 would have to be enlarged to 160-ft bottom width to carry the flow, 3400 cfs plus the existing Canal 1 flow. Eight 10-ft span x 8-ft rise culverts would be required. The length required to get to Canal 1 is approximately 6400 ft. After traveling down the enlarged Canal 1, culverts would also be required to carry this flow from Canal 1 out to the Mississippi Sound for a distance of approximately 5200 ft.

Table 4.

Normal Depth in Trapezoidal Channel		
(Newton-Raphson Convergence)		
Slope=	0.001	
Q=	4300	
N=	0.03	
Bot W =	120	
Z=	3	
	Normal Depth =	**6.33617817**
	Velocity at Nml Dep =	**4.88202335**

- ***Alternative 2*—Raising 28th Street.** Another option would be to raise 28th Street to elevation 24 ft NGVD. The culverts described in Alternative 1 could be used to prevent damages in the Canals and downstream of 28th Street if 28th Street was raised to elevation 24 ft NGVD and only the existing weir flow (3300 cfs) is put into the culverts. The areas flooded by the raised street upstream of 28th Street would have to be bought out and some upstream channelization and berms would be required to allow 3300 cfs to flow freely to the culverts. All the flow downstream would be in the culverts or in Canal 1. The culverts would exit into Canal 1, near RS 40752.

Canal 1 would have to be enlarged to 160-ft bottom width to carry the flow 3300 plus the existing Canal 1 flow. Eight 10-ft span x 8-ft rise ft culverts would be required. The length required to get to Canal 1 is approximately 6400 ft. After traveling down the enlarged Canal 1, culverts would also be required to carry this flow from Canal 1 out to the Mississippi Sound for a distance of approximately 5200 ft.

- ***Alternative 3*— Culvert Enlargement at 28th Street and modification of Canals 2&3.** Alternative 3 consists of increasing the 28th Street bridge at Canal 2 and Klondike Road and modifying the geometry of Canal 2/3. Figure 11 shows the limits and location of the Canal 2 modification alternative. This alternative would alleviate flooding of 28th street and provide reductions in flood elevations along Canal 2 from Menge Avenue to 28th Street and along the upper portion of Canal 1.

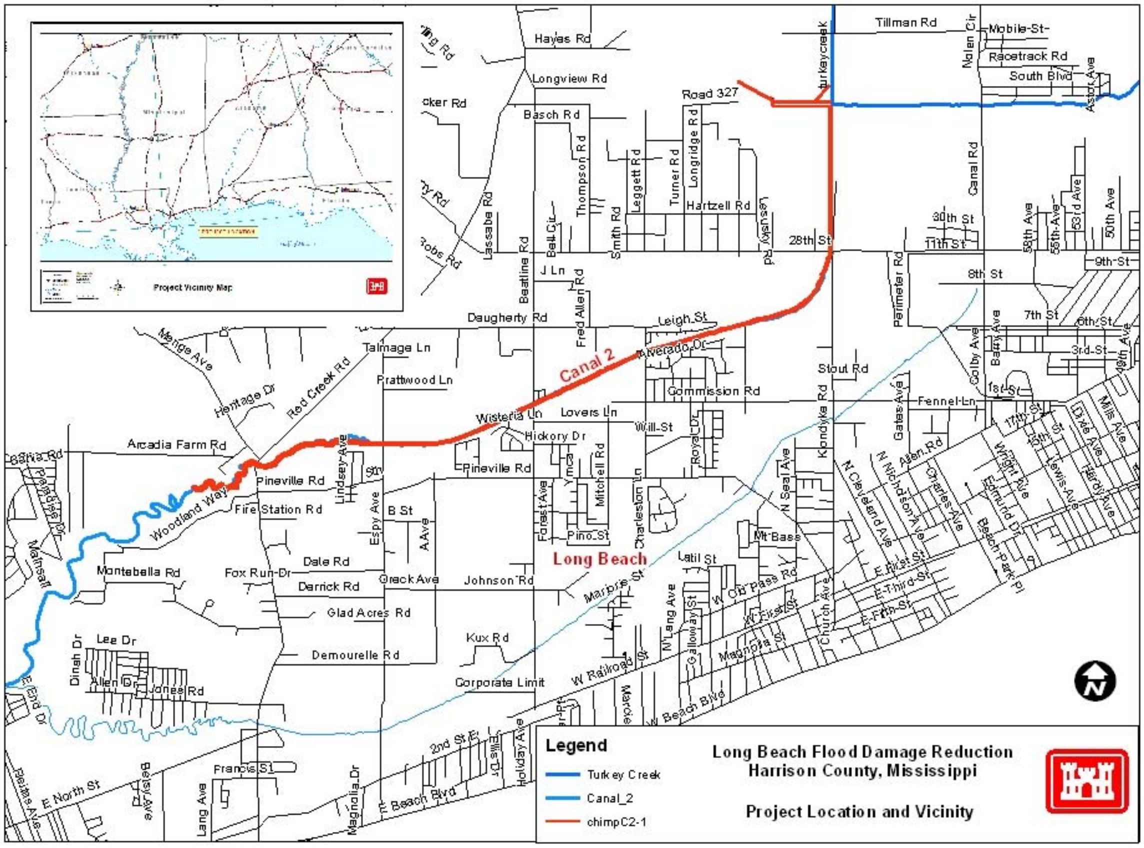


Figure 11. Alternative 3 (C2-1) Canal 2 Channel Modification) Alignment

The Canal 2 channel modification would include a 100-ft bottom width channel from Canal 2 station 14280 to 23414, 60-ft bottom width channel from 23814 to a constructed diversion channel near Turkey Creek. The channel width would transition to near vertical gabion walls at the remaining bridge crossings. To alleviate the flooding of 28th Street, an earthen berm and diversion channel at the upper limits of Canal 2 at Turkey Creek would be constructed to divert Turkey Creek overbank flows into the modified Canal 2 and toward Bayou Portage. Figure 12 shows the reduction in the 100-yr flood due to the Canal 2 channel modification plan. This plan would provide a significant reduction in the water surface elevation by reducing the depth of the water up to approximately 3.3 feet along Canal 2 upstream of Menge Avenue to 28th Street. The profiles indicate that there would be no changes to the water surface elevations along Turkey Creek, which indicates that only existing overbank flows from Turkey Creek across the floodplain and 28th Street would be directed and conveyed by the modified Canal 2. A reduction in the water surface elevation by up to 1.3 feet along the upper end of Canal 1 would occur due to the modified Canal 2 being able to convey the water that overflows the Canal 2 banks and into the Canal 1 as occurs under the existing flood conditions. Along Canal 2, the water surface elevations would decrease significantly at the upper end of the modified Canal 2 where existing flooding occurs. However, the existing model results indicate that the water surface could rise in the downstream Bayou Portage. Further analysis during the engineering design phase would incorporate mitigation for any downstream affects.

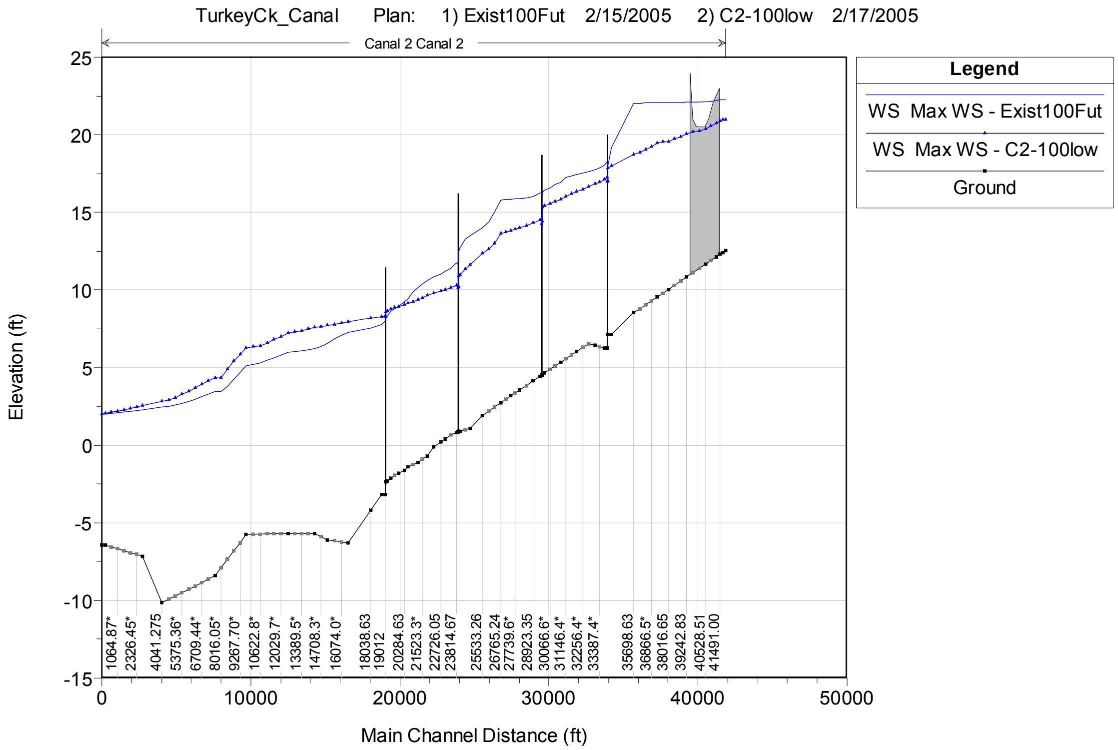


Figure 12. Alternative 3 (c2-1) Canal 2 100-yr Water Surface Profiles

The bridges crossing Canal No. 2 to the south and west of the main channel of Turkey Creek are at 28th Street near the upstream end of the canal, then proceeding downstream, Daugherty Road, Beat Line Road, Espy Avenue, and Menge Avenue. The only crossing requiring modification to the existing structure is at 28th Street. The existing 28- foot wide by 30-foot long bridge would be removed and replaced in its entirety. The replacement bridge would be 28 feet wide by 120 feet long, and would consist of 4 precast-prestressed concrete roadway spans with precast New Jersey Curb type side barrier walls, all supported on precast concrete bent caps and abutments and 14" square precast-prestressed concrete piling. The abutments would be fitted with appropriate precast concrete wing walls also supported on 14" square precast-prestressed concrete piling. The tentative layout for this bridge was based on a department of transportation standard for bridges of this size and capacity. For all the rest of the bridges which cross the Canal, the stream banks would be altered using gabion sidewalls to steepen the banks to provide a nominally larger bridge opening while using the existing bridges.

- ***Alternative 4*—Levee at 28th Street.** This alternative consists of a levee just north of 28th Street crossing the upstream ends of Canal 1 and Canal 2. The levee would be at approximately elevation 25 ft NGVD. Because of the flow patterns in Canal 1, a pump station would be required on the inside of the levee at the Canal Road area. A culvert through the levee would also be required at this site. The culvert would have a flap gate on the Turkey Creek side to prevent high water in the creek from coming through the levee. Figure 13 shows the location and alignment of the 28th Street levee alternative.

This alternative was considered only briefly because it would tend to increase the flooding on the lower main stem of Turkey Creek by preventing the existing outflow of water to Canal 1 and Canal 2.

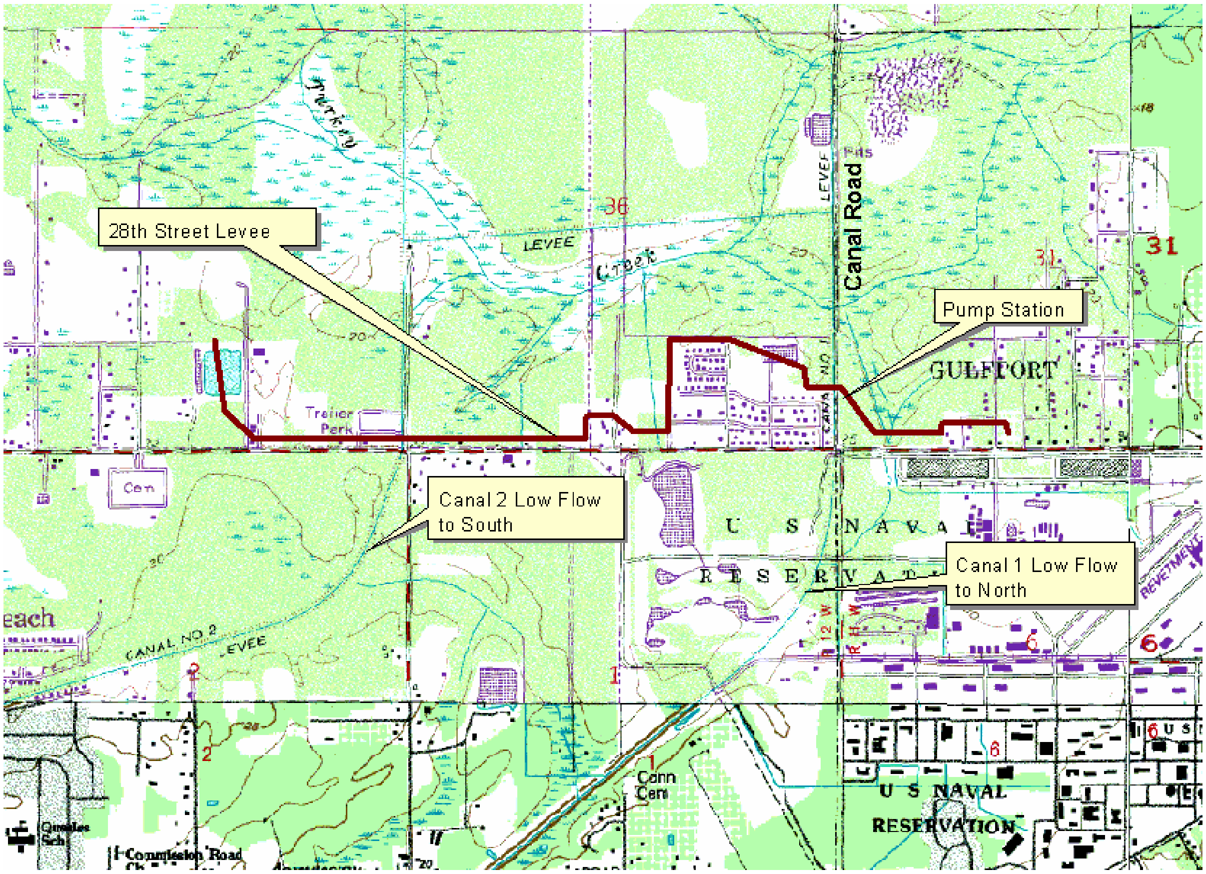


Figure 13. Levee at 28th Street Alternative Alignment

Construction Procedures and Water Control Plan—Alternative 3

Construction would be done by using mechanical excavation equipment and dump trucks. Material could be hauled to a disposal area. No water control would be required.

Project Security—Alternative 3

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance—Alternative 3

Operation and maintenance (O&M0 activities for this project will be minimal and will include only periodic visual inspection, bank clearing, and turf management. Shoaling is expected to be minimal except in the event of a rare hurricane event. Maintenance costs are included in this report.

Cost Estimates

Rough Order of Magnitude (ROM) Cost Estimates were developed for the recommended alternative. The ROM cost estimates for the alternatives 1 and 3 are included as Table 5 and 6. The O&M cost estimates are provided in tables 7 and 8.

Table 5.
Alternative 1 – Initial Cost Estimate

Summary: Alternates — Preliminary "ROM" Estimate

PROJECT: Coastal Ms- Long Beach	ITEM NO. 1		DATE 27-Jul-06
LOCATION: Gulfport, MS	SHEET NO. 1		OF 1
	PREPARED: parmer		CHECKED:
WORK ITEM:	BASIS of ESTIMATE:		Info furnished per Randall B. Harvey
	FILE NAME: longbeach6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Alternative 1 28th St Culvert				
Mobilization, Preparatory Work, Demobilization	1	ls	allow	$100,000
Concrete Box Culvert	96,000	cy	650	62,400,000
Earthwork	485,000	cy	10	4,850,000
Clearing, Grubbing & Snagging	100	ac	3500	350,000
Channel Excavation	600,000	cy	7	4,200,000
Dewatering	1	ls	50000	50,000
Diversion Structure				
Roller Compacted Concrete	680	cy	400	272,000
Riprap 24"	12,973	cy	100	1,297,300
Riprap, Grouted 24"	893	cy	200	178,600
Bedding Material	3,467	cy	100	346,700
Filter Cloth	20,800	sy	6	124,800
Bridge Modifications	1	job	allow	300,000
Grassing, seed & mulch	100	acr	4000	400,000
Restoration of Site (staging & access areas)	5	acr	2000	10,000
Environmental Protection (silt fencing, haybales)	5,000	lf	6.00	30,000
Misc. Items (signage, etc.)	1	ls	allow	10,000
09 Account, Channels and Canals	Current Contract Cost, Oct 06			$74,919,400
CONTINGENCY		@	20.0%	14,983,880
				$89,903,280
01 Account, Lands & Damage	PCA		LS	
	Relocations		LS	
	Real Estate		LS	975,000
xx Account, Environmental Mitigation			LS	0
				$90,878,280
30 Account, Plan, Engr.& Design		@	8.0%	7,270,262
				$98,148,542
31 Account, Constr. Management		@	6.0%	5,888,913
				$104,037,455
				rounded
TOTAL PROJECT COST, FY-07				**104,040,000**

Asumptions:
Price Level, Oct 06.
Estimate excludes: Recreation Cost, Lands and Damages, Relocations, Mitigation
Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Table 6.
Alternative 3 – Initial Cost Estimate

Summary: Alternates — Preliminary "ROM" Estimate

PROJECT:	Coastal Ms- Long Beach	ITEM NO.	1	DATE	27-Jul-06
LOCATION:	Gulfport, MS	SHEET NO.	1	OF	1
		PREPARED:	parmer	CHECKED:	
WORK ITEM:		BASIS of ESTIMATE:		Info furnished per Randall B. Harvey	
		FILE NAME:	bigbeach6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Alternate C2-1 Canal 2 Modification				
Mobilization, Preparatory Work, Demobilization	1	ls	allow	$100,000
Clearing, Grubbing, & Snagging	40	acr	3500	140,000
Channel Excavation, mechanical (5 mi. haul)	263,200	cy	7.00	1,842,400
Dewatering	1	job	allow	50,000
Berm Fill, off-site material	22,640	cy	8.00	181,120
Gabions	49,610	cy	225.00	11,162,250
Diversion Structure				
Roller Compacted Concrete	680	cy	400	272,000
Riprap, 24"	12,973	cy	100	1,297,300
Riprap, grouted, 24"	893	cy	200	178,600
Bedding Material	3,467	cy	100	346,700
Filter Cloth	20,800	sy	6.00	124,800
Culverts, RCP, 24"	375	lf	45.00	16,875
Headwall, Wingwall, Apron Concrete	100	cy	700.00	70,000
Bridge Modifications	1	job	allow	300,000
Grassing, seed & mulch	40	acr	4000	160,000
Restoration of Site (staging & access areas)	5	acr	2000	10,000
Environmental Protection (silt fencing, hay bales)	3,000	lf	6.00	18,000
Misc. Items (signage, etc.)	1	ls	allow	10,000
09 Account, Channels and Canals		**Current Contract Cost, Oct 06**		$16,280,045
CONTINGENCY		@	**20.0%**	3,256,009
				$19,536,054
01 Account, Lands & Damage		Real Estate	**LS**	975,000
				$20,511,054
30 Account, Plan, Engr.& Design		@	**8.0%**	1,640,884
				$22,151,938
31 Account, Constr. Management		@	**6.0%**	1,329,116
				$23,481,055
				rounded
		TOTAL PROJECT COST, FY-07		**$23,480,000**

Asumptions:

Price Level, Oct 06.

Estimate excludes: Recreation Cost, Lands and Damages, Relocations, Mitigation

Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Table 7.
Alternative 1 – O&M Cost Estimate

Summary: Alternates — Preliminary "ROM" Estimate

PROJECT:	Coastal Ms- Long Beach	ITEM NO.	2	DATE 31-May-05
LOCATION:	Gulfport, MS	SHEET NO.	3	OF 3
		PREPARED:	parmer	CHECKED:
WORK ITEM:		BASIS of ESTIMATE:		info furnished per Randall B. Harvey
		FILE NAME:	o-m longbeach5-31.xls	

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
28th St Culvert				
Mobilization, Preparatory Work, Demobilization	1	ls	allow	$20,000
Clearing & Snagging	11,600	lf	10	116,000
Channel Excavation	32,250	cy	7	225,750
Misc grass maint	14	acr	2000	27,000
09 Account, Channels and Canals	**Current Contract Cost, Oct 06**			$388,750
CONTINGENCY		@	**20.0%**	77,750
				$466,500
01 Account, Lands & Damage	PCA		**LS**	
	Relocations		**LS**	
	Real Estate		**LS**	0
xx Account, Environmental Mitigation			**LS**	0
				$466,500
30 Account, Plan, Engr.& Design		@	**8.0%**	37,320
				$503,820
31 Account, Constr. Management		@	**6.0%**	30,229
				$534,049
ESCALATION, FY-07		@	**1.0%**	5,340
				$539,390
				rounded
TOTAL PROJECT COST, FY-07				**540,000**

Asumptions:

Price Level, Oct 06.

Estimate excludes: Recreation Cost, Lands and Damages, Relocations, Mitigation

Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Table 8.
Alternative 3 – O&M Cost Estimate

Summary: Alternates	Preliminary "ROM" Estimate

PROJECT:	Coastal Ms- Long Beach	ITEM NO.	1	DATE	31-May-05
LOCATION:	Gulfport, MS	SHEET NO.	2	OF	3
		PREPARED:	parmer	CHECKED:	
WORK ITEM:		BASIS of ESTIMATE:		info furnished per Randall B. Harvey	
		FILE NAME:	o-m longbeach5-31.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Alternate C2-1 Canal 2 Modification					
Mobilization, Preparatory Work, Demobilization		1	ls	allow	$20,000
Clearing & Snagging		38,500	lf	10	385,000
Channel Excavation, mechanical (5 mi. haul)		57,100	cy	7.00	399,700
Gabions		2,481	cy	225.00	558,225
Misc Grass maint		45	acr	2000	90,000
09 Account, Channels and Canals		**Current Contract Cost, Oct 06**			$1,452,925
	CONTINGENCY		@	**20.0%**	290,585
					$1,743,510
	01 Account, Lands & Damage	PCA		**LS**	
		Relocations		**LS**	
		Real Estate		**LS**	0
	xx Account, Environmental Mitigation			**LS**	0
					$1,743,510
	30 Account, Plan, Engr.& Design		@	**8.0%**	139,481
					$1,882,991
	31 Account, Constr. Management		@	**6.0%**	112,979
					$1,995,970
	ESCALATION, FY-07		@	**1.0%**	19,960
					$2,015,930
					rounded
		TOTAL PROJECT COST, FY-07			**$2,020,000**

Asumptions:

Price Level, Oct 06.
Estimate excludes: Recreation Cost, Lands and Damages, Relocations, Mitigation
Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Schedule for Design and Construction

A typical schedule for preparation of P&S through construction is shown in Table 6 below.

Table 6.
Typical Schedule for P&S

Draft P&S	4 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	6 months after NTP

References

Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report, FEMA (URS Group, Inc.), 16 January 2006.

Draft Report, Hurricane Katrina Flood Frequency Analysis, FEMA, September 2005.

NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b).

HARRISON COUNTY BEACHES

General

The purpose of this document is to provide engineering information for planning and design analysis as related to protection of the rehabilitated beaches that were damaged by Hurricane Katrina storm surge in Harrison County, Mississippi.

Location

The beaches are located along the entire coast of Harrison County, the center coastal county in Mississippi. It is located on Mississippi Sound, running from Biloxi to the east to beyond Long Beach on the west about midway between Mobile, Alabama and New Orleans, Louisiana. The beaches are positioned south of Highway 90 as shown below.

Existing Conditions

Prior to Hurricane Katrina, the sandy beach at the project site extended from Hwy 90 approximately 230 ft to elevation 3.5 ft NGVD, and then another 40 ft to the water at elevation 0.0 ft NGVD. Storm water culverts passed beneath Hwy90 draining parts of Biloxi, Long Beach and Pass Christian. See Figures 1, 2 and 3 for project location and limits.

The project incurred damage from wind driven waves, debris scour, storm surge and ebb flow after the hurricane. The nature of the damage is scour or erosion of the beach, as well as clogging and destruction of storm drain culverts. The rehabilitation of the beach project under authority of PL 84-99 consists of re-nourishment of the beach and repair/replacement of storm drain culverts to their authorized limits. The area to be repaired extends the full length of the project limits, a distance of approximately 24 miles. This project proposes to build dunes atop the reconditioned beach and an alternative would place fencing and plants atop the dunes. Figure 4 shows pre-Katrina conditions at Gulfport Harbor and adjacent typical beach, in the midst of the project site. Figure 5 is the post-Katrina condition at the same site. Figure 6 shows the before-storm conditions at Pass Christian Harbor, while Figure 7 shows the after-storm condition. Figure 8 shows the pre-Katrina beach in Biloxi. Figure 9 shows post-Katrina damage along the coast in Biloxi.

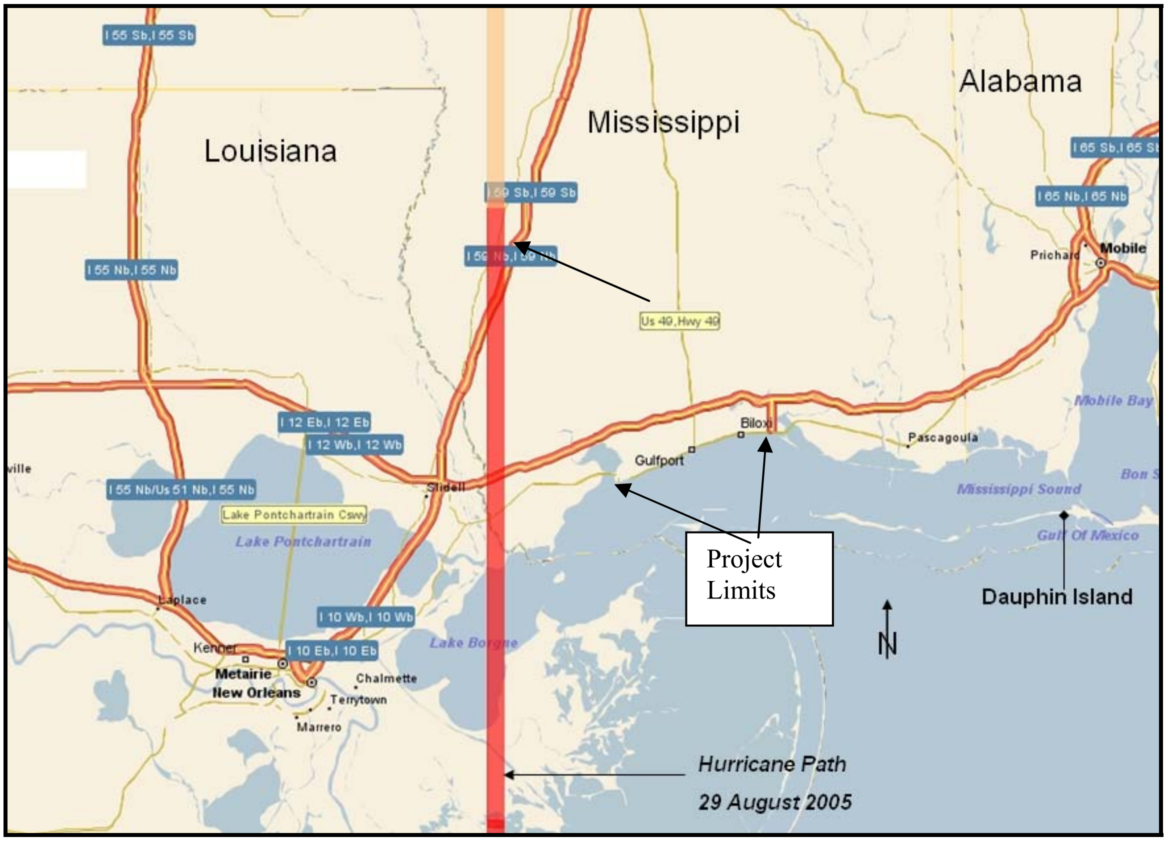


Figure 1. Project Location Map

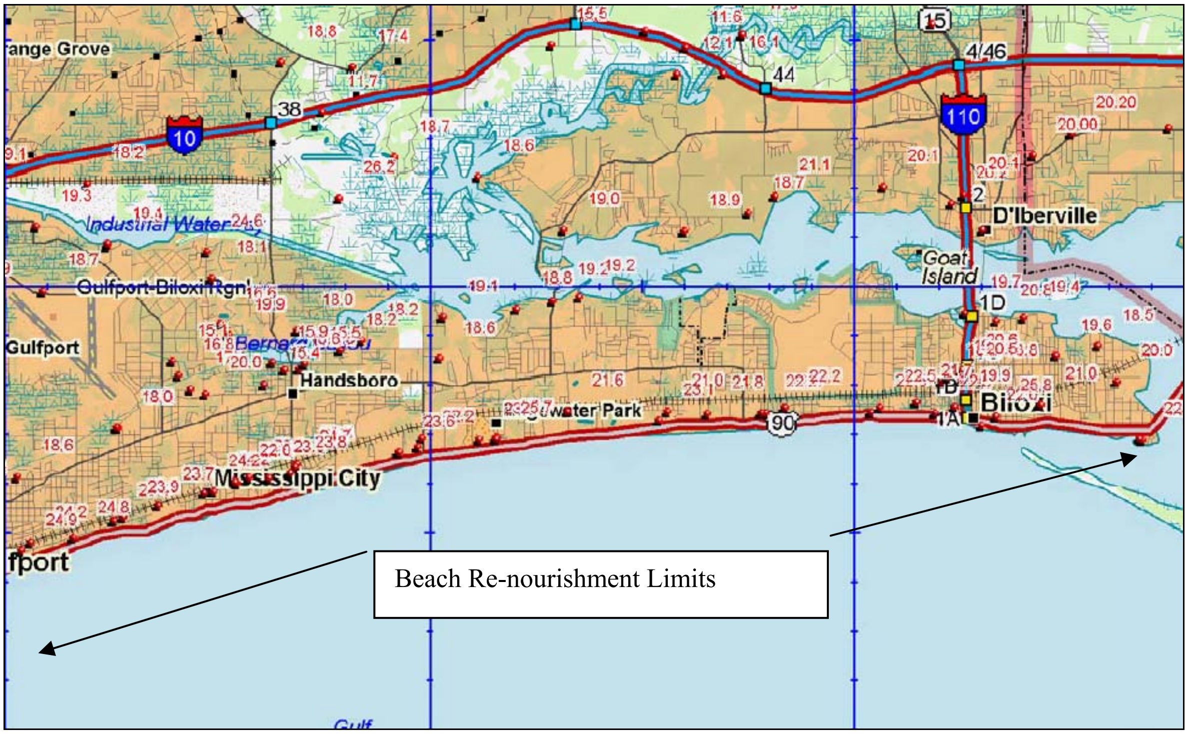


Figure 2. Project Limits Map

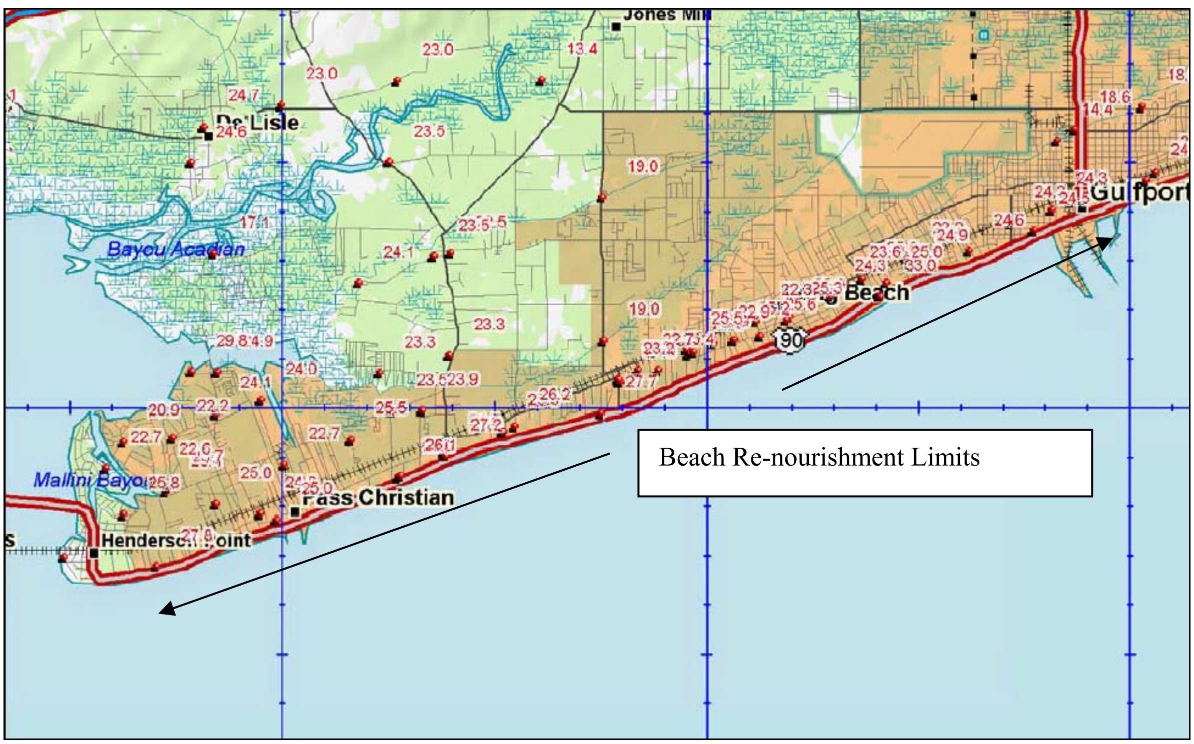


Figure 3. Project Limits Map

Figure 4. Gulfport Harbor Pre-Katrina Condition

Figure 5. Gulfport Harbor Post-Katrina Condition

Figure 6. Pass Christian Harbor Pre-Katrina Condition

Figure 7. Pass Christian Harbor Post-Katrina Condition

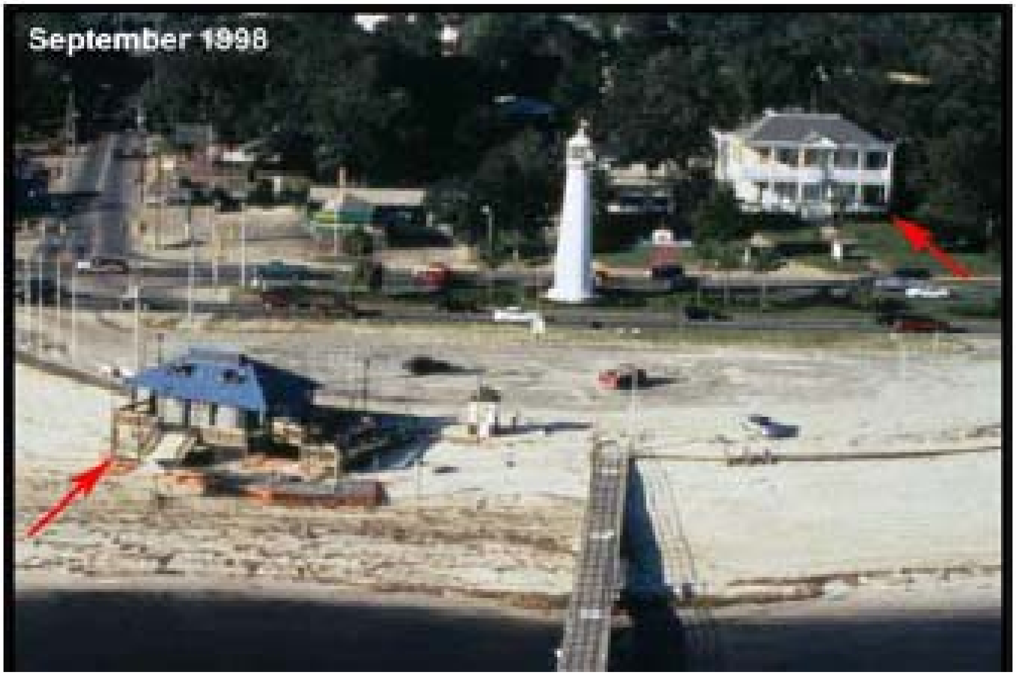


Figure 8. Biloxi Along Coast Pre-Katrina Condition

Figure 9. Biloxi Along Coast Post-Katrina Condition

Coastal and Hydraulic Data

Mississippi Sound is a shallow water body bordered approximately 10 miles to the south by the barrier islands Cat Island and Ship Island. Typical depths in the sound range from 2 to 5 meters. The shoreline slope in the vicinity is relatively flat with the 2 meter depth contour located a few hundred yards offshore and as far as 1.5 miles offshore.

Sea bed materials are primarily fine sands and silt, with some areas of clay content and others, particularly offshore of Bay St. Louis, occupied by expansive oyster beds.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides and prevailing winds. Aerial photography suggests an east-to-west littoral drift in the site vicinity, as is typical for the Mississippi coast. Some local variation in the generalized east to west drift pattern may exist in the lee of the pier due to influences of shoreline infrastructure. The mean diurnal tide range at Harrison County 1.6 feet, and the extreme (except during storms) range is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline.

The Harrison County Seawall was originally constructed between 1925 and 1928 to protect Highway 90. The seawall is a stepped concrete type wall founded on piles. The seawall crest elevation varies between approximately 8 to 11 feet mean sea level and is penetrated in a number of locations by drainage channels and culverts.

The study area has been impacted by several tropical storms and hurricanes, most recently from Tropical Storm Isidore in 2002 and Hurricane Katrina in August 2005. The preliminary high water mark data indicate that H. Katrina surge reached a height of approximately 22 ft NGVD near the mouth of Biloxi Bay, and approximately 27-28 ft NGVD near Pass Christian. The project site covers approximately 24 miles between these locations. Eight major hurricanes rated above Category 3 on the Saffir-Simpson Scale have hit the Mississippi coast prior to Katrina during the period 1851 – 2004. During the period 1950-2004, the area was hit by Elena (1985), Camille (1969), and Frederic (1979). Some of the historic storm induced water surface elevations in the project vicinity are presented in Table 1. The county was also damaged by H. Betsy in 1965. Hurricane Katrina maximum surge heights exceed the previous record of Hurricane Camille by six to seven feet, and exceed by an order of nearly two the surge heights of all other storms.

Table 1.
Historic Maximum Storm Surge Elevations

Storm Event	Point of Landfall	High Water (ft NGVD)	Location of High Water
29 September 1915	Grand Isle, LA	12.8	Pass Christian, MS
July 1916	Gulfport, MS	10.8	Mobile, AL
19 September 1947	New Orleans, LA	15.2	Bay St. Louis, MS
12 September 1979 (H. Frederic)	Dauphin Island, AL	8.17	Dauphin Island, AL
17 Aug 1969 (H. Camille)	Waveland, MS	22.6	Pass Christian, MS
2 September 1985 (H Elena)	Biloxi, MS	6-8	-
29 August 2005 (H. Katrina)	Pearlington, MS	23.8	Biloxi Back Bay, Pt Cadet, MS
29 August 2005 (H. Katrina)	Pearlington, MS	27-28	Pass Christian, MS

Geotechnical Data

Typical profiles for this plan can be seen herein. There are a total of 24 possible borrow sites for this project, located at least 1500 feet offshore. Materials used for the re-nourishment and dune construction will have 90% passing the #40 sieve and only 10% will pass the #200 sieve. The sand fill shall not have noticeable amounts of shell and/or gravel. The sand will be pumped ashore and shaped along the proposed alignment.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

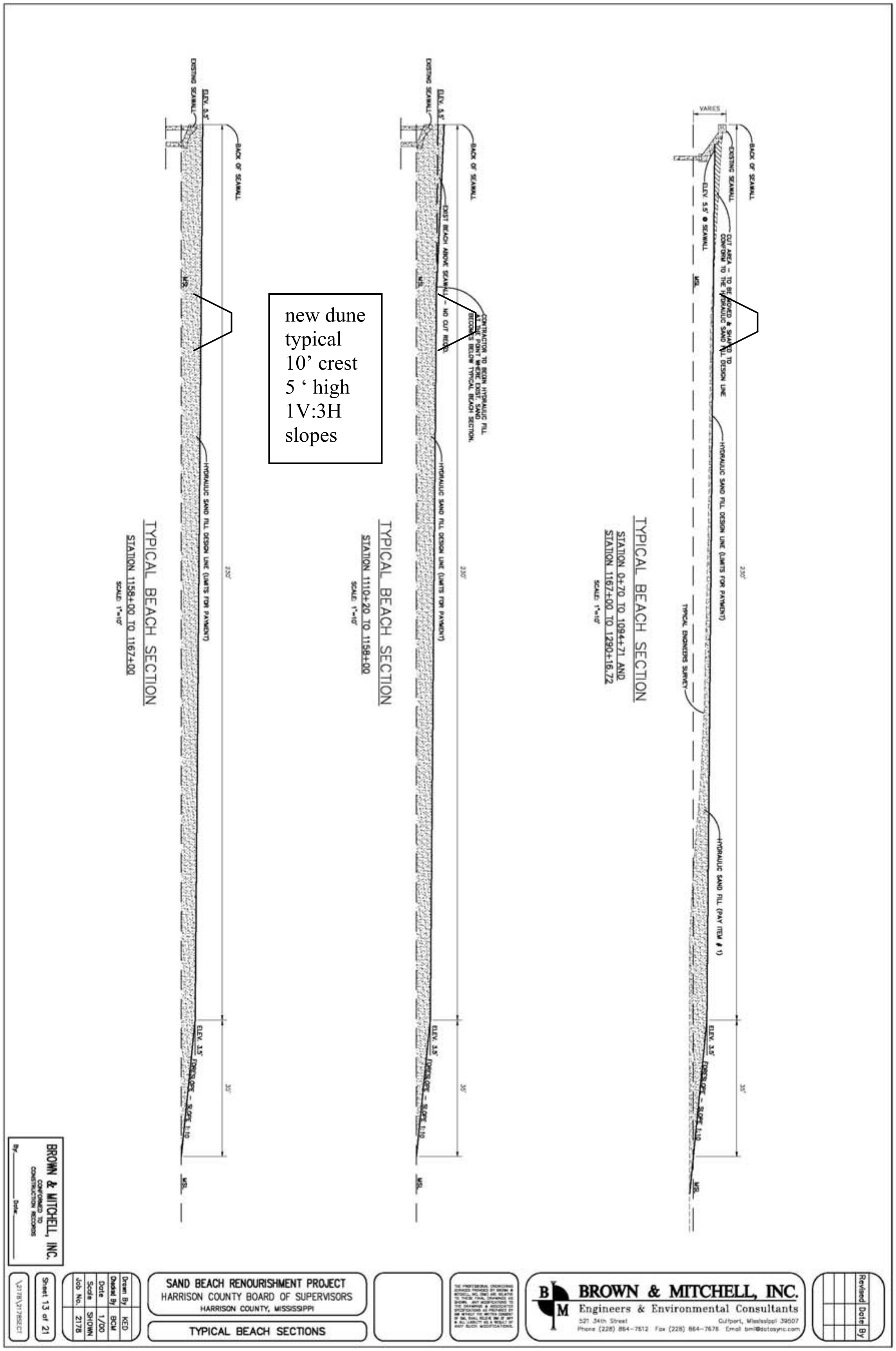
It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternative Plans

Two plans were evaluated for enhanced beach protection at the study site. All involved providing a dune atop the re-nourished beach. One alternative will be to place the dune material alone and the other alternative will be to place the dune material and add stabilizing fencing and dune vegetation. The finished stable dune will be 5 feet high with a crest width of 10 feet and side slopes of one vertical to three horizontal. The material will come from the established borrow areas a minimum of 1,500 feet offshore. The plantings will have a density of 1 plant per 4 square feet and the fence will include the entire linear length of the project. The dune alone project will require replacement within 10 years and the dune with plantings and fence will require replacement within 15 years.


new dune
typical
10' crest
5 ' high
1V:3H
slopes
TYPICAL BEACH SECTION
STATION 1158+00 TO 1167+00
TYPICAL BEACH SECTION
STATION 1110+20 TO 1158+00
TYPICAL BEACH SECTION
STATION 0+70 TO 1094+71 AND
STATION 1167+00 TO 1290+16.72
EXISTING SEAWALL
BACK OF SEAWALL
HYDRAULIC SAND FILL DESIGN LINE (LIMITS FOR PAYMENT)
HYDRAULIC SAND FILL (PAY ITEM # 1)
TYPICAL ENGINEERS SURVEY
BROWN & MITCHELL, INC.
SAND BEACH RENOURISHMENT PROJECT
HARRISON COUNTY BOARD OF SUPERVISORS
HARRISON COUNTY, MISSISSIPPI
TYPICAL BEACH SECTIONS
BROWN & MITCHELL, INC.
Engineers & Environmental Consultants
Sheet 13 of 21

Construction Procedures and Water Control Plan

Beach material will be dredged from established borrow areas within 1,500 feet offshore and pumped to the shoreline, where it will be graded to suit by earthworking equipment. Construction surveys will be necessary to lay out the design beach template and to confirm as-built grading meets design intents.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

The “dune alone” alternative will require replacement within 10 years and the “dune with plantings and fence” alternative will require replacement within 15 years. Both alternatives will require removal of wind-blown sand from Highway 90 by street sweeping equipment, and transferal of wind-blown sand from the lee of the dunes to the front. It is estimated that relocation of sand due to ‘normal’ wind and weather will be required twice annually with a total estimated annual amount to be relocated of no more than 0.25 cubic feet of sand per foot of beach (approximately 32,000 cubic yards) (reference 1) for the “dune alone” alternative, the “dune with plantings and fence” alternative requiring perhaps 70% of this effort.. Severe storms, such as hurricanes, could severely damage the project regardless of the presence or absence of fencing and vegetation and require replacement of the dunes. The base of the dune is at elevation 5.5 NGVD. If the still-water elevation at the base of the dune is the elevation at which storm surge, with additional wave action, would begin to erode the dune, an approximately 10-year recurrence interval surge corresponds to this elevation based on frequency analysis of annual maximum water surface elevations at Biloxi.

Cost Estimates

Estimated costs for the alternative plans are shown in Table 2. Quantity estimates are based on drawings and surveys provided. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 2.
Estimated Costs

ALTERNATIVES	QUANTITY	UNIT	ESTIMATED COST
BEACH DUNE	681,000	CY	
TOTAL		LS	$10,220,000.
ANNUAL O & M		LS	$340,000.
DUNE	681,000	CY	
FENCING	134,000	LF	
PLANTING	125	ACRE	
TOTAL		LS	$13,580,000.
O & M		LS	$260,000.

The selected alternative of providing fencing and plantings to supplement the dune system is based on the reduction in replacement needs since the borrow areas are of finite quantity and this resource must be prolonged as much as possible.

Schedule for Design and Construction

A typical schedule for preparation of P&S through construction is shown in Table 3 below.

Table 3.
Typical Schedule for P&S

Draft P&S	1 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	6 months after NTP

References

1. Sand Beach Planning Team (1986). “Sand Beach Master Plan, Harrison County, Mississippi.” Mississippi Department of Wildlife Conservation, Bureau of Marine Resources.

2. U.S. Army Corps of Engineers, “Design of Coastal Revetments, Seawalls, and Bulkheads”, Engineer Manual No. 1110-2-1614.

3. U.S. Army Corps of Engineers, “Engineering and Design for Civil Works Projects”, Engineer Regulation No. 1110-2-1150.

4. U.S. Army Corps of Engineers, “Hydraulic Design for Coastal Shore Protection Projects”, Engineer Regulation No. 1110-2-1407.

COURTHOUSE ROAD

General

The purpose of this document is to provide engineering information for planning and design analysis for environmental restoration and drainage channel repairs at Courthouse Road Pier, City of Gulfport, Harrison County, Mississippi.

Location

The site is located in Gulfport on Mississippi Sound. Harrison County is the central coastal county in Mississippi, Gulfport is Mississippi's second largest city with a circa 1993 population exceeding 70,000 and is 75 miles by road west of Mobile, Alabama and 78 miles east of New Orleans, Louisiana. Nearly the entire length of the county shoreline is fronted by four-lane Highway 90, which is protected by a concrete seawall.

The project site is seaward of Highway 90, known locally as East Beach Boulevard, at the Courthouse Road intersection. The site is occupied by the Courthouse Road Pier, a public fishing and boat launch facility; a sand beach fronting the seawall; and a concrete sheet-pile walled open channel drain typical of those on the county shoreline. The site location is shown on Figure 1.

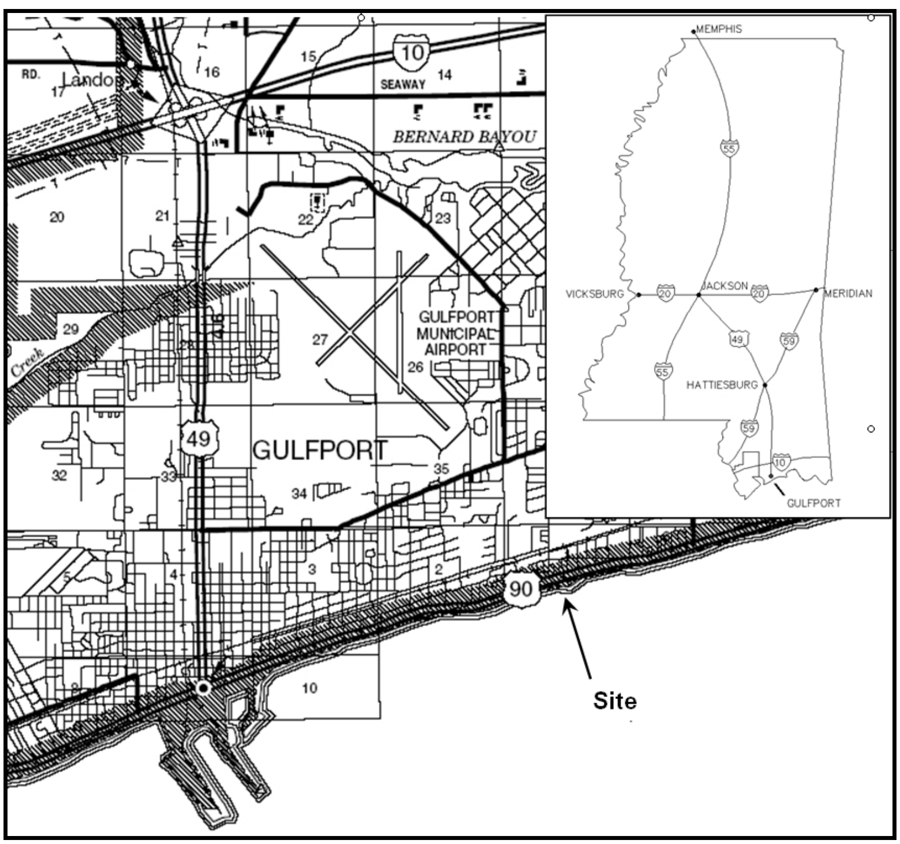


Figure 1. Location Map

Existing Conditions

Figure 2 shows pre- and post-Katrina aerial views of the site. The pier was originally extended from fill placed over an existing beach groin in the late 1940's. The pier was extended 400 feet in 1992, at which time an area of the beach between the open channel drain and the original groin was filled to accommodate expansion of the parking lot (Reference 1). Construction was underway in the summer of 2005 to improve the pier and boat launch facility. Notable improvements included a new boat ramp, ramp approach jetties and markers, parking lot revisions, and a mitigation marsh. The pre-Katrina view in Figure 2 pre-dates these recent improvements and shows the pre-improvement marsh; Figure 3 shows the facility improvement plan overlain on the post-Katrina aerial. The marsh (Figure 4) is not evident in Figures 2 and 5, having been completed approximately two weeks before Hurricane Katrina struck (Reference 2). It was destroyed by the hurricane, as were other existing and in-progress features of the public facility.

The seawall was originally constructed between 1925 and 1928 to protect Highway 90 (Reference 3). The seawall is a stepped concrete type wall founded on piles. The seawall crest elevation varies between approximately 8 to 11 feet mean sea level and is penetrated in a number of locations by drainage channels and culverts.

The existing drainage channel issues from the seawall and was probably completed by 1952, the year the Harrison County Shore Protection project was completed. That project provided for shoreline drainage improvements, seawall repairs, and beach construction along 24 miles of the Harrison County waterfront resultant mainly from the destructive 1947 hurricane.

The drainage channel (Figures 6 and 7) is approximately 235 feet long with a flow width of approximately 12.5 feet. The channel is a stormwater network discharge point. The stormwater network consists primarily of drainage pipes connected to a trunk line beneath the seawall. The channel walls are tongue-and-groove concrete sheetpile panels with a concrete cap. The top of cap elevation slopes about 1 percent from the channel headwall to the channel terminus. Fourteen lateral braces originally spanned the channel to provide active support to the channel walls. The braces were displaced during Hurricane Katrina and all appear damaged, though 10 of these were recovered and placed back on top of the wall as a temporary measure. The braces are made of reinforced concrete of dimensions 12 inches wide by 11 inches deep and approximately 13.5 feet long.

Figure 2. Post-Katrina (left) and Pre-Katrina Aerial Site Photos

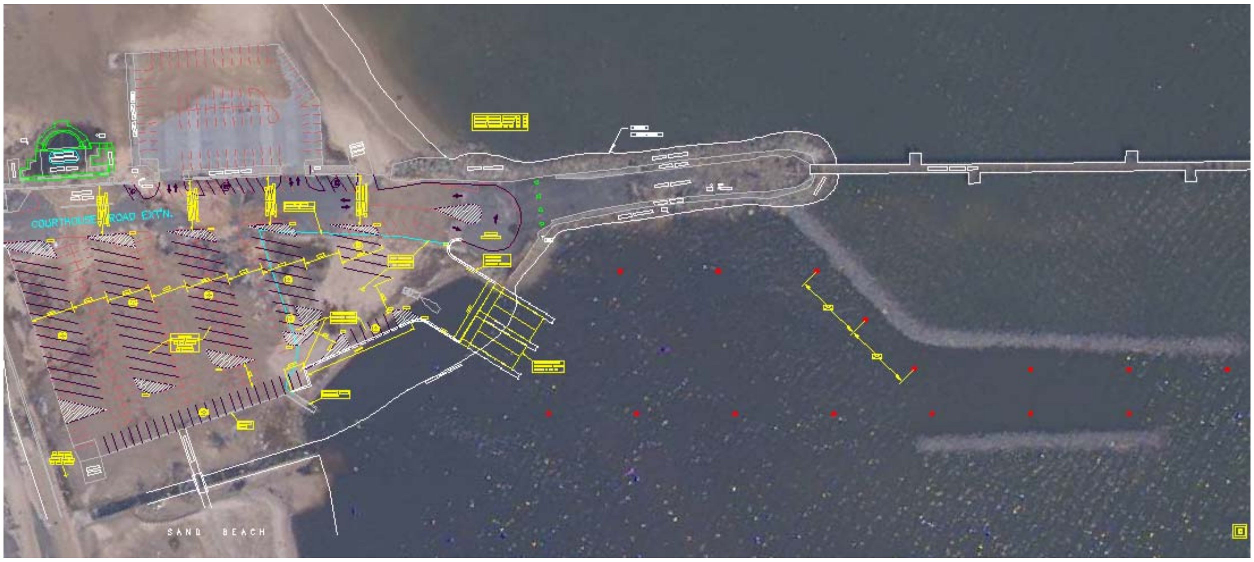


Figure 3. Plan of Improvements Overlain on Post-Katrina Photo. Jetty construction was progressing towards land as in Figure 4.4. Design by Brown and Mitchell, Gulfport, MS

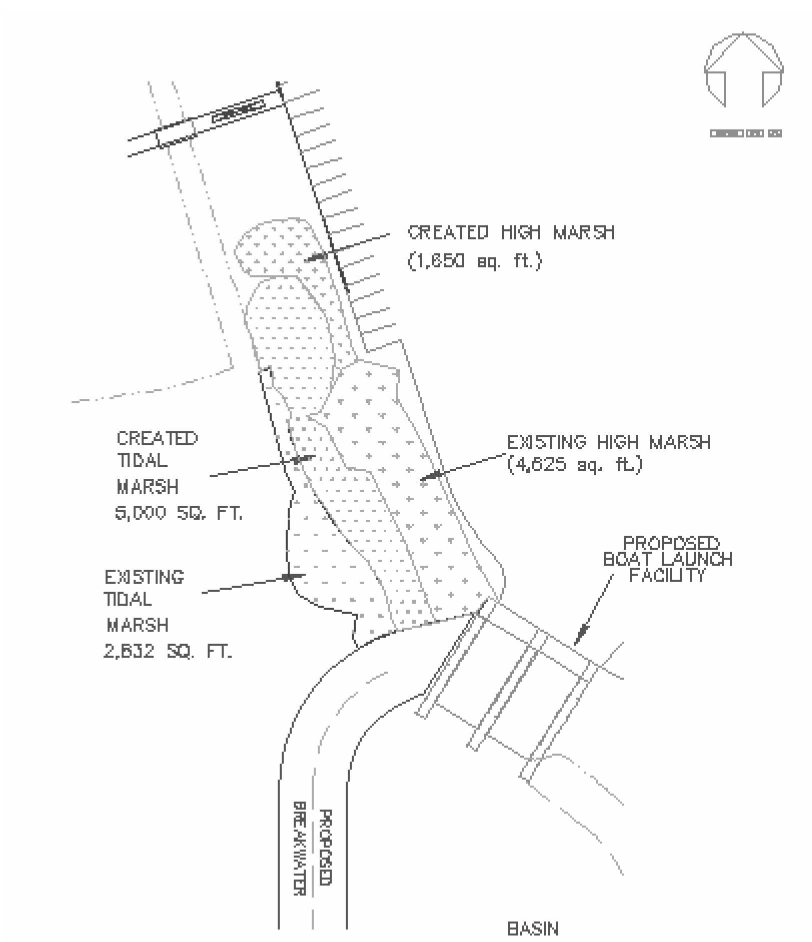


Figure 4. Facility Improvement Mitigation Marsh Plan Detail. Design by Brown and Mitchell, Gulfport, MS

Figure 5. Looking towards location of destroyed marsh. Approximately 100 feet of erosion in the vicinity of the marsh occurred during Katrina. 3 April 2006 photo

Figure 6. Drainage channel, looking towards the seawall. Broken braces are shown stacked on the beach to the left of the channel. 3 April 2006 photo

Figure 7. Drainage channel, looking towards the beach. Ends of braces are cracked or broken, and all braces show impact damage on at least one surface 3 April 2006 photo.

Coastal and Hydraulic Data

The climate in the site area is subtropical, characterized by warm summers and short, mild winters. The average daily temperature ranges in the summer and winter are 72–89 and 42–63 degrees Fahrenheit, respectively. The average annual rainfall is about 60 inches, and is well distributed throughout the year. Precipitation records indicate July as the wettest month, while October is the driest. The climactic summary for the Gulfport Naval Center weather station is shown on Table 1.

Table 1.
Climactic Summary, Gulfport Naval Center, MS (Station No. 223671)

Period of Record : 1/ 1/1948 to 9/30/2005

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	60.9	64.1	69.7	76.7	83.7	88.7	90.7	90.6	87.0	79.6	70.4	63.1	77.1
Average Min. Temperature (F)	42.5	45.5	51.0	58.6	66.1	71.5	73.5	73.0	69.2	59.0	50.4	44.4	58.7
Average Total Precipitation (in.)	5.46	5.15	5.67	5.23	4.85	5.45	7.33	5.83	6.91	3.00	4.18	5.06	64.12

Source: Southeast Regional Climate Center

Mississippi Sound is a shallow water body bordered approximately 10 miles to the south by the barrier islands Cat Island and Ship Island. Typical depths in the sound range from 2 to 5 meters. The shoreline slope in the vicinity of Gulfport is relatively flat with the 2 meter depth contour located a few hundred yards offshore and as far as 1.5 miles offshore.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides and prevailing winds. Aerial photography suggests an east-to-west littoral drift in the site vicinity, as is typical for the Mississippi coast. Some local variation in the generalized east to west drift pattern may exist in the lee of the pier due to influences of the pier facility infrastructure and discharge from the drain channel. The mean diurnal tide range at Harrison County is 1.6 feet, and the extreme (except during storms) range is about 3.5 feet. The velocity of normal tidal currents ranges from 0.5 to 1.0 foot per second (fps) and their direction is generally east to west. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The study area has been impacted by several tropical storms and hurricanes, most recently from Tropical Storm Isidore in 2002 and Hurricane Katrina in August 2005. Hurricane Katrina surge estimated from high-water marks range from 23 to 25 feet mean sea level near the site (Reference 4). Based on the frequency curve for Biloxi, Mississippi, a community to the east of Gulfport in Harrison County, these heights suggest a 250 year recurrence interval event.

Geotechnical Data

The site lies directly on the coastal interface with the Mississippi Sound and has been altered with the construction of the seawall and related drainage structures in the late 1920’s and early 1930’s. The pier facility with parking, boat ramp and other structures was added years later. The stratigraphy of the work area is characterized by poorly graded loose to medium dense sands and silty sands from the surface to El. -5.0 NGVD. This is underlain by loose, silty sands with possible pockets of organics from El. -5.0 to -15.0 NGVD. These materials are further underlain by denser poorly graded and silty sands for the subsequent 20 to 30 feet.

Material used for marsh creation will require a greater silt and organic content than found locally at the shoreline and will be imported from off site sources within 5 miles of this site.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternative Plans

Three plans were evaluated for shore protection at the study site.

- ***No Action Alternative:*** This alternative assumes that the drainage channel bracing is not repaired and that the marsh is not replaced. If the bracing is not replaced, it is assumed that the bracing will cease to be effective due to displacement by breaking waves for events exceeding the 7 feet NGVD elevation (approximately the 15-year recurrence interval event) and that failure of a significant portion of the channel walls would accompany that event. This alternative also assumes that the marsh would not re-establish itself.
- ***Alternative 1: Replace Open Channel Drain Lateral Bracing.*** This alternative would involve removal and disposal of all fourteen (14) of the original concrete braces. The braces would be replaced by reinforced pre-cast concrete braces that would be anchored to the pile wall cap. A typical brace design has been developed and is shown in Figure 8.

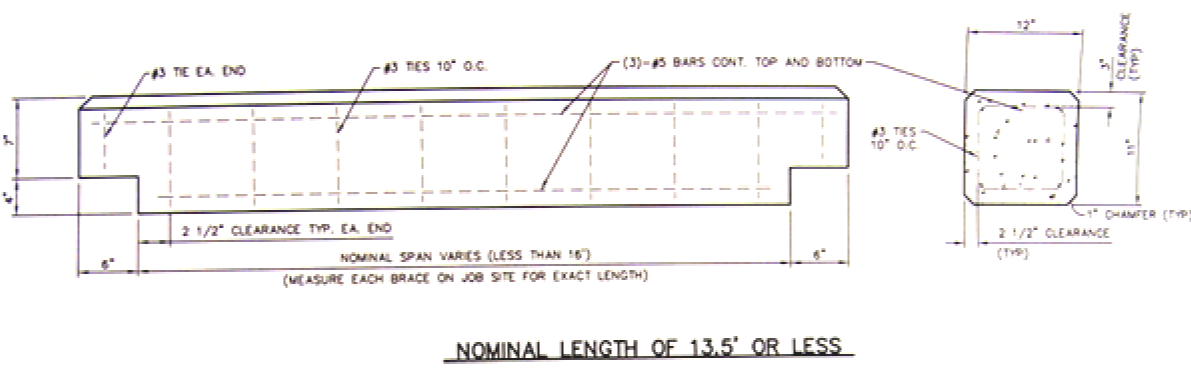


Figure 8. Typical Brace Elevation and Section. Design by Brown and Mitchell, Gulfport, MS

- ***Alternative 2: Marsh Restoration.*** This alternative would replace the existing (prior to improvement) and mitigation high marshes and tidal marshes in the areas shown on Figure 4. Approximately one-third of an acre of marsh would be created, composed of approximately 6,300 square feet of high marsh and 7,900 square feet of tidal marsh. High marsh would be established by grading the existing sandy soils and adding soils to suit for planting high marsh species. Tidal marsh would be established by placing suitable soils and planting tidal marsh plant species within. Assuming an average depth of soil placement to be 3 feet, and that the entirety of the marsh area was eroded to mean low water as suggested by the post-hurricane photo, approximately 1,500 cubic yards of soil would be required.

- ***Alternative 3: Replace Open Channel Drain Lateral Bracing and Marsh Restoration.*** This alternative is a combination of alternatives described in sections 7.2 and 7.3 as described above without additions or deletions.

Construction Procedures and Water Control Plan

Soils suitable for marsh development would be obtained from inland sources within five miles of the site and delivered by truck via the facility entrance. The soil would be dumped at the location of the marsh, graded to suit by light earth-moving equipment, and planted with suitable plant species.

The channel wall pile caps would be prepared to receive new braces. Chipped, damaged, cracked, or otherwise eroded concrete at the replacement brace seats would be patched with durable material. Brace anchors would be set on either end of the brace location. Replacement braces would be pre-cast off-site and transported to the site by truck, where they would be placed mechanically upon the wall.

Project Security

There is no reason to believe that this public facility constitutes a high-priority terrorist target.

Operations and Maintenance (O&M)

Alternative 1, ‘Replace Channel Lateral Bracing,’ does not introduce new features or elements to the coastal waterfront, and therefore, no new operating and maintenance costs should be incurred as a result of that alternative. The other alternatives, unless otherwise not provided for, should be monitored for marsh vegetation survival twice per year. Estimated O&M costs are shown in Table 2.

Table 2.
Annual O&M Costs

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Courthouse Road	ITEM NO.	Summary	DATE	9-May-06
LOCATION:	Harrison County MS.	SHEET NO.	1	OF	3
		PREPARED	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Summary - O and M annual cost***	BASIS of ESTIMATE:	info furnished per PDT Team		
		FILE NAME:	o-m courthouse road4-22.xls		

Alt No.	DESCRIPTION	Quantity	Unit	ESTIMATED AMOUNT
1.	***Replace Channel Lateral Bracing***			-n/a-
2.	***Marsh Restoration***	1	job	$5,000
3.	***Marsh Restoration and Replace Channel Lateral Bracing***	1	job	$5,000

Cost Estimates

Alternatives cost estimates are shown in Tables 3 through 5. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 3.
Replace Lateral Channel Bracing Estimated Costs

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Courthouse Road	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Harrison County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Replace Open Channel Drain Lateral Braci***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	courthouse road6-26.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Replace Open Channel Drain Lateral Bracing					
Mobilization, Preparatory Work, Demobilization		1	job	allow	25,000
Remove existing braces		14	ea	500.00	7,000
New precast conc braces		14	ea	2,500.00	35,000
Anchor to wall		28	ea	500.00	14,000
Misc. Site Items		1	ls	allow	20,000
	Total Direct Construction Cost				$101,000
	Indirect Cost	@		15%	15,150
					116,150
	Profit	@		9%	10,454
					126,604
	Bond	@		1.5%	1,899
		Current Contract Cost, Oct 06			$128,503
	01 Account, Lands & Damage (PCA)			LS	75,000
					203,503
	30 Account, Plan, Engr.& Design			10%	20,350
					223,853
	31 Account, Constr. Management			6%	13,431
					237,284
	CONTINGENCY			20%	32,457
					269,741
					$269,741
					rounded
		TOTAL PROJECT COST, FY-07			**$270,000**

Table 4.
Marsh Replacement Estimated Costs

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Courthouse Road	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Harrison County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Wetland Restoration***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	courthouse road6-26.xls		

D E S C R I P T I O N	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Wetland Restoration				
Mobilization, Preparatory Work, Demobilization	1	job	allow	25,000
fill material	1,500	cy	20.00	30,000
grade site	1	ls	10,000.00	10,000
plantings	0.5	ac	10,000.00	5,000
Misc. Site Items	1	ls	allow	20,000
	Total Direct Construction Cost			$90,000
	Indirect Cost	@	15%	13,500
				103,500
	Profit	@	9%	9,315
				112,815
	Bond	@	1.5%	1,692
06 Account, Fish & Wildlife		Current Contract Cost, Oct 06		$114,507
	01 Account, Lands & Damage (PCA)		LS	75,000
				189,507
	30 Account, Plan, Engr.& Design		10%	18,951
				208,458
	31 Account, Constr. Management		6%	12,507
				220,965
	CONTINGENCY		20%	29,193
				250,159
				$250,159
				rounded
	TOTAL PROJECT COST, FY-07			**$250,000**

Table 5.
Replace Lateral Channel Bracing and Marsh Replacement Estimated Costs

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Courthouse Road	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Harrison County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Replace Open Channel Drain Lateral Braci***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	courthouse road6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Combination Bracing and Wetland Restoration				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
Remove existing braces	14	ea	500.00	7,000
New precast conc braces	14	ea	2,500.00	35,000
Anchor to wall	28	ea	500.00	14,000
Misc. Site Items	1	ls	allow	40,000
fill material	1,500	cy	20.00	30,000
grade site	1	ls	10,000.00	10,000
plantings	0.5	ac	10,000.00	5,000
Total Direct Construction Cost				$191,000
Indirect Cost	@		15%	28,650
				219,650
Profit	@		9%	19,769
				239,419
Bond	@		1.5%	3,591
Current Contract Cost, Oct 06				$243,010
01 Account, Lands & Damage (PCA)			LS	150,000
				393,010
30 Account, Plan, Engr.& Design			10%	39,301
				432,311
31 Account, Constr. Management			6%	25,939
				458,249
CONTINGENCY			20%	61,650
				519,899
				$519,899
				rounded
TOTAL PROJECT COST, FY-07				**$520,000**

Design and Construction Schedule

A typical schedule for preparation of P&S through construction is shown in Table 6.

Table 6.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 weeks after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

Meyer-Arendt, K.J. (1995). “Beach and Nearshore Sediment Budget of Harrison County, Mississippi: A Historical Analysis.” Open-File Report 43. By Department of Geosciences, Mississippi State University for Mississippi Office of Geology, Department of Environmental Quality. September 1995.

Personal Communication, 4 April 2006. K. Gunter, Brown and Mitchell.

US. House of Representatives (1948). “Harrison County, Miss., Beach Erosion Control Study.” Document No. 682, 80th Congress, 2d. Session. 28 May 1928.

FEMA (2006). “Hurricane Katrina Rapid Response, Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection.” FEMA-1604-DR-MS. Prepared for FEMA Region 4 by URS Group, Gaithersburg, MD. Draft Report, 16 January 2006.

SHEARWATER BRIDGE

General

The purpose of this document is to provide engineering information for planning and design analysis as related to protection of the approaches and abutments for the Shearwater Bridge that were damaged by Hurricane Katrina storm surge in Jackson County, Mississippi.

Location

The bridge is located in Jackson County, the easternmost coastal county in Mississippi. It is located on Mississippi Sound 54 miles west of Mobile, Alabama and about 93 miles east of New Orleans, Louisiana. The bridge is located on Shearwater Drive in Ocean Springs, MS, on a paved road at the east end of the Ocean Springs harbor as shown below.

Existing Conditions

The existing timber retaining walls protecting both approaches and abutments to the bridge are failing. The timber has deteriorated and the walls were inundated by the storm surge, which caused additional failure and loss of fill material. This bridge also is a local evacuation route. Another strong storm surge could cause the bridge to fail or the approaches to become impassable. Figures 1 through 9 show the project site, location, bridge station, partial plan for bridge replacement in 2003, and several photographs, including an aerial view of the bridge, approach to the bridge, and abutments.

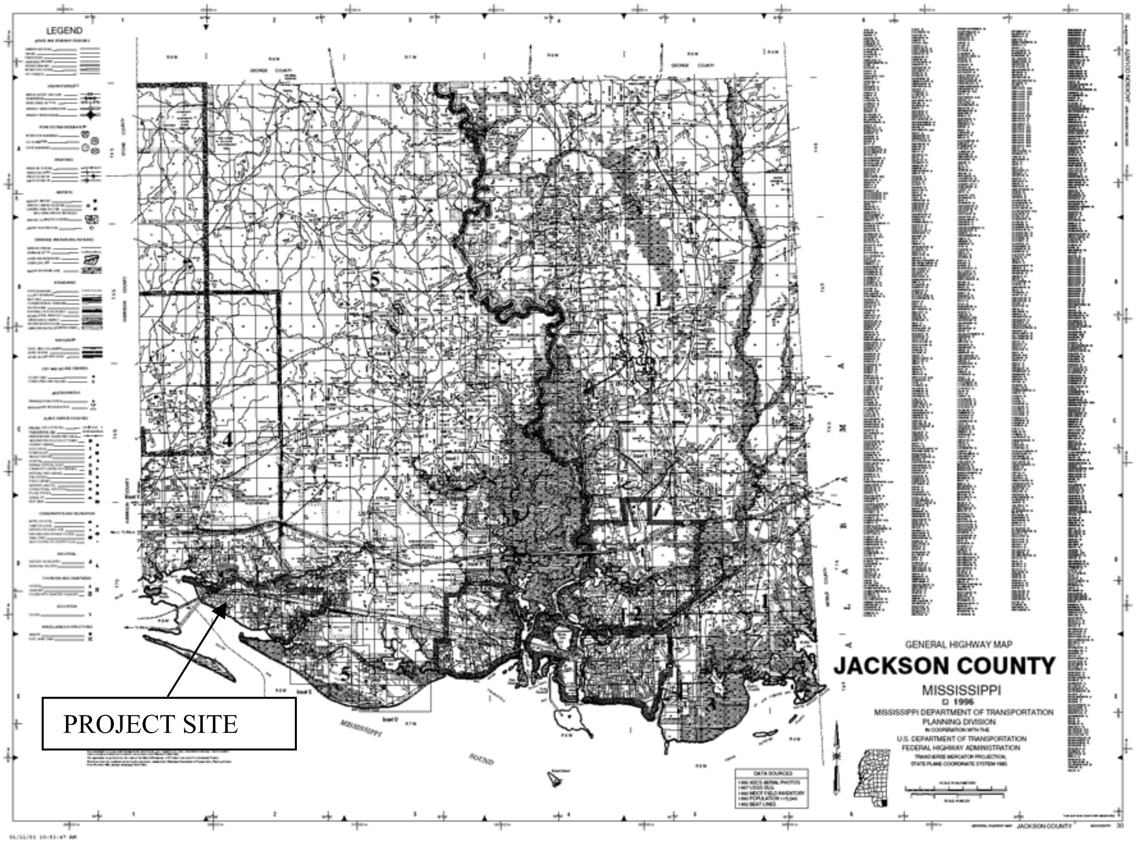


Figure 1. Project Site

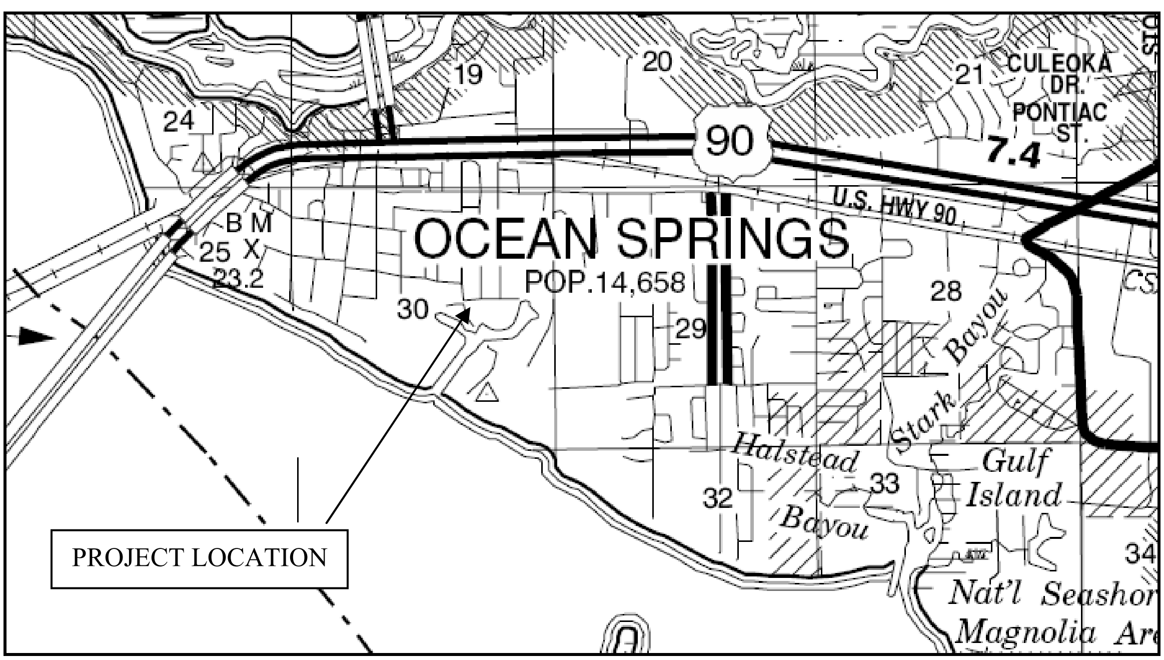


Figure 2. Project Location

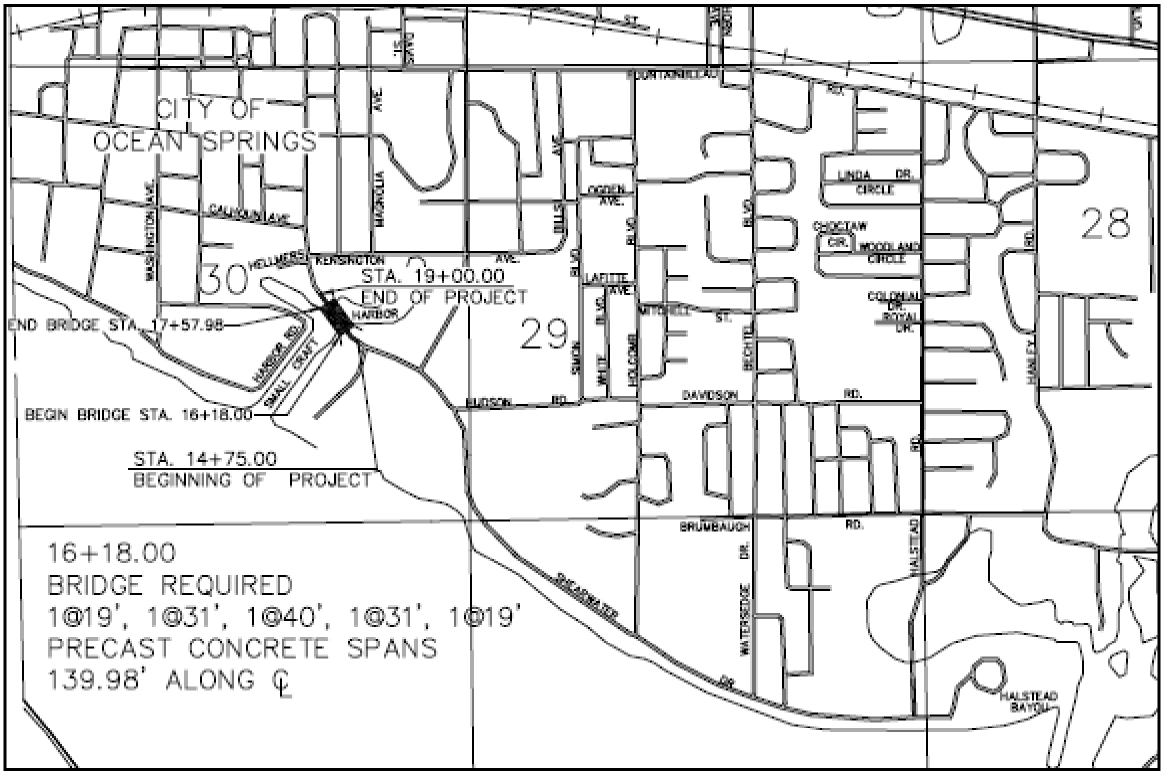


Figure 3. Bridge Station

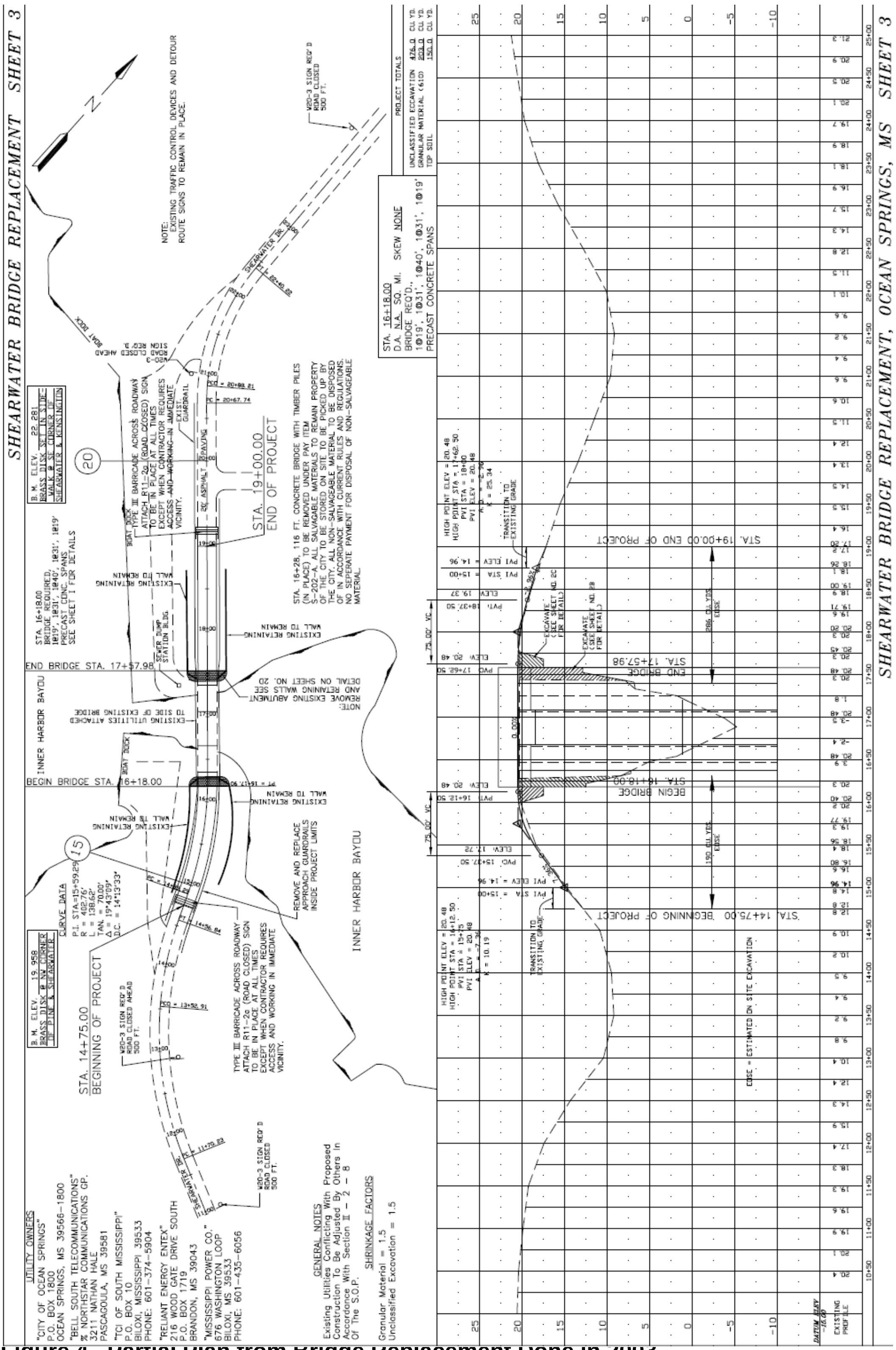


Figure 4. Partial Plan from Bridge Replacement Done in 2003

Figure 5. Aerial View of Shearwater Bridge

Figure 6. Timber Wall Failure at North Abutment

Figure 7. View of North Approach


Figure 8. East Side of North Abutment

Figure 9. West Side of South Abutment

Figure 10. Shearwater Bridge Inundation Limits and 21-ft. Contour

Coastal and Hydraulic Data

Storm surge inundation limits at Shearwater Drive Bridge site from Katrina are estimated approximately at elevation 21 ft NGVD as shown in Figure 10.

Additional data is provided in a report to FEMA by URS Group, Inc., titled “Hurricane Katrina Rapid Response Mississippi Coastal and Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report,” 16 January 2006. A draft flood frequency analysis of tide gage data of available tide data was made to quickly provide information to assist in the planning and rebuilding efforts while more detailed analyses are being conducted. The results of this study are provided in “Draft Report, Hurricane Katrina Flood Frequency Analysis,” dated September 2005, and are summarized below.

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b). The flood frequency analysis only represents conditions at and near the gage.

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.
- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.
- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.
- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet.
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet.
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet.

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 1. As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 1.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations
Location Katrina Elevation

Location	Katrina Elevation (Ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
Dauphin Island, AL	5.81	20
Pensacola, FL	6.07	50

Geotechnical Data

The project lies within the Ocean Springs metropolitan area at the head of the Jackson County Harbor entering the Mississippi Sound. The bridge carries Shearwater Road over the upland harbor channel. The bridge elevation is approximately El. 20 NGVD with the approaches falling at a 3 percent grade to El. 10 to the west and falling at a grade of 7.4 percent to the east to El. 10. The toes of the south side of the approach embankments are accessible from a road that connects to the approach ends and runs to the channel edge on each side. This road allows access to the bulkhead and slips along the harbor channel. The bridge was replaced in 2003 but the embankments and any stabilization efforts remain from the original construction. The approach side embankments are extremely steep (>1V:1H), rendering conventional slope protection unstable. Attempts have been made to stabilize the slopes through a combination of timber bulkheading with closely spaced piled installed for lateral support and concrete rubble. These walls have deteriorated to the point of failure and no longer provide adequate support. The existing embankments have been constructed by placing compacted poorly graded sands and silty sands from El. 5.0 to El. 20. Medium to dense poorly graded sands and silty sands can be expected below El. 5.0 with more silty sands and possible organic content beyond El. -10.0, becoming less silty beyond El. -15.

The alternative solutions provide for various types of sheet piling to be driven from the edge of the concrete abutment wall to the sag points of the embankment approaches on all four sides. The design of the sheet piles should be based on soils having an in place density of 110 PCF, a cohesion of 300 PSF and an angle of internal friction of 25 degrees. The soils will assume to be saturated below El. 3.0 NGVD. The new wall can be access from the entire south side so horizontal tie rods can be installed by drilling under the road through each of the side of the wall. Lateral earth pressure coefficients can be derived from the soil values provided but the wall penetration should be on the order of 1.5 times the unsupported length for any section of wall. The shoulder of the road should be covered with base course stone material and the new fills placed behind the wall should comprise of clean poorly graded sands from off site sources within 5 miles of the project and hand compacted to 90 percent of the materials maximum modified density in 4 inch lifts. The exposed slope below the wall should be covered by a 12 inch layer of riprap underlain by a nonwoven filter fabric. The fabric should be anchored top and bottom and the riprap should have a thickened keyed section at the toe.

HTRW

Is not applicable to this project

Alternative Plans

Three plans were evaluated for bridge protection at the study site. All involved extending and raising the protective walls from what was existing. Figure 11 shows a cross section of a protective wall.

- ***Steel Sheet piling.*** This alternative would consist of the installation of continuous interlocked steel sheet piling along both sides of the north and south approaches of the bridge. The total of sheet pile wall would be approximately 675 feet the top elevation varies from elevation 20.48 ft to elevation 12, the average height is 17 feet. The sheet pile bulkheads would be anchored to each other by using steel tie rods under the roadway; filter fabric will be placed behind the bulkhead and the bulkhead would be backfilled with gravel and sealed at the top with a reinforced concrete cap.
- ***Vinyl Sheet piling.*** The vinyl sheet pile alternative is essentially the same as the steel alternative except for the sheet pile material. There would some different considerations for material thickness and anchorage spacing, but otherwise the plans would be very similar.
- ***Timber.*** This option would be similar to what is there now, with the exception of extending and raising the walls. More substantial piling and depth of embedment would be used. Filter fabric and proper backfill will also be used.

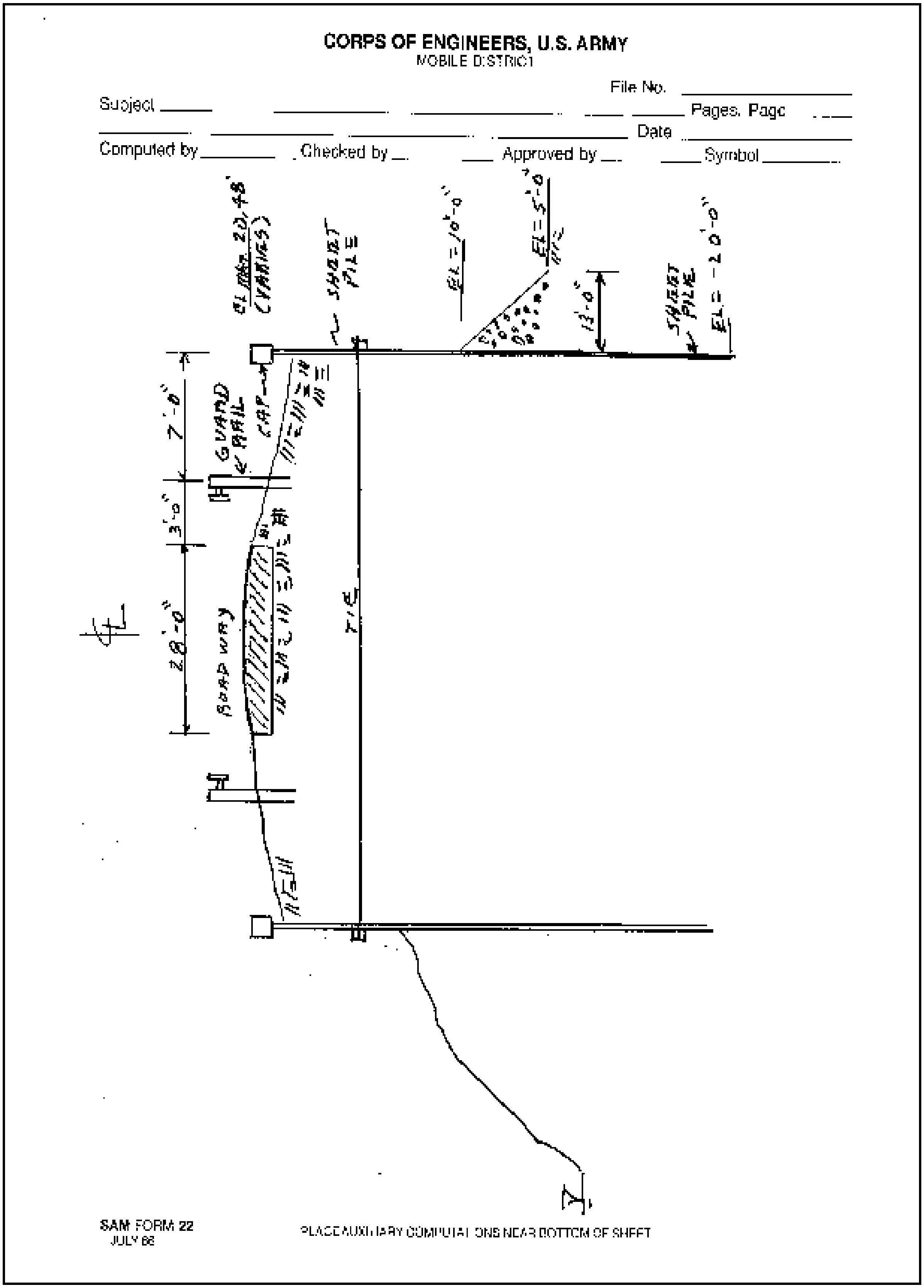


Figure 11. Cross Section of Proposed Wall

Structural Considerations

In assessing the relative value and effectiveness of steel sheet piling and vinyl sheet piling, the durability provided was of paramount importance. The forces imposed on these sheet pile structures will be moderate to high, consisting of vertical weight of a portion of the small concrete cap, backfill pressures generated by the sand fill placed behind the bulkhead, roadway or vehicular loads, and periodic lateral seepage pressures generated by migrating water. The sheet pile sections used should be more than adequate to resist these pressures. The critical factor will be the ability of the bulkhead to resist saltwater corrosion. Considering all these parameters, the vinyl or some vinyl composite Z-pile, engineered for the purpose, is considered best for this application. These materials are extremely resistant to saltwater environments, are strong enough to resist loads imposed on the bulkhead, and therefore should serve almost indefinitely in providing protection for the bridge approaches.

Driving the sheet pile should be best accomplished from the roadway.

Cost Estimates

Estimated costs for each alternative plan are shown in Table 2. Quantity estimates are based on drawings and surveys provided. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 2.
Estimated Costs

Alternatives	Quantity	Unit	Estimated Cost
Timber Wall	4	LS	$850,000
Vinyl Sheet pile	4	LS	$1,480,000
Steel Sheet pile	4	LS	$1,810,000

Schedule

A typical schedule for preparation of P&S through construction is shown in Table 3 below.

Table 3.
Typical Schedule for P&S

Draft P&S	1 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	1 months after NTP

References

U.S. Army Corps of Engineers, “Design of Coastal Revetments, Seawalls, and Bulkheads,” Engineer Manual No. 1110-2-1614.

U.S. Army Corps of Engineers, “Engineering and Design for Civil Works Projects,” Engineer Regulation No. 1110-2-1150.

U.S. Army Corps of Engineers, “Hydraulic Design for Coastal Shore Protection Projects,” Engineer Regulation No. 1110-2-1407.

GAUTIER COASTAL STREAMS

General

The hurricanes of 2005 caused damage to drainage ways by blowing trees and other debris into these areas and by deposition of sediment in many areas. This document provides information regarding damage to several drainage way sites and presents quantity and cost estimates for restoring the capacity of the streams.

Location

The first location is a drainage way crossing Old Spanish Trail highway and the others are located at the mouths at Mississippi Sound. They are:

- Unnamed Bayou (located at the southern end of Ladnier Road).
- Seacliffe Bayou (located immediately east of Seacliffe Drive).
- Unnamed Bayou (located south of Hiram Drive).
- Graveline Bayou (located at the western end of Graveline Road).

The sites are located in Jackson County in Gautier, MS. These sites are shown below.

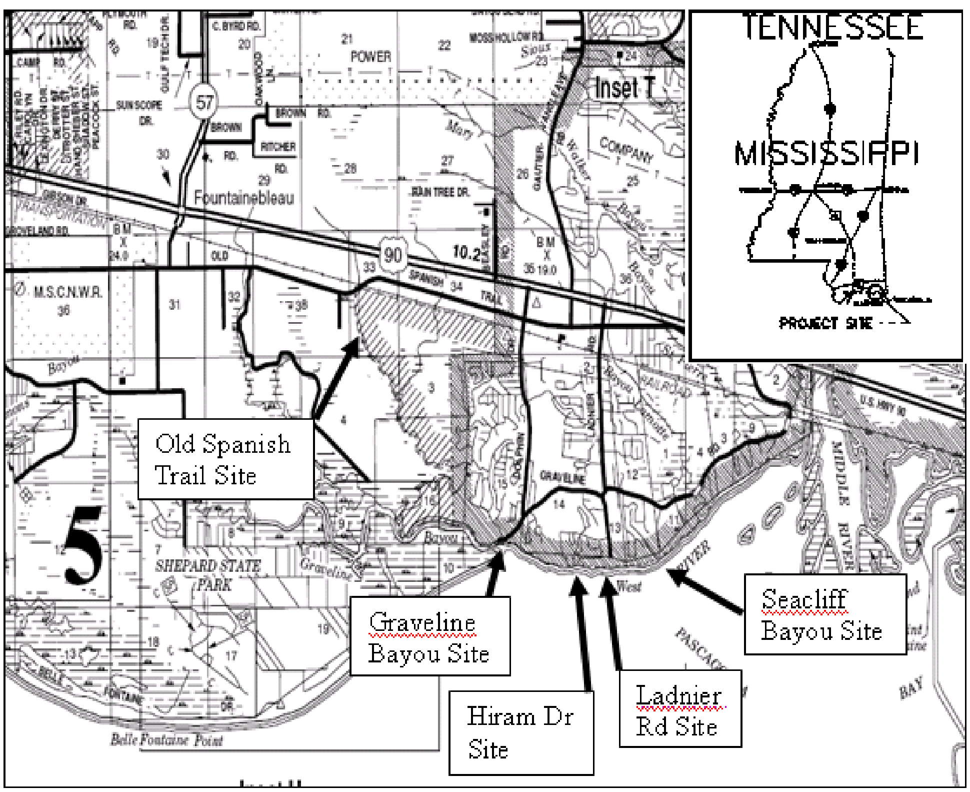


Figure 1. Location of Gautier Sites

The sites were evaluated separately, then combined to form a combination 2-3 foot excavation plan and a 1 foot excavation plan. Cost estimates are provided for the initial costs of the two combination plans, but individual site maintenance costs are reported.

Old Spanish Trail Site

Existing Conditions

Trees, debris and sediment are blocking drainage in a previously improved stream for approximately 1750 ft downstream of Old Spanish Trail in the area shown on the figures below.

Figure 2. Old Spanish Trail Site

Figure 3. Sediment in Drainage Way at Old Spanish Trail

At the end of the previously improved section, the flow enters a wooded area which extends approximately 1000 ft, then the flow reaches a larger area of tidal water. This is shown on the figure below.

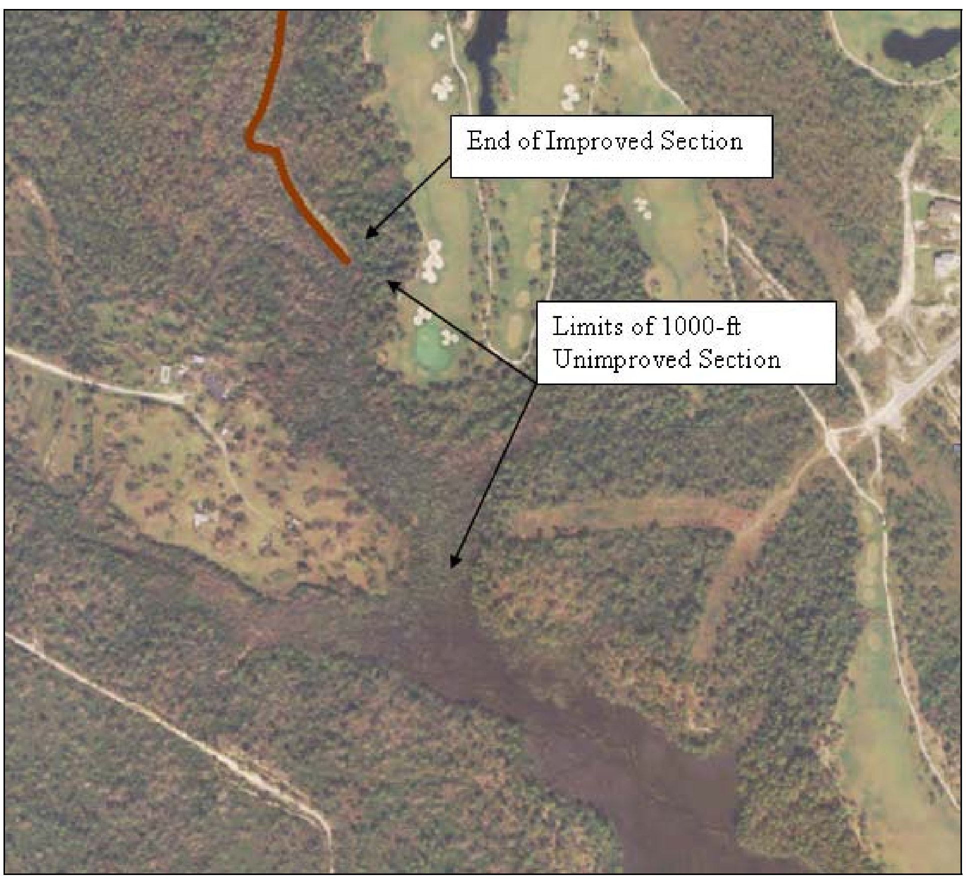


Figure 4. Limits of Old Spanish Trail Unimproved Channel

Coastal/Hydraulics

Storm surge inundation limits provided by FEMA at Old Spanish Trail site from Katrina are estimated approximately at elevation 19 ft NGVD as shown below.

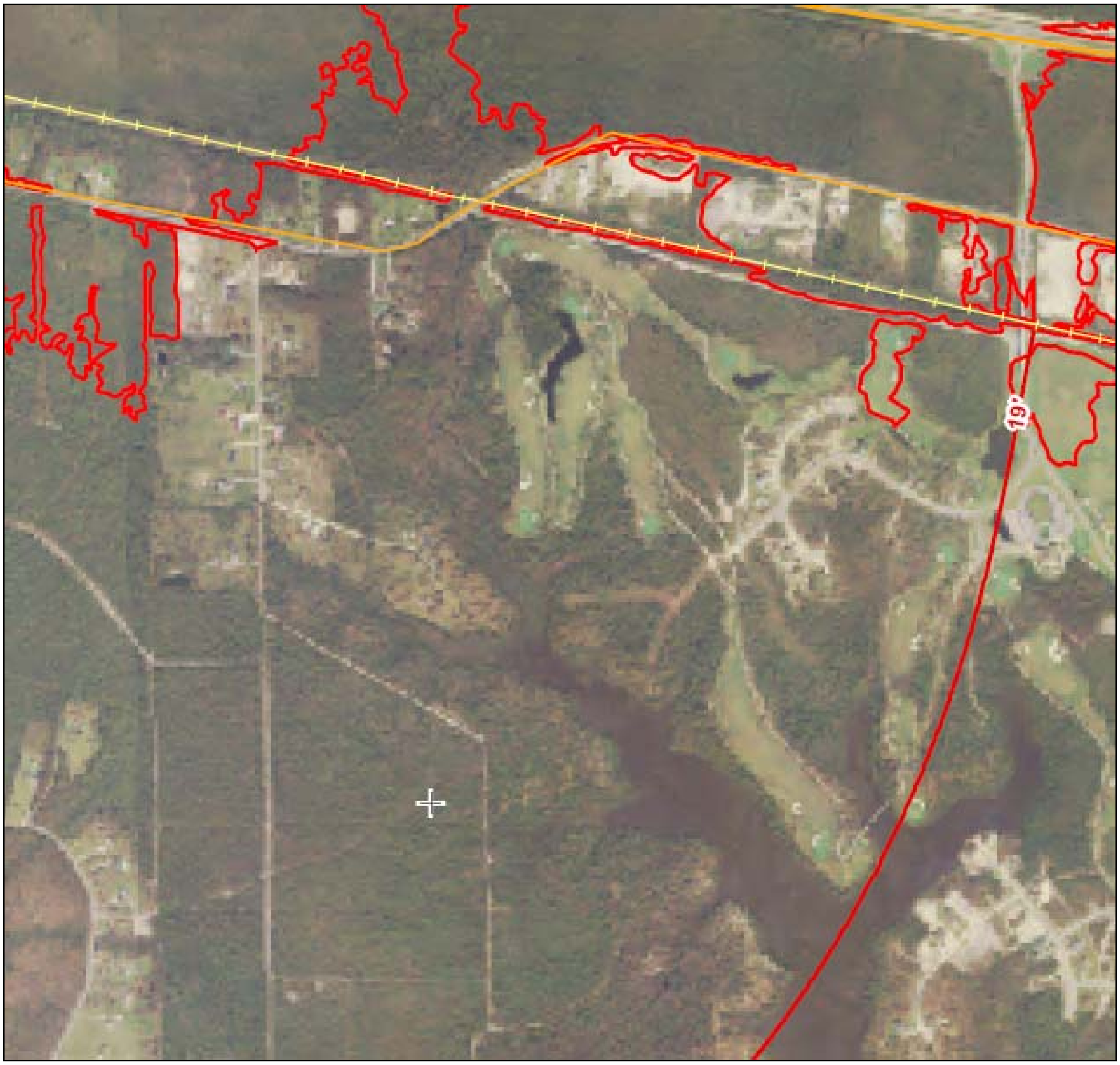


Figure 5. Old Spanish Trail Inundation Limits and 19-ft Contour

Additional data is provided in a report to FEMA by URS Group, Inc., titled "Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report," 16 January 2006, as well as in a report by FEMA titled "Draft Report, Hurricane Katrina Flood Frequency Analysis," dated September 2005. Results are summarized below.

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b) (Tables 1 and 2 and Figure 6). The flood frequency analysis only represents conditions at and near the gage.

Table 1.
Selected Tidal Gage Stations from NOAA

Station ID	Name	Latitude	Longitude	Begin Year	End Year
8729840	Pensacola, Pensacola Bay, FL	30.40 N	87.21 W	1924	2005
8735180	Dauphin Island, Mobile Bay, AL	30.25 N	88.08 W	1967	2005
8747766	Waveland, Mississippi Sound, MS	30.28 N	89.37 W	1979	2005
8761724	Grand Isle, East Point, LA	29.26 N	89.96 W	1972	2005

Table 2.
Selected Tidal Gages from USGS/USACE

Name	Latitude	Longitude	Begin Year	End Year
Back Bay Biloxi at Biloxi, MS	30.40 N	88.84 W	1882	1998
Pascagoula River at Pascagoula, MS	30.37 N	88.56 W	1940	1998

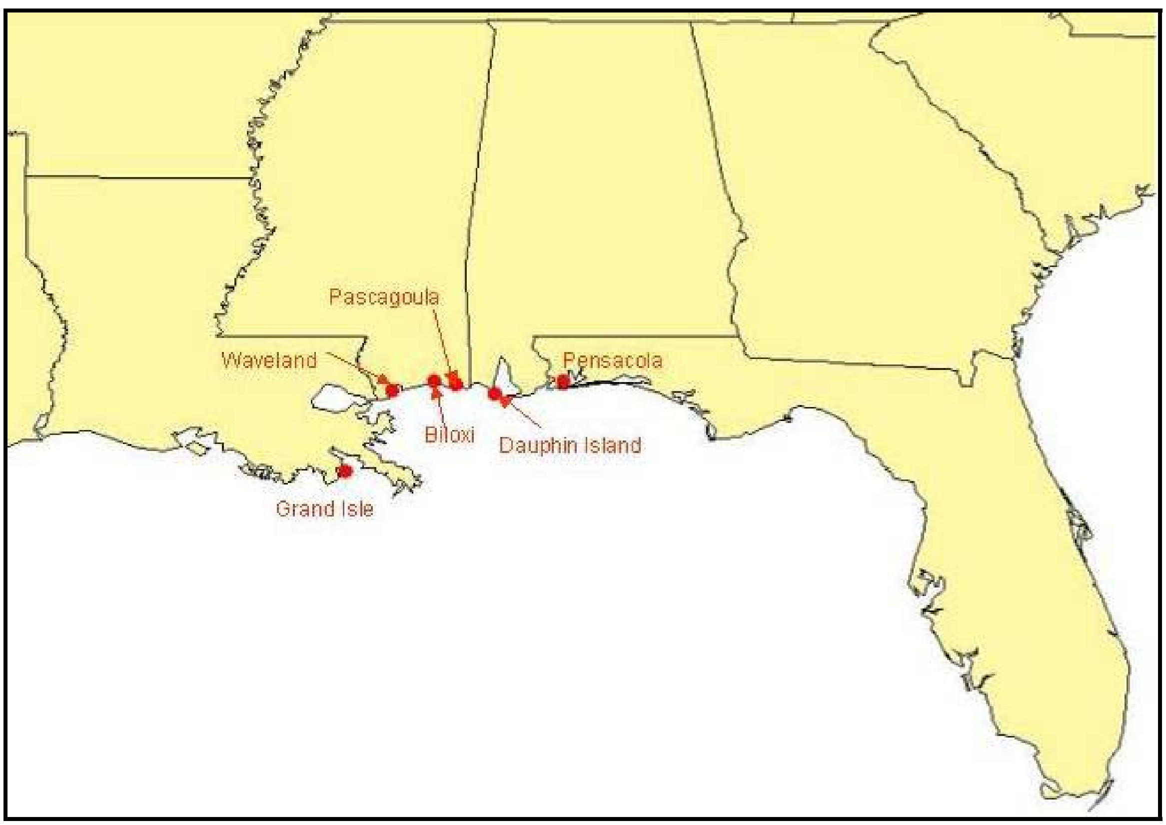


Figure 6. Tidal Gage Locations in the Area Impacted by Hurricane Katrina

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.
- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.
- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.
- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet.
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet.

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 3 As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 3.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations Location Katrina Elevation

Location	Katrina Elevation (Ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
DauphinIsland, AL	5.81	20
Pensacola, FL	6.07	50

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK15. This boring is located approximately 16000 feet south-southwest of the site. The data for boring JK15 indicates fine and medium sands with less than 4% fines to a depth of 115 feet. Boring JK15 is not believed to be necessarily indicative of the subsurface conditions at the site. Finer grained soils are believed to be present at least at some locations based on observations of surface soils.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives for improving the condition of Old Spanish Trail are listed below.

Alternative 1: Sediment Removal (2 ft)

This alternative is a short term alternative that would provide some immediate relief for the area and restore the stream to close to a pre-Katrina condition. Sediment would be removed from the channel for 1750 feet as shown in Figure 2, above. Some clearing would be required to provide access for removing the sediment. This alternative would remove approximately 1600 cubic yards (cy) of material and reduce flooding a minimal degree.

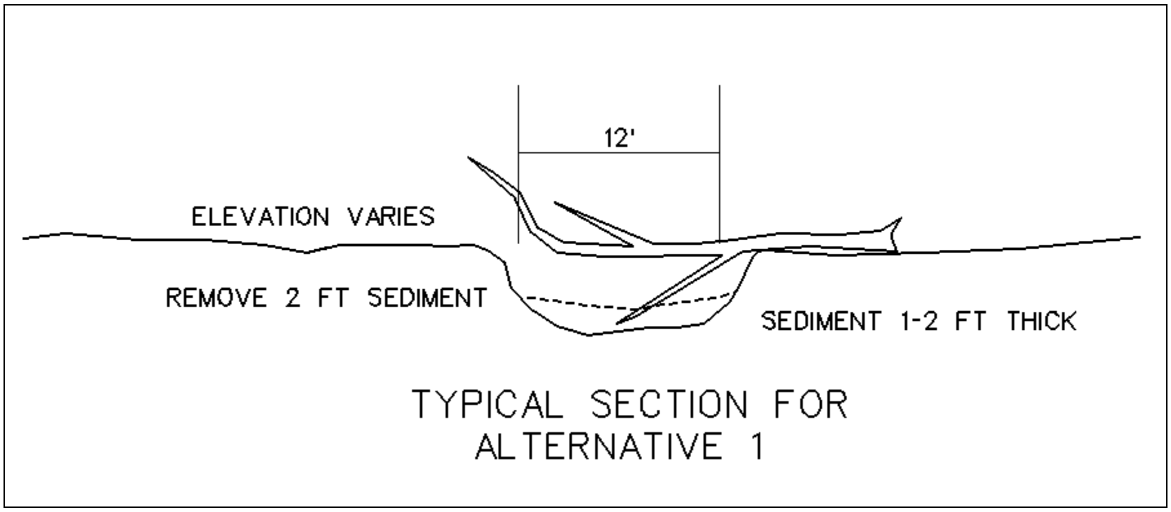


Figure 7. Old Spanish Trail Alternative 1

Alternative 2: Sediment Removal (1 ft)

This alternative is basically the same as Alternative 1 except that only 1 ft of sediment would be removed. No additional drawing is included. Reduction in flooding would be minimal.

Long Term Alternative: Channelization to Tidal Water

This alternative should be considered as part of a long term project. It would consist of channel work as in Alternative 1 with the addition of channel excavation from the end of the existing improved 1750-ft section of the stream to the tidal water, a distance of approximately 1000 additional feet. The new channel would have a bottom width of approximately 15 ft and sideslopes of 1 vertical to 2.5 horizontal. The plan would allow for more reduction in flooding below Old Spanish Trail but have less impact on flooding above the road. An addition to this alternative would include additional culvert capacity at the road.

Construction Procedures and Water Control Plan

Construction would be done by clearing a 30-ft wide area adjacent to the stream along one side and using back-hoe or other mechanical excavation equipment and dump trucks. Excavated material would be hauled out to a upland disposal area. Construction would be done in low flow months, when there is very little water in the channel. On the day of the inspection, 29 March 2006, there was very little water in the stream and no observable flow.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Gautier HSDR Graveline Bayou Site

Existing Conditions

Debris and sediment are blocking drainage in stream for approximately 6900 ft upstream of the mouth in the area shown on the figure below. The width of the stream and sediment deposition is approximately 50 ft wide. Shoaling is estimated to be approximately 3 ft deep.

Figure 8. Gautier HSDR - Graveline Bayou Site

Coastal/Hydraulics

Katrina inundation elevations near all four Mississippi Sound sites are estimated approximately at elevation 21 ft NGVD along the coast and somewhat less inland as shown below.

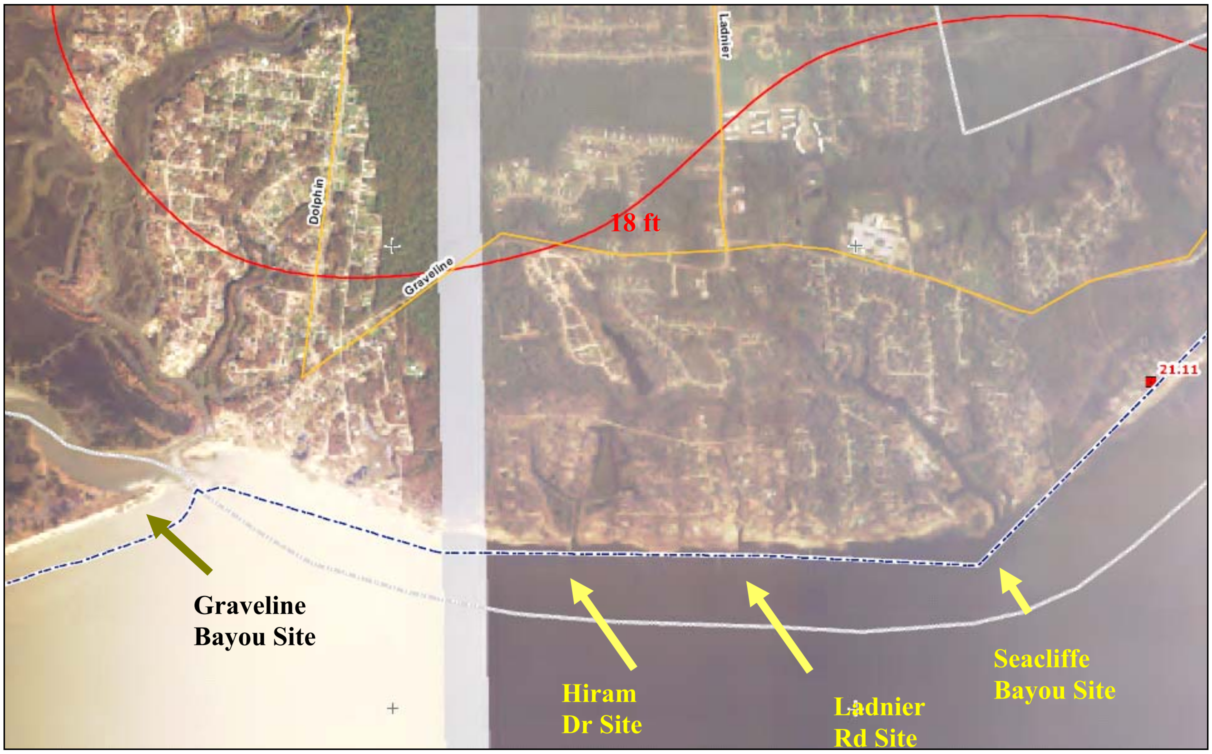


Figure 9. Gautier Flooding Elevations

Other storm data is presented in the Coastal/Hydraulics section of the Old Spanish Trail site.

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK15. This boring is located approximately 10000 feet southwest of the site. The data for boring JK15 indicates fine and medium sands with less than 4% fines to a depth of 115 feet. Boring JK15 is not believed to be necessarily indicative of the subsurface conditions at the site. Finer grained soils are believed to be present at least at some locations based on observations of surface soils.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives for improving the condition are listed below.

Alternative 1: Sediment Removal (3 ft)

This alternative consists of removal of sediment for approximately 6900 feet upstream of the mouth. Removal of some debris would be necessary to get to the sediment. The average width of the channel is approximately 50 ft and depth is an average of approximately 3 ft thick. Some parts of the stream are lined with bulkheads.

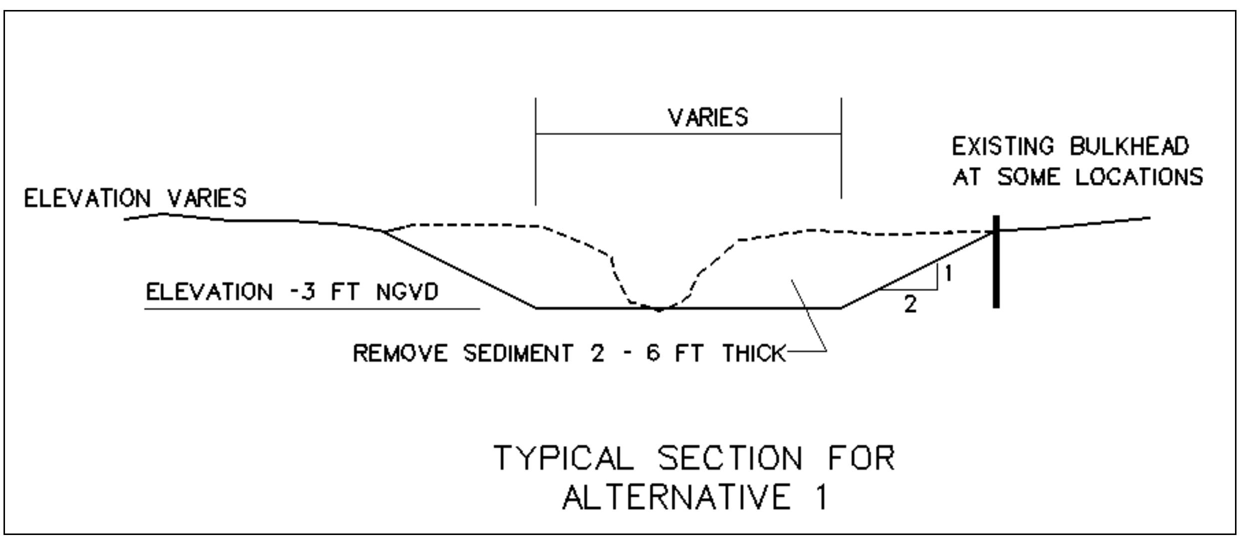


Figure 10. Graveline Bayou Alternative 1

The excavation would provide a moderate reduction in flooding on the lower end of the channel and would reduce flooding to a lesser degree above Lucina Cove, 2000 ft above the mouth. A longer term addition to this alternative could include a jetty at the mouth of the channel to help prevent sediment from entering the channel.

Alternative 2. Sediment Removal (1 ft)

This alternative consists of removing sediment in the channel for a depth of 1 ft only. Reductions in flooding would be minimal.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If a marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Maintenance will be the responsibility of Jackson County.

Gautier HSDR – Hiram Dr. Site

Existing Conditions

Debris and sediment are blocking drainage in stream for approximately 2640 ft upstream of the mouth in the area shown on the figure below. The width of the stream and sediment deposition is approximately 50 ft wide and depth is approximately 3 ft.

Figure 11. Gautier HSDR – Hiram Dr Site

Coastal/Hydraulics

Katrina inundation elevations near all four Mississippi Sound sites are estimated approximately at elevation 21 ft NGVD along the coast and somewhat less inland as shown below.

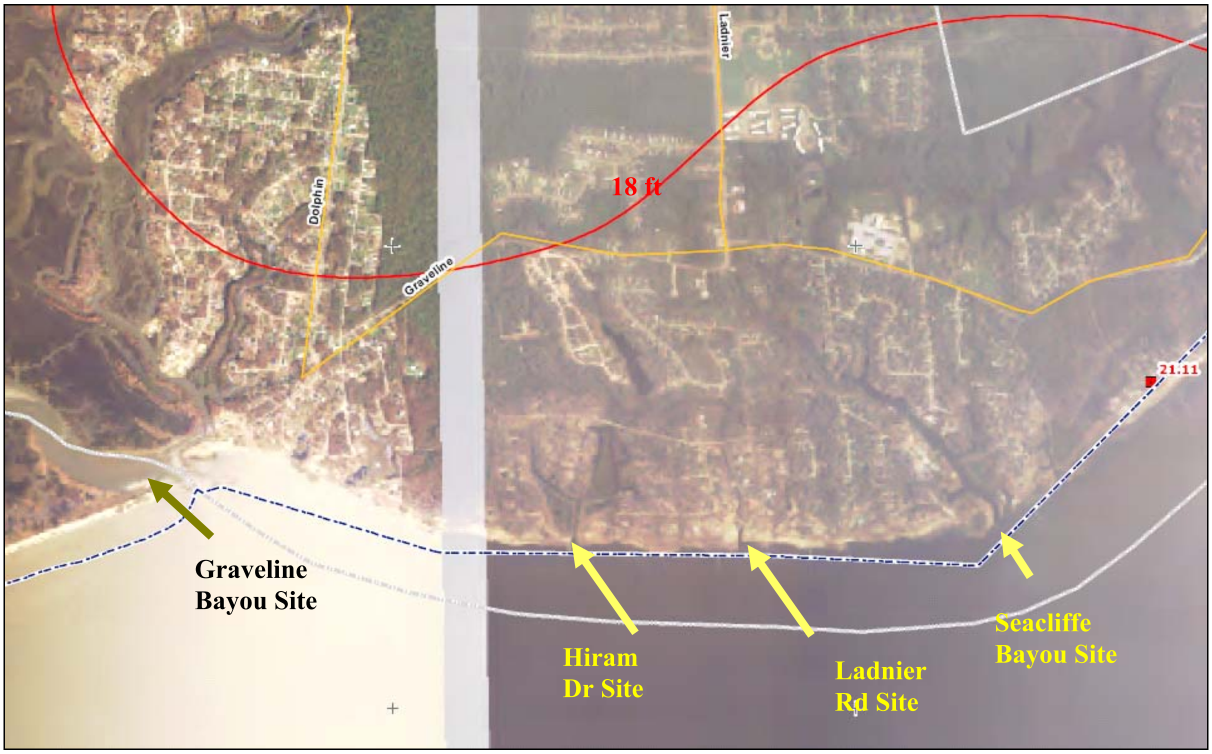


Figure 12. Gautier Flooding Elevations

Other storm data is presented in the Coastal/Hydraulics section of the Old Spanish Trail site.

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK15. This boring is located approximately 10000 feet southwest of the site. The data for boring JK15 indicates fine and medium sands with less than 4% fines to a depth of 115 feet. Boring JK15 is not believed to be necessarily indicative of the subsurface conditions at the site. Finer grained soils are believed to be present at least at some locations based on observations of surface soils.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives for improving the condition are listed below.

Alternative 1: Sediment Removal (3 ft)

This alternative consists of removal of sediment for approximately 2640 feet upstream of the mouth. Removal of some debris would be necessary to get to the sediment. The average width of the channel is approximately 50 ft and depth is an average of approximately 3 ft thick. Some parts of the stream are lined with bulkheads.

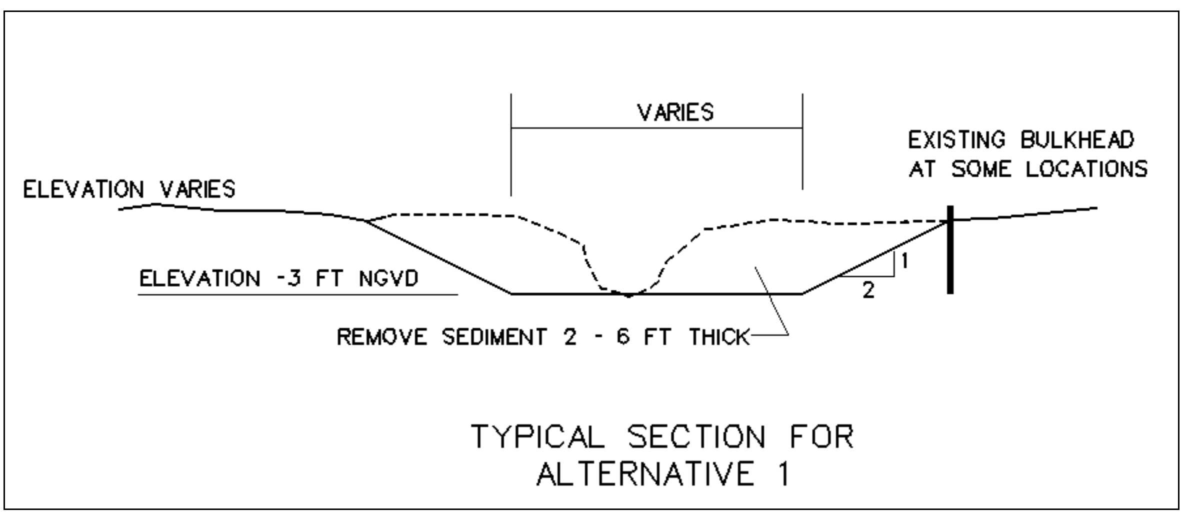


Figure 13. Hiram Dr. Alternative 1

The excavation would provide a moderate reduction in flooding on the lower end of the channel and would reduce flooding to a lesser degree above Hiram Drive, 1700 ft above the mouth. A longer term addition to this alternative could include a jetty at the mouth of the channel to help prevent sediment from entering the channel.

Alternative 2: Sediment Removal (1 ft)

This alternative consists of removing sediment in the channel for a depth of 1 ft only. Reduction in flooding would be minimal.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If a marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Maintenance will be the responsibility of Jackson County.

Gautier HSDR - Ladnier Road Site

Existing Conditions

Debris and sediment are blocking drainage in stream for approximately 1150 ft upstream of the mouth in the area shown on the figure below. The width of the stream and sediment deposition is approximately 40 ft wide.

Figure 13. Gautier HSDR - Ladnier Road Project Site

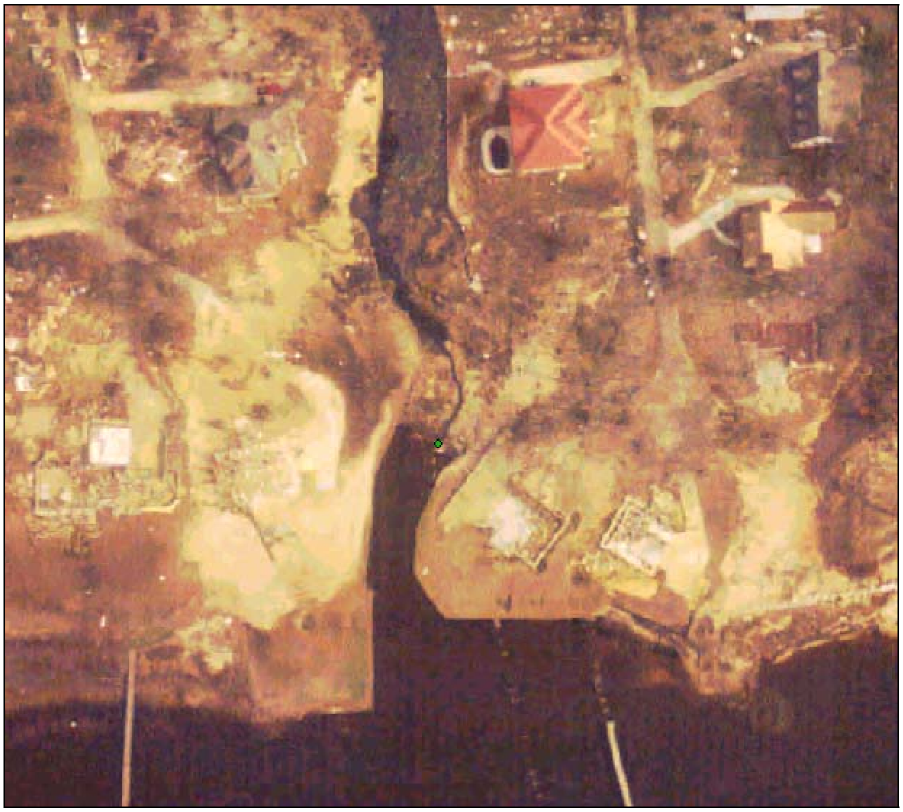


Figure 14. Gautier HSDR - Ladnier Road Site with Sediment Visible

Figure 15. Gautier HSDR - Ladnier Road Site Sedimentation

Coastal/Hydraulics

Katrina inundation elevations near all four Mississippi Sound sites are estimated approximately at elevation 21 ft NGVD along the coast and somewhat less inland as shown below.

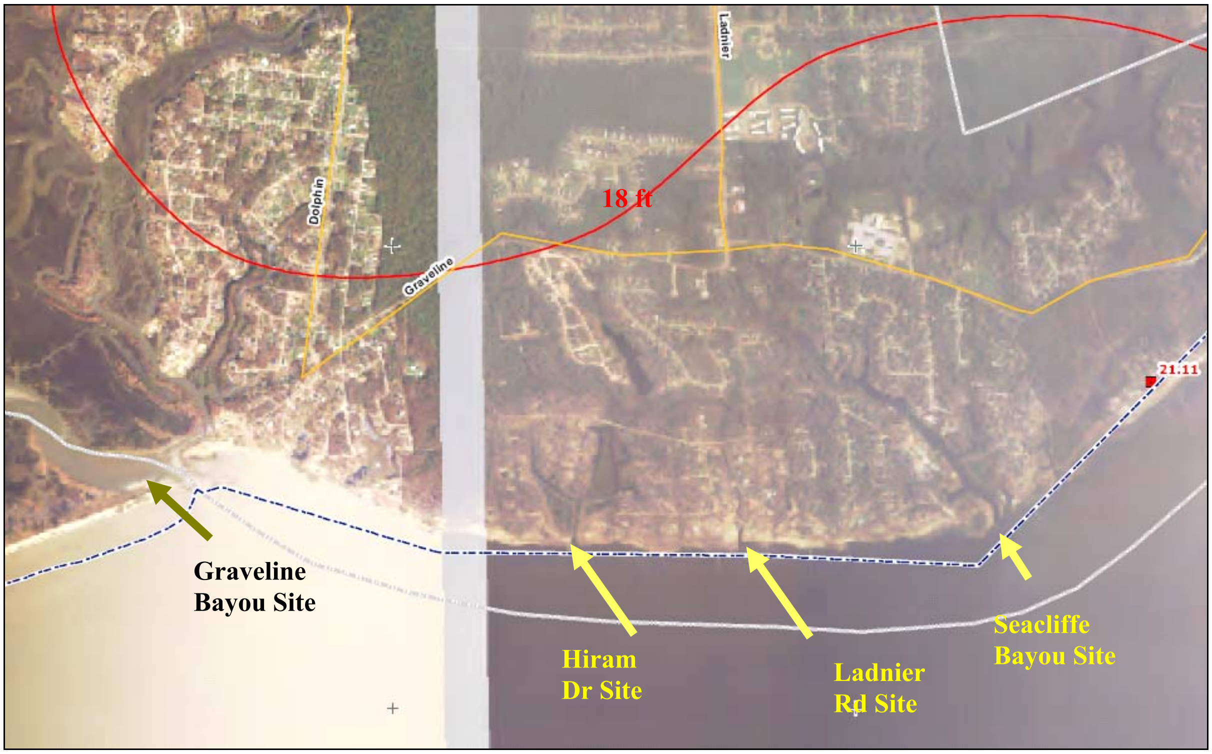


Figure 16. Gautier Flooding Elevations

Other storm data is presented in the Coastal/Hydraulics section of the Old Spanish Trail site.

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK15. This boring is located approximately 10000 feet southwest of the site. The data for boring JK15 indicates fine and medium sands with less than 4% fines to a depth of 115 feet. Boring JK15 is not believed to be necessarily indicative of the subsurface conditions at the site. Finer grained soils are believed to be present at least at some locations based on observations of surface soils.

HTRW

Site inspections are currently being conducted by the U.S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

Alternative 1: Sediment Removal (3 ft)

This alternative consists of removal of sediment for approximately 1150 feet upstream of the mouth. Removal of some debris would be necessary to get to the sediment. The average width of the channel is approximately 40 ft and depth is an average of approximately 3 ft thick. Some parts of the stream are lined with bulkheads.

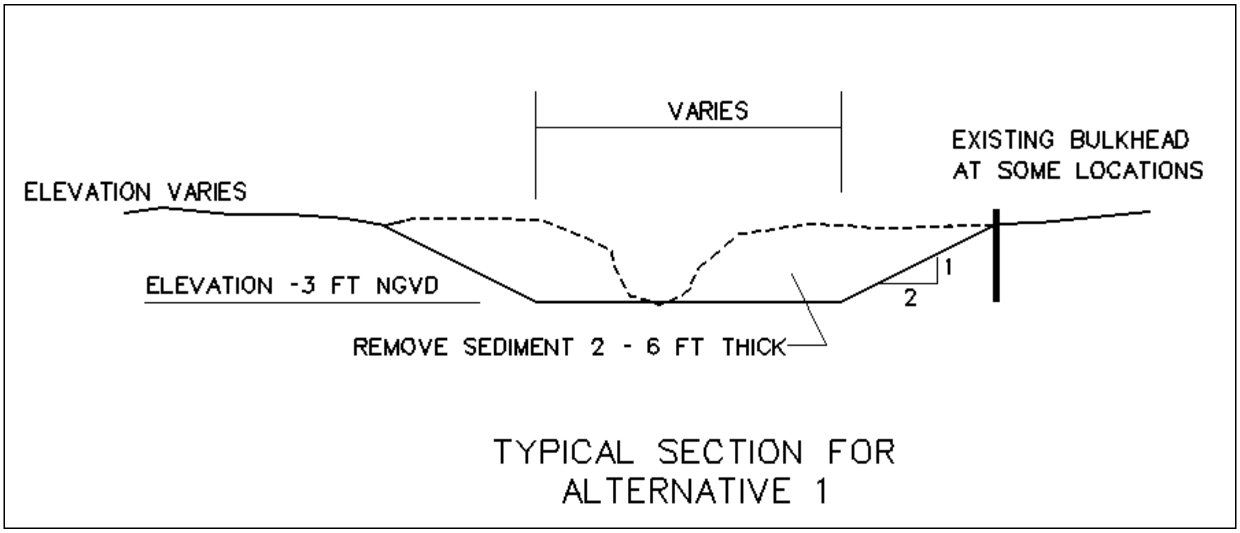


Figure 17. Ladnier Road Site Alternative 1

The excavation would provide a moderate reduction in flooding on the lower end of the channel and would reduce flooding to a lesser degree above Graveline Road, 3000 ft above the mouth. A longer term addition to this alternative could include a jetty at the mouth of the channel to help prevent sediment from entering the channel.

Alternative 2: Sediment Removal (1 ft)

This alternative consists of removing sediment in the channel for a depth of 1 ft only. Reductions in flooding would be minimal.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If marsh buggy equipment is used, water control would not be a problem. On the day of the inspection, 29 March 2006, there was very little water flowing in the stream and no observable flow.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Maintenance will be the responsibility of Jackson County.

Gautier HSDR – Seacliffe Bayou Site

Existing Conditions

Debris and sediment are blocking drainage in stream for approximately 2440 ft upstream of the mouth in the area shown on the figure below. The width of the stream and sediment deposition is approximately 50 ft wide and depth is approximately 3 ft.

Figure 18. Gautier HSDR - Seacliffe Bayou Site

Coastal/Hydraulics

Katrina inundation elevations near all four Mississippi Sound sites are estimated approximately at elevation 21 ft NGVD along the coast and somewhat less inland as shown below.

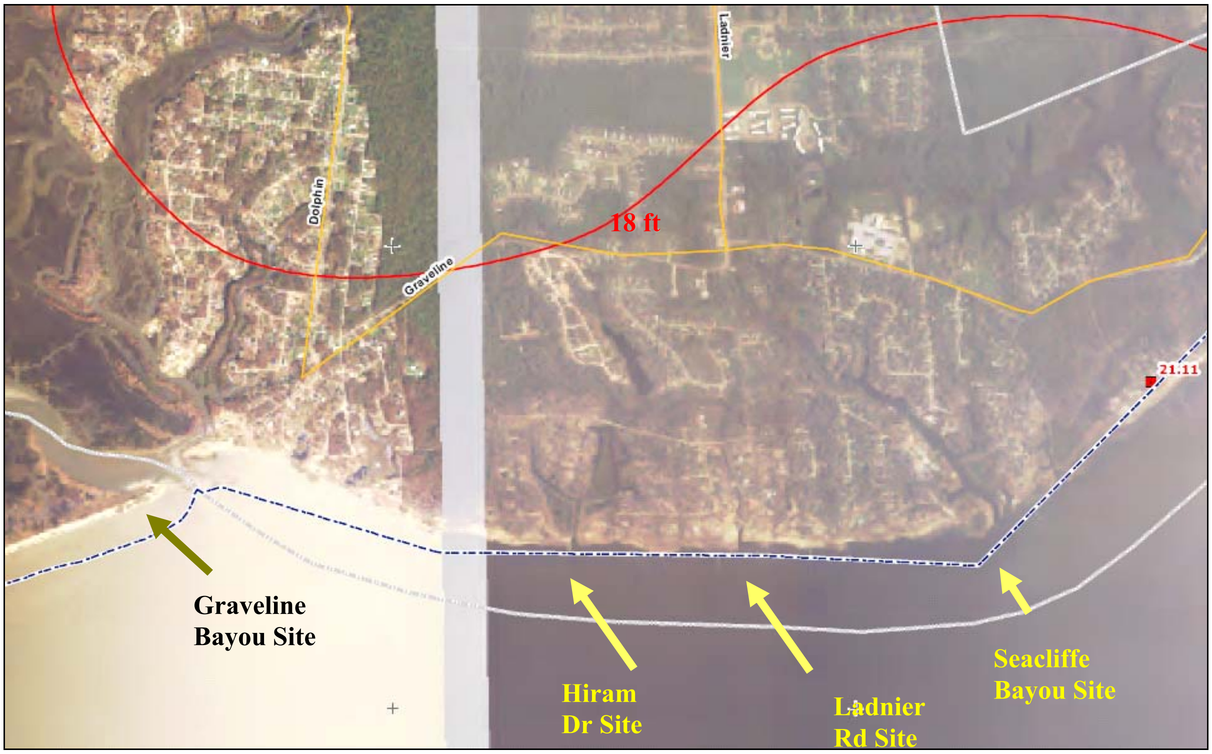


Figure 19. Gautier Flooding Elevations

Other storm data is presented in the Coastal/Hydraulics section of the Old Spanish Trail site.

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK15. This boring is located approximately 10000 feet southwest of the site. The data for boring JK15 indicates fine and medium sands with less than 4% fines to a depth of 115 feet. Boring JK15 is not believed to be necessarily indicative of the subsurface conditions at the site. Finer grained soils are believed to be present at least at some locations based on observations of surface soils.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

Alternative 1: Sediment Removal (3 ft)

This alternative consists of removal of sediment for approximately 2440 feet upstream of the mouth. Removal of some debris would be necessary to get to the sediment. The average width of the channel is approximately 50 ft and depth is an average of approximately 3 ft thick. Some parts of the stream are lined with bulkheads.

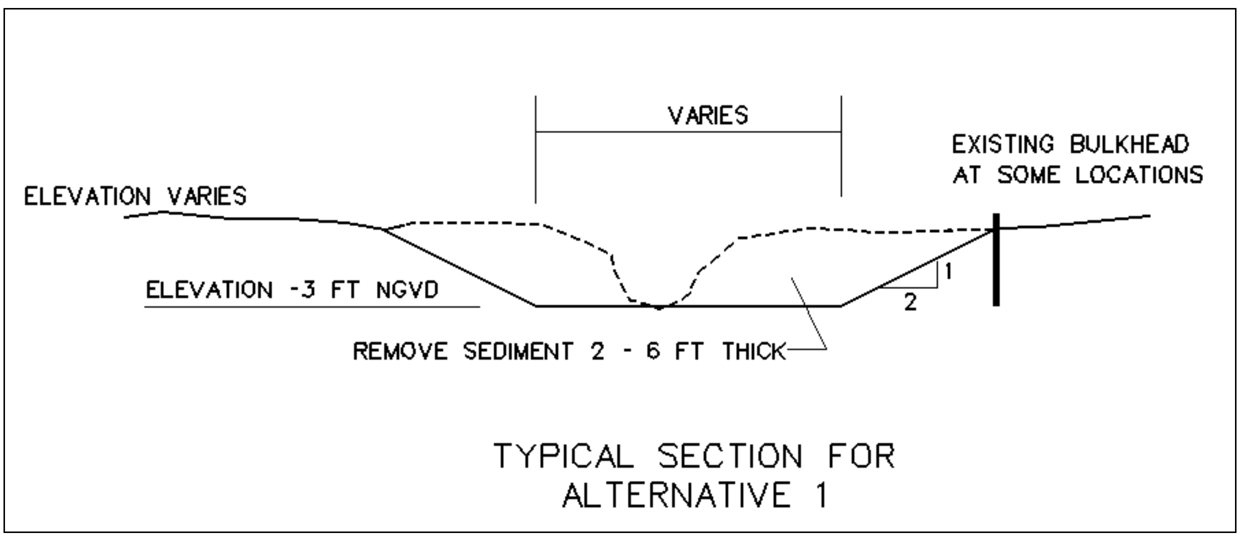


Figure 20. Seacliffe Bayou Alternative 1

The excavation would provide a moderate reduction in flooding on the lower end of the channel and would reduce flooding to a lesser degree above Seacliffe Dr., 2400 ft above the mouth. A longer term addition to this alternative could include a jetty at the mouth of the channel to help prevent sediment from entering the channel.

Alternative 2: Sediment Removal (1 ft)

This alternative consists of removing sediment in the channel for a depth of 1 ft only. Reductions in flooding would be minimal.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Maintenance will be the responsibility of Jackson County.

Cost Estimates

Cost estimates are presented for the following alternative combinations.

Recommended Alternative: This alternative consists of 2 foot of excavation at Old Spanish Trail and three foot excavation at each of the following sites: Graveline Bayou, Hiram Drive, Ladnier Road, and Seacliff.

Maintenance estimates are presented for each of the sites separately.

Combination 1-foot Alternative: This alternative consists of 1 foot of excavation at the following sites: Old Spanish Trail, Graveline Bayou, Hiram Drive, Ladnier Road, and Seacliff

Maintenance estimates are presented for each of the sites separately

Table 4
Gautier HSDR – Recommended Plan – Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Recommended Plan***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	gautier5-12.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Recommended Plan					
Mobilization, Preparatory Work, Demobilization		1	job	allow	250,000
excavation (2 Ft.)		1,600	cy	30.00	48,000
excavation (3. Ft.)		71,700	cy	20.00	1,434,000
clear/grub		1	ac	5000.00	5,000
grass		1	ac	6000.00	6,000
Misc. Site Items		1	ls	allow	320,000
	Total Direct Construction Cost				$2,063,000
	Indirect Cost	@		15%	309,450
					2,372,450
	Profit	@		9%	213,521
					2,585,971
	Bond	@		1.5%	38,790
09 Account, Channels & Canals		Current Contract Cost, Oct 06			$2,624,760
	01 Account, Lands & Damage (PCA)			LS	375,000
					2,999,760
	30 Account, Plan, Engr.& Design			8%	239,981
					3,239,741
	31 Account, Constr. Management			6%	194,384
					3,434,125
	CONTINGENCY			20%	611,825
					4,045,950
					$4,046,950
					rounded
	TOTAL PROJECT COST, FY-07				**$4,050,000**

Alternative 1 – Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 5.
Old Spanish Trail - Alternative 1 – Maintenance Cost Estimate.

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	22-Apr-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Old Spanish Trail Removal 2 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier4-22r.xls		

DESCRIPTION			Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Old Spanish Trail Removal 2 Ft.						
Mobilization, Preparatory Work, Demobilization			1	job	allow	50,000
excavation			1,600	cy	30.00	48,000
clear/grub			1	ac	5000.00	5,000
grass			1	ac	6000.00	6,000
Misc. Site Items			1	ls	allow	20,000
	Total Direct Construction Cost					$129,000
	Indirect Cost	@			**15%**	19,350
						148,350
	Profit	@			**9%**	13,352
						161,702
	Bond	@			**1.5%**	2,426
		Current Contract Cost, Oct 06				$164,127
	01 Account, Lands & Damage (PCA)				**LS**	0
						164,127
	30 Account, Plan, Engr.& Design				**10%**	16,413
						180,540
	31 Account, Constr. Management				**6%**	10,832
						191,372
	CONTINGENCY				**20%**	38,274
						229,647
	ESCALATION, FY-07				**1%**	2,296
						$231,943
						rounded
	TOTAL PROJECT COST, FY-07					**$230,000**

Alternative 1– Maintenance Cost Estimate

It is estimated that maintenance will be required every 25 years at the cost shown below.

Table 6.
Gautier HSDR - Graveline Bayou – Alternative 1 - Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	9-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Graveline Bayou Removal 3 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier5-9.xls		

D E S C R I P T I O N		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Graveline Bayou Removal 3 Ft.					
Mobilization, Preparatory Work, Demobilization		1	job	allow	25,000
excavation		33,800	cy	20.00	676,000
	Total Direct Construction Cost				$701,000
	Indirect Cost	@		**15%**	105,150
					806,150
	Profit	@		**9%**	72,554
					878,704
	Bond	@		**1.5%**	13,181
		Current Contract Cost, Oct 06			$891,884
	01 Account, Lands & Damage (PCA)			**LS**	0
					891,884
	30 Account, Plan, Engr.& Design			**10%**	89,188
					981,072
	31 Account, Constr. Management			**6%**	58,864
					1,039,937
	CONTINGENCY			**20%**	207,987
					1,247,924
	ESCALATION, FY-07			**1%**	12,479
					$1,260,403
					rounded
	TOTAL PROJECT COST, FY-07				**$1,260,000**

Table 7.
Gautier HSDR – Hiram Dr. Site – Alternative 1 – Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	5-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Hiram Drive Removal 3 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	gautier5-5.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Hiram Drive Removal 3 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	14,700	cy	20.00	294,000
Misc. Site Items	1	ls	allow	75,000
Total Direct Construction Cost				$419,000
Indirect Cost @			**15%**	62,850
				481,850
Profit @			**9%**	43,367
				525,217
Bond @			**1.5%**	7,878
Current Contract Cost, Oct 06				$533,095
01 Account, Lands & Damage (PCA)			**LS**	168,750
				701,845
30 Account, Plan, Engr.& Design			**8%**	56,148
				757,992
31 Account, Constr. Management			**6%**	45,480
				803,472
CONTINGENCY			**20%**	126,944
				930,416
ESCALATION, FY-07			**1%**	9,304
				$939,720 rounded
TOTAL PROJECT COST, FY-07				**$940,000**

Table 8.
Gautier HSDR – Ladnier Rd. Site – Alternative 1 – Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	5-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Ladiner Road Removal 3 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	gautier5-5.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Ladiner Road Removal 3 Ft.					
Mobilization, Preparatory Work, Demobilization		1	job	allow	50,000
excavation		5,100	cy	20.00	102,000
Misc. Site Items		1	ls	allow	75,000
	Total Direct Construction Cost				$227,000
	Indirect Cost	@		**15%**	34,050
					261,050
	Profit	@		**9%**	23,495
					284,545
	Bond	@		**1.5%**	4,268
		Current Contract Cost, Oct 06			$288,813
	01 Account, Lands & Damage (PCA)			**LS**	131,250
					420,063
	30 Account, Plan, Engr.& Design			**8%**	33,605
					453,668
	31 Account, Constr. Management			**6%**	27,220
					480,888
	CONTINGENCY			**20%**	69,928
					550,815
	ESCALATION, FY-07			**1%**	5,508
					$556,323 rounded
	TOTAL PROJECT COST, FY-07				**$560,000**

Table 9.
Gautier HSDR - Seacliffe Bayou Site – Alternative 1 – Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	5-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Seacliffe Removal 3 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	gautier5-5.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seacliffe Removal 3 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	13,600	cy	20.00	272,000
Misc. Site Items	1	ls	allow	75,000
	Total Direct Construction Cost			$397,000
	Indirect Cost	@	**15%**	59,550
				456,550
	Profit	@	**9%**	41,090
				497,640
	Bond	@	**1.5%**	7,465
	Current Contract Cost, Oct 06			$505,104
	01 Account, Lands & Damage (PCA)		**LS**	168,750
				673,854
	30 Account, Plan, Engr.& Design		**8%**	53,908
				727,762
	31 Account, Constr. Management		**6%**	43,666
				771,428
	CONTINGENCY		**20%**	120,536
				891,964
	ESCALATION, FY-07		**1%**	8,920
				$900,883
				rounded
	TOTAL PROJECT COST, FY-07			**$900,000**

Table 10.
Gautier HSDR – Combination 1 Ft. Plan – Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Combination 1 Ft. Plan***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	gautier5-12.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Combination 1 Ft. Plan					
Mobilization, Preparatory Work, Demobilization		1	job	allow	250,000
excavation		17,700	cy	20.00	354,000
excavation		7,000	cy	30.00	210,000
clear/grub		1	ac	5000.00	5,000
grass		1	ac	6000.00	6,000
Misc. Site Items		1	ls	allow	225,000
	Total Direct Construction Cost				$1,050,000
	Indirect Cost	@		15%	157,500
					1,207,500
	Profit	@		9%	108,675
					1,316,175
	Bond	@		1.5%	19,743
09 Account, Channels & Canals		Current Contract Cost, Oct 06			$1,335,918
	01 Account, Lands & Damage (PCA)			LS	375,000
					1,710,918
	30 Account, Plan, Engr.& Design			8%	136,873
					1,847,791
	31 Account, Constr. Management			6%	110,867
					1,958,658
	CONTINGENCY			20%	316,732
					2,275,390
					$2,275,390
					rounded
	TOTAL PROJECT COST, FY-07				**$2,280,000**

Alternative 2 – Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 11.
Old Spanish Trail - Alternative 2 – Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	22-Apr-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Old Spanish Trail Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier4-22r.xls		

D E S C R I P T I O N	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Old Spanish Trail Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	800	cy	30.00	24,000
clear/grub	1	ac	5000.00	5,000
grass	1	ac	6000.00	6,000
Misc. Site Items	1	ls	allow	20,000
Total Direct Construction Cost				$105,000
Indirect Cost	@		**15%**	15,750
				120,750
Profit	@		**9%**	10,868
				131,618
Bond	@		**1.5%**	1,974
Current Contract Cost, Oct 06				$133,592
01 Account, Lands & Damage (PCA)			**LS**	0
				133,592
30 Account, Plan, Engr.& Design			**10%**	13,359
				146,951
31 Account, Constr. Management			**6%**	8,817
				155,768
CONTINGENCY			**20%**	31,154
				186,922
ESCALATION, FY-07			**1%**	1,869
				$188,791 rounded
TOTAL PROJECT COST, FY-07				**$190,000**

Alternative 2– Maintenance Cost Estimate

It is estimated that maintenance will be required every 25 years at the cost shown below.

Table 12.
Gautier HSDR - Graveline Bayou – Alternative 2 - Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	9-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Graveline Bayou Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier5-9.xls		

DESCRIPTION			Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Graveline Bayou Removal 1 Ft.						
Mobilization, Preparatory Work, Demobilization			1	job	allow	25,000
excavation			12,800	cy	20.00	256,000
	Total Direct Construction Cost					$281,000
	Indirect Cost	@			**15%**	42,150
						323,150
	Profit	@			**9%**	29,084
						352,234
	Bond	@			**1.5%**	5,284
		Current Contract Cost, Oct 06				$357,517
	01 Account, Lands & Damage (PCA)				**LS**	0
						357,517
	30 Account, Plan, Engr.& Design				**10%**	35,752
						393,269
	31 Account, Constr. Management				**6%**	23,596
						416,865
	CONTINGENCY				**20%**	83,373
						500,238
	ESCALATION, FY-07				**1%**	5,002
						$505,240
						rounded
	TOTAL PROJECT COST, FY-07					**$510,000**

Alternative 2– Maintenance Cost Estimate

It is estimated that maintenance will be 900 cy per year at the cost shown below.

Table 13.
Gautier HSDR - Hiram Dr. Site – Alternative 2 - Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	9-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Hiram Drive Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier5-9.xls		

D E S C R I P T I O N		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Hiram Drive Removal 1 Ft.					
Mobilization, Preparatory Work, Demobilization		1	job	allow	25,000
excavation		4,900	cy	25.00	122,500
	Total Direct Construction Cost				$147,500
	Indirect Cost	@		**15%**	22,125
					169,625
	Profit	@		**9%**	15,266
					184,891
	Bond	@		**1.5%**	2,773
		Current Contract Cost, Oct 06			$187,665
	01 Account, Lands & Damage (PCA)			**LS**	0
					187,665
	30 Account, Plan, Engr.& Design			**10%**	18,766
					206,431
	31 Account, Constr. Management			**6%**	12,386
					218,817
	CONTINGENCY			**20%**	43,763
					262,580
	ESCALATION, FY-07			**1%**	2,626
					$265,206 rounded
		TOTAL PROJECT COST, FY-07			**$270,000**

Alternative 2– Maintenance Cost Estimate

It is estimated that maintenance will be required every 25 years at the cost shown below.

Table 14.
Gautier HSDR – Ladnier Rd. Site – Alternative 2 - Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	9-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Ladiner Road Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier5-9.xls		

DESCRIPTION		Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Ladiner Road Removal 1 Ft.					
Mobilization, Preparatory Work, Demobilization		1	job	allow	25,000
excavation		1,700	cy	30.00	51,000
	Total Direct Construction Cost				$76,000
	Indirect Cost	@		**15%**	11,400
					87,400
	Profit	@		**9%**	7,866
					95,266
	Bond	@		**1.5%**	1,429
		Current Contract Cost, Oct 06			$96,695
	01 Account, Lands & Damage (PCA)			**LS**	0
					96,695
	30 Account, Plan, Engr.& Design			**10%**	9,669
					106,364
	31 Account, Constr. Management			**6%**	6,382
					112,746
	CONTINGENCY			**20%**	22,549
					135,296
	ESCALATION, FY-07			**1%**	1,353
					$136,649
					rounded
		TOTAL PROJECT COST, FY-07			**$140,000**

Alternative 2– Maintenance Cost Estimate

It is estimated that maintenance will be required every 25 years at the cost shown below.

Table 15.
Gautier HSDR – Seacliffe Bayou Site – Alternative 2 - Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Gautier	ITEM NO.	1	DATE	9-May-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Seacliffe Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m gautier5-9.xls		

D E S C R I P T I O N	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seacliffe Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	50,000
excavation	4,500	cy	20.00	90,000
Total Direct Construction Cost				$140,000
Indirect Cost	@		**15%**	21,000
				161,000
Profit	@		**9%**	14,490
				175,490
Bond	@		**1.5%**	2,632
Current Contract Cost, Oct 06				$178,122
01 Account, Lands & Damage (PCA)			**LS**	0
				178,122
30 Account, Plan, Engr.& Design			**10%**	17,812
				195,935
31 Account, Constr. Management			**6%**	11,756
				207,691
CONTINGENCY			**20%**	41,538
				249,229
ESCALATION, FY-07			**1%**	2,492
				$251,721
				rounded
TOTAL PROJECT COST, FY-07				**$250,000**

Table 16.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report, FEMA (URS Group, Inc.), 16 January 2006.

Draft Report, Hurricane Katrina Flood Frequency Analysis, FEMA, September 2005.

NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b)

PASCAGOULA BEACH BOULEVARD

General

The purpose of this document is to provide engineering information for planning and design analysis for Hurricane Katrina storm damage repair, drainage channel erosion protection, and environmental enhancement in Pascagoula, Jackson County, Mississippi.

Location

The study shoreline areas are located in Jackson County, the eastern-most coastal county in Mississippi. The site is located on the western half of the Pascagoula waterfront on Mississippi Sound. More than 28 million tons of cargo pass annually through the Port of Pascagoula, the state's largest port. Pascagoula is 44 miles by road west of Mobile, Alabama, and 110 miles east of New Orleans, Louisiana.

Figure 1. Location Map Showing Path of Hurricane Katrina

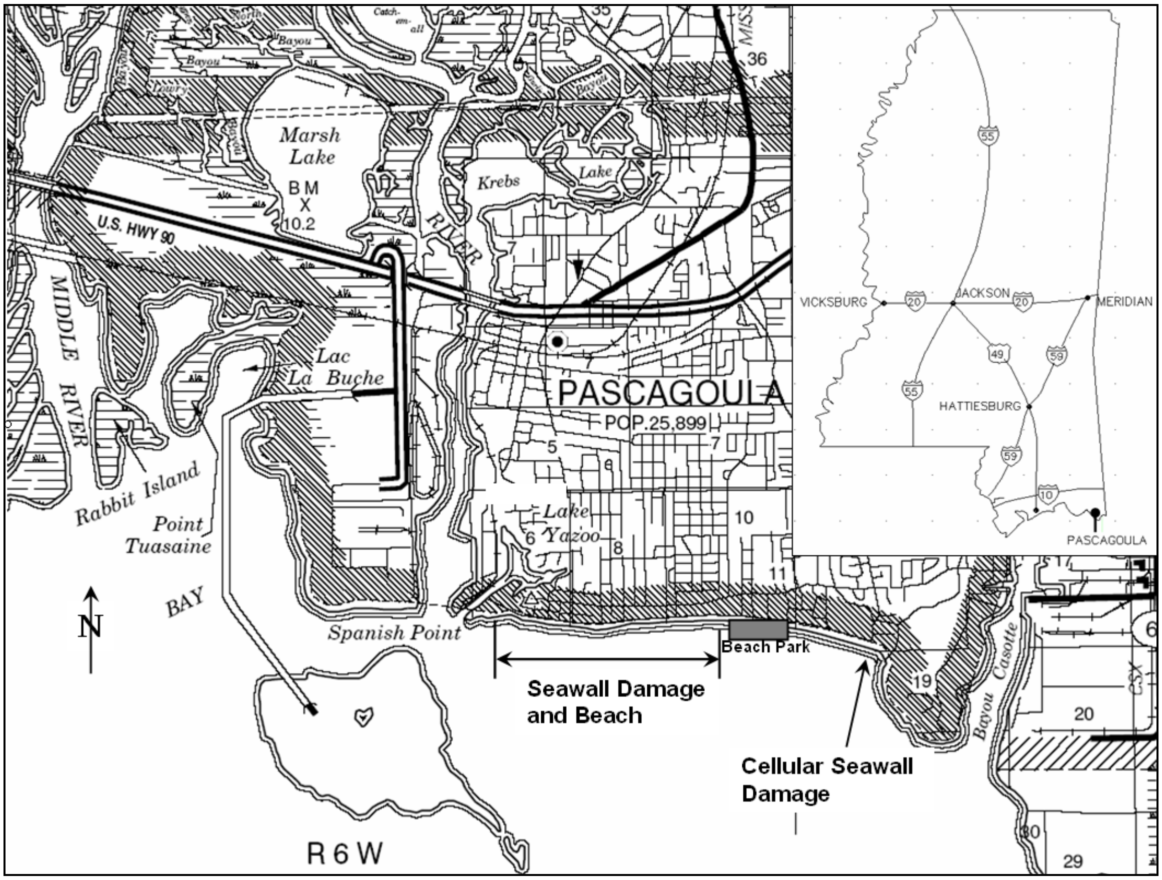


Figure 2. Area Map

The site location is shown on Figures 2, 3, and 4. The Pascagoula Shoreline between Spanish Point and Bayou Chico is fronted by Beach Boulevard, which is protected by a concrete seawall. The project site is primarily seaward of Beach Boulevard between Spanish Point and Beach Park, and about 50 feet of the shoreline at the east end of Beach Boulevard.

Existing Conditions

The Pascagoula waterfront extends from Spanish Point, at the mouth of the Pascagoula River (drainage area 9,498 sq. miles) east to Bayou Cassotte. The project limits generally coincide with the western half of the waterfront, from Spanish Point to Beach Park.

The majority of the shoreline there is protected by a concave seawall constructed in 1929 to protect Beach Boulevard (Figure 5). The original seawall was approximately 12,500 feet long. Electrical conduits for street lighting are housed in a concrete curb placed adjacent to and landward of a low concrete railing atop the seawall. The conduit housing was broken and severed in numerous locations during Hurricane Katrina. The seawall foundation details are presently unknown, but it is believed to be founded variously on the shoreline bed, and/or upon irregularly spaced pile, and/or upon debris placed historically to stem beach erosion and storm damage. The seawall crest elevation varies somewhat and is slightly lower than the centerline of Beach Boulevard, the elevation of which varies between 4.1 and 6.2 feet NGVD[6], which is on the order of 10 to 15 feet below the estimated maximum Hurricane Katrina surge heights in the area as depicted in Figure 3. Land use

[6] Assumed datum, information received is unclear as to the profile datum.

north of the roadway in this area is primarily residential, with at grade elevations generally between elevations 8 and 14 feet NGVD.

The seawall is penetrated in fourteen locations by circular concrete drainage culverts as shown in Figure 6. The culverts are held in place by “T” shaped concrete monoliths. Chevron shaped concrete flow deflectors are deployed off the southeast corner of each monolith to help prevent the culverts from filling with sediments. The culverts and monoliths appear to be in good condition, as do most of the flow deflectors, though some exhibit a good deal of wear. No direct evidence of hurricane damage was observed on the culvert outfall structures.

The seawall is also penetrated in two locations by open drainage channels. One channel is west of 11th Street (see Figures 7 through 9), the other adjacent to the west end of Beach Park. The channel drains relatively low-lying areas west and north of Beach Park. Upstream of the Beach Boulevard bridge, this channel consists of concrete panel walls with a concrete cap and a natural streambed. Each pile panel is approximately 30 feet long, one foot thick, and of unknown height (possibly 12 to 15 feet). The channel is “S” shaped between the Beach Boulevard bridge and the Washington Street bridge and approximately 330 feet long. Extreme storm surge can be preferentially conveyed, and evacuated, through this channel. All concrete panels exhibit rotational and/or translational failure with localized erosion behind the panels, and many panel caps exhibit impact damage. While the pre-storm condition of the channel walls and bed are not presently known, vegetation lines, terrestrial erosion patterns, and signs of haphazard fill operations observed during post-storm field inspections suggest that the great majority of substantial wall failures are associated with Hurricane Katrina storm surge. Erosion of fill material behind the walls is ongoing and exacerbated due to wall displacement. Downstream of the Beach Boulevard Bridge, the channel is confined by concrete pile training walls extending out to sea. About 60 feet of the concrete pile cap upon the right (looking to sea) training wall appears to have been damaged during Hurricane Katrina.

A vertically-walled, cellular seawall extends east of Beach Park to the east end of Beach Boulevard (Figure 10). This seawall was apparently built in the 1970s and is approximately 900 feet long. The seawall protects the west end of Beach Boulevard and a portion of unarmored public right-of-way beyond it. This portion of the seawall is approximately 8 feet wide and also functions as a walkway It appears to have been backfilled with soil and capped with approximately 6 feet by 8 feet irregularly-shaped architectural concrete panels. Seven adjacent panels were displaced and broken by hurricane surge over a distance of about 60 feet in the region of the unarmored shoreline. The open cells have since served as an informal repository for miscellaneous storm debris. The seawall otherwise appears to be in good condition.

The Beach Boulevard roadway surface reportedly has been historically subject to periodic, localized subsidence associated with persistent wave activity and aggravated by wind and rain storm events due to migration of fill to Mississippi Sound. Fill migration paths are interpreted to be through failed construction joint seals, through buried stormwater discharge conduits, and/or beneath exposed portions of the seawall footing. Loss of fill through these pathways may be exacerbated by storm events, but the constant ebb and flow of the tide and the normal wave regime can also recruit fines past the seawall. Several hundreds of thousands of dollars have been spent trying to halt or reduce the flow of fines, primarily by injecting grout behind the seawall and periodic maintenance of construction joint seals. No evidence of damages directly attributable to loss of fines under or through the seawall due to Hurricane Katrina were observed during field inspections in April 2006. It appears then that the grouting and sealing maintenance program has been somewhat successful, though it is probable that fines migration pathways still exist. Culverts were not inspected to reveal whether fines were migrating through them. Periodic grouting will probably continue to be necessary periodically.

Where construction joints had been sealed with caulk, with little exception the caulk exhibited tears, voids, or missing sealant. In those few instances where joints had been sealed by the application of rubber strips pressed into the joint gap, the seals appeared to be in good condition. There were also numerous locations of impact damage on the seawall face, with exposed and corroding reinforcement steel at some of these locations (Figure 11). The majorities of impact damages appeared to be recent and were assumed to be caused by Hurricane Katrina. In some instances, due to the high rate of deterioration of rebar in marine environments, it was difficult to tell whether observed impact damages were caused by Hurricane Katrina, or had existed prior to it. Because the seawall appears to have been reasonably well maintained over the years, it is assumed that most of these impact damages were related to Hurricane Katrina. Nearly every seawall panel exhibited longitudinal cracking, and in some instances vertical cracking. Cracks, unlike most impact damages, can form in the absence of a fierce wave environment, and it is impossible to say whether hurricane conditions caused all of the cracks observed. However, there is evidence of programmatic pre-storm crack repair, and given the attention to maintenance given the seawall, it is assumed that many of these are attributable to immediate or latent effects of recent hurricanes.

Construction joints are a potential roadbed fines migration pathway and all construction joints should be sealed, at a minimum, along their entire length to at least the level of mean low water. Likewise, where other cracks on the seawall face are on the order of ¼" wide or wider, they should be cleaned and sealed in the most appropriate manner. Impact damages should be repaired to restore the original seawall surface in order to best protect reinforcement steel and maintain the structural integrity of the seawall. Exposed and corroded reinforcement steel should be cleaned or replaced as necessary and then covered with an adequate thickness of durable patching material.

Additionally, the drainage channel walls in the vicinity of the 11th street outlet have experienced rotational and/or translation failure with loss of bank material behind them.

Coastal and Hydraulic Data

The climate in the project area is subtropical, characterized by warm summers and short, mild winters. The average daily temperature ranges in the summer and winter are 72–89 and 42–63 degrees Fahrenheit, respectively. The average annual rainfall is about 64 inches, and is well distributed throughout the year. Precipitation records indicate July as the wettest month, while October is the driest. The National Climatic Data Center climactic summary for Pascagoula is shown on Table 1.

Table 1.
Climactic Summary, Pascagoula 3 NE, MS (Station No. 226718)

Period of Record : 1/ 1/1948 to 8/31/2005

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	60.6	63.5	69.2	76.3	83.0	88.2	90.1	90.0	87.0	79.7	70.2	63.3	76.7
Average Min. Temperature (F)	41.9	44.4	50.8	58.1	65.6	71.5	73.6	73.0	69.2	58.6	49.7	44.1	58.4
Average Total Precipitation (in.)	4.90	4.94	6.03	4.70	4.81	5.59	7.30	6.73	6.99	3.81	4.11	4.54	64.47
Average Total SnowFall (in.)	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Source: Southeast Regional Climate Center.

Figure 3. Pascagoula Shoreline West From the East End of Beach Park Hurricane Katrina High Water Marks in Feet NAVD 88 Datum Shown in Red.

Figure 4. Pascagoula Shoreline West from Beach Park Before Hurricane Katrina.

Figure 5. Concave Seawall, Culvert Outfall, and Pier, Looking East. Beach Boulevard is to the Left.

Figure 6. Concrete Culvert Outfall Monolith and Chevron Deflector, Near Spanish Point.

Figure 7. Drainage Channel Near 11th Street Before Hurricane Katrina.

Figure 8. Drainage Channel Near 11th Street After Hurricane Katrina.

Figure 9. Drainage Channel From Beach Blvd. Looking Upstream to Washington Street Bridge.

Figure 10. Intact Cap on West End of Cellular Concrete Box Seawall.

Figure 11. Damaged Seawall Joint with Exposed Reinforcement Steel.

Mississippi Sound is a shallow water body bordered approximately 10 miles to the south by the barrier islands Cat Island and Ship Island. Typical depths in the sound range from 2 to 5 meters. The shoreline slope in the vicinity of Pascagoula is relatively flat with the 2 meter depth contour located a few hundred yards offshore and as far as 1.5 miles offshore. The water is knee deep at 150 yards in the vicinity of Beach Park.

Circulation patterns within the vicinity of the study area are controlled by astronomical tides and prevailing winds. Accumulations at culvert outfalls suggest a weak east-to-west littoral drift in the site vicinity. Bottom sediments near Pascagoula are composed of fine sands and silts. The mean diurnal tide range is on the order of 1.6 feet, and the extreme (except during storms) range is about 3.5 feet. Predominant winds average eight miles per hour (mph) from the south during the summer and from the northeast during the winter. Though the tides produced by astronomical forces are relatively small in magnitude, the wind can produce larger variations. Strong winds from the north can evacuate the sound causing current velocities of several knots in the passes to the gulf. Winds from the southeast can produce high tides, piling water up against the shoreline. The study area has been impacted by several tropical storms and hurricanes, most recently from Tropical Storm Isidore in 2002 and Hurricane Katrina in August 2005. Hurricane Katrina surge estimated from high-water marks range from 16.5 to 18 feet mean sea level (appx. 12 to 14 feet above the top of seawall) near the site. Based on the maximum annual sea stage-frequency curve for Biloxi, Mississippi, a community in Harrison County to the west of Pascagoula, these heights suggests, approximately, a 100-year recurrence interval storm surge event.

Geotechnical Data

The existing sub-grade behind the existing concrete stream walls to be replaced consist of poorly graded sands and silty sands from El. 5.0 with more silty sands and possible organic content beyond El. -10.0, becoming less silty beyond El. -15.

The alternative solutions provide for various types of sheet piling to be driven along the streambank channel wall alignment on both sides. The design of the sheet piles should be based on soils having an in place density of 110 PCF, a cohesion of 300 PSF and an angle of internal friction of 25 degrees. The soils will assume to be saturated below El. 3.0 NGVD. The new walls can be access from both sides. Lateral earth pressure coefficients can be derived from the soil values provided but the wall penetration should be on the order of 1.5 times the unsupported length for any section of wall.

Typical profiles for the beach and dune alternatives are shown in Figure 12. The materials to be used for this effort will come from inland sources within 10 miles of the project. Materials used for the re-nourishment and dune construction will have 90% passing the #40 sieve and only 10% will pass the #200 sieve. The sand fill shall not have noticeable amounts of shell and/or gravel. The sand will be transported by truck, dumped ashore and shaped along the proposed alignment. The beach will be about 150 feet wide and sloped from elevation 5.0+/- to Elevation 3.5 and then sloped to MLLW at 1 vertical to 10 horizontal.

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternative Plans

Three alternative plans were evaluated for shoreline protection and environmental enhancement. The first plan addresses seawall repair, replacement of the streambank walls of the drainage channel west of 11th Street, and repair of that channel's right extension wall pile cap. These measures are recommended as part of all other alternatives due to the need to preserve a robust shoreline protection and drainage system. The second alternative addresses environmental restoration objectives. The third alternative is a permutation of the second in that a dune is added to enhance the environmental value of the sand beach and to provide a source of sand replenishment for beach sand lost over time to littoral processes.

- ***Alternative 1: Seawall and Channel Repair.*** The objective of this alternative is to restore the shoreline storm defense system by improving the seawall's damaged condition, replacing the failed drainage channel walls, and replacing that channels right extension wall pile cap.

  The concave seawall west of Beach Park would have its joints cleaned and re-sealed (approximately 237 joints); impacted and spalled areas re-surfaced; exposed rebar cleaned, treated, and re-covered; and significant longitudinal and transverse cracks would be sealed.

  The seven destroyed cell caps of the cellular seawall east of Beach Park would receive new cell caps. The cells covered by the caps would first be cleared of debris and backfilled with suitable material.

  The failed stream bank panels of the drainage channel west of 11th street would be removed and replaced and the remains of approximately 60 feet of this stream's extension wall cap would be removed and replaced with a new reinforced concrete cap. Vinyl sheet pile and concrete wall panels have been evaluated for channel wall replacement. Preliminary materials and construction costs are about equal for both concrete and vinyl. Concrete panels would be founded upon piles. Real estate boundaries at the channel margin are not definitively known at this time, but because private property owners utilize land right up to the stream banks, it is possible that real estate costs may be substantial.

- ***Alternative 2: Seawall and Channel Repair and Beach.*** Historically, a delicate balance existed between the available sand supplied to the beach and that borne away by near-shore currents. Where they exist, seawalls along the Mississippi Coast have eliminated the shoreline supply and reflected local wave energy. Over time, the sand beaches have disappeared most of the armored south facing Mississippi coast, as have the shoreline ecological communities dependent upon the sand beaches. In addition to seawall and channel repair, this alternative would provide for the placement of a sand beach to enhance the environmental value of the shoreline. A secondary benefit is that the sand, being placed up to and against the seawall, would greatly reduce the migration of fines through and beneath the seawall, which is a recurring and expensive maintenance issue for the city. The beach would extend from the west end of the seawall near Spanish Point to the drainage channel just west of Beach Park, a distance of approximately 7,700 feet. A schematic elevation view of the alternative is shown in Figure 12 (the dune shown in that figure applies to Alternative 3). Assuming an average depth of placement of four feet and a waste factor of 15%, approximately 229,000,cubic yards of medium to fine-grained sand would be needed. The beach would need to be periodically re-nourished; beach maintenance experience in neighboring Harrison County suggests a 12-year re-nourishment cycle. Existing drainage channel guidewalls would not need to be extended for this alternative. However, because the drainage culverts on the beach side of the seawall must be extended at nearly 15 times their current length, it is necessary to assume that all 14 culverts would need to be enlarged. If adequate discharge could be provided by joining the ends of the existing culverts to an enlarged culvert via an expanding section the cost and effort would not be great. Otherwise, if the culverts must be replaced in their entirety in order to provide adequate conveyance, the culvert replacement and extension cost would be greater than presently estimated, as excavation would need to proceed upstream through the seawall and, possibly, Beach Boulevard, to the nearest convenient location (perhaps a junction) to insert the replacement culvert sections.

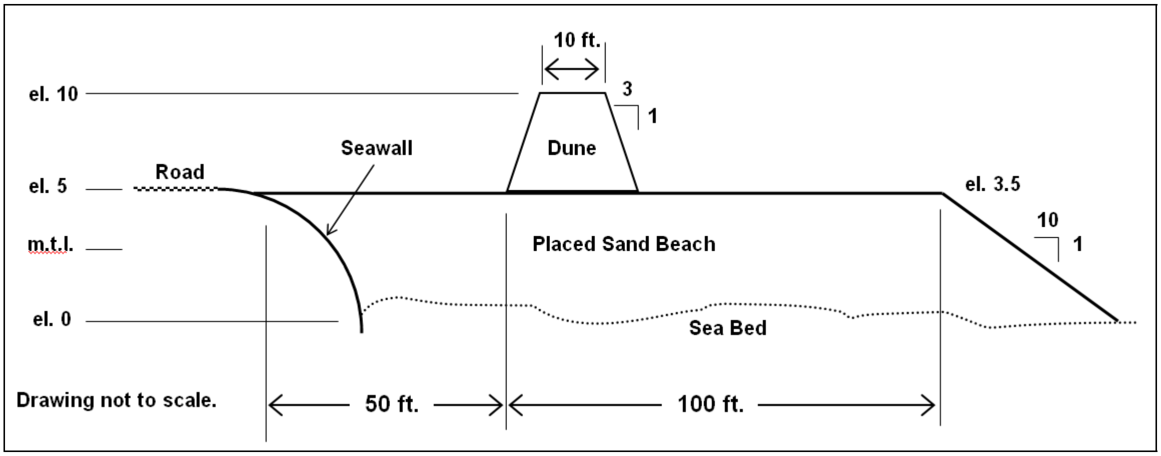


Figure 12. Elevation View, Alternative 2 and Alternative 3 (with Dune) Beach.

- ***Alternative 3: Seawall and Channel Repair and Beach with Dune.*** This alternative adds a dune to the beach profile of Alternative 2 as shown in Figure 12. The purpose of the dune is to provide vertical ecological complexity to an otherwise horizontal beach environment. The dune would be utilized by species that would otherwise not inhabit a horizontal sand beach. Because the primary purpose of the dune is to provide ecological benefit, pedestrian pathways would not be constructed over them. The dunes would be vegetated and sand fencing would be installed to help resist landward dune migration and wind-borne sand loss. The dune would also provide a source of beach material for sand borne away by nearshore currents. The estimated quantity of sand required to construct the dunes as shown in the figure is approximately 41,000 cubic yards, with 8 acres of plantings and approximately 8,470 feet of sand fencing.

Construction Procedures and Water Control Plan

11th Street Channel Walls

It is assumed that the channel will not have to be dewatered. Remove the existing walls, backfill material, trees, and existing building slab. Drive new pile, pour new cap, and backfill. Private property bounds may be very close to the channel alignment. About 8 fairly large trees may need to be removed to accommodate the work, as well as the remains of a building slab and foundation. South of the road remove 60 feet of damaged training wall cap by jackhammer to expose the tops of the existing concrete pile sheets, place formwork and steel reinforcement, then pour new cap.

Repair of existing concave seawall

This work would consist of removing caulking material, cleaning and opening the joints sufficient to press durable, flexible or rubberized gasket material into the joints. Twenty to thirty joints have been repaired in this manor and appear to have held up very well. There are also substantial spalls that need to be repaired, this would involve removing loose spall material, sand blasting, and patching with epoxy cement. Also in some areas there are fairly wide longitudinal cracks and a few transverse cracks, these should be sealed with a suitable epoxy or other durable material to help protect the concrete reinforcement from corrosion.

Repair of existing cellular seawall

The existing cells would be vacated of accumulated storm debris and backfilled with suitable material. Caps may be pre-cast or cast in place. Some attention will need to be given to the color of the finished concrete in order to match the yellowish earth tone of the existing works.

Beach and Dune Construction

Sand would be trucked in from upland sources, dumped on-site, and graded to suit. The 14 concrete culverts would be extended approximately 185 feet to the beach edge with the discharge ends anchored by monolithic concrete slabs. Some culverts may need to be replaced by larger culverts to maintain capacity with increased length.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance

Estimated O&M costs are summarized in Table 2 and detailed in Tables 3 and 4. The proposed seawall and channel repairs do not introduce new operations and maintenance (O&M) costs to local interests. Therefore, O&M costs for these items are summarized in Table 2. Alternatives 2 and 3 introduce new features to the waterfront, and O&M cost estimates have been developed for them as shown in Tables 3 and 4.

The beach without dunes is thought to require re-nourishment on a 5-year cycle on average. Wind-blown sand will also need to be swept off of Beach Boulevard on a regular basis, probably once a week. Maintenance would be similar with the dunes; however, the beach re-nourishment cycle might be lengthened to 7 to 10 years. Shoreward dune migration will require relocation of sand from the back of the beach to the shoreline approximately two times a year. Estimates for Harrison County suggest the quantity of wind-blown sand to be relocated would be less than 0.25 cubic feet per foot of beach (approximately 1,925 cubic yards) on an annualized basis. If a hurricane strikes, the dunes, dune vegetation, and fencing may be devastated and replacement may be necessary. Considering otherwise significant storm events, the base of the dune is at elevation 5.0 NGVD. If the still-water elevation at the base of the dune is the elevation at which storm surge, with additional wave action, would begin to erode the dune, an approximately five- to 10-year recurrence interval surge corresponds to this elevation based on frequency analysis of annual maximum water surface elevations at Biloxi.

Table 2.
O&M Summary

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	Summary	DATE	26-Jun-06
LOCATION:	Jackson County MS.	SHEET NO.	1	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	Summary	BASIS of ESTIMATE:	info furnished per PDT Team		
		FILE NAME:	pascagoula6-26.xls		

Alt No.	DESCRIPTION	Quantity	Unit		ESTIMATED AMOUNT
1.	***Seawall and Channel Repair***	1	job	xxx	$1,860,000
2.	***Seawall and Channel Repair and Beach***	1	job	xxx	$6,530,000
3.	***Seawall and Channel Repair and Beach with Dunes***	1	job	xxx	$7,510,000

Table 3.
O&M, Alternative 2

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	1	DATE	26-Jun-06
LOCATION:	Jackson County MS.	SHEET NO.	3	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Seawall and Channel Repair and Beach***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	pascagoula6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seawall and Channel Repair and Beach				
Mobilization, Preparatory Work, Demobilization	1	job	allow	200,000
demo exist conc channel	236	cy	50.00	11,800
new conc wall panel	19,800	sf	28.00	554,400
conc cap	40	cy	600.00	24,000
wood piles	660	lf	20.00	13,200
excav	300	cy	18.00	5,400
tie backs	1,320	lf	25.00	33,000
spall repair	1,140	lf	5.00	5,700
joint repair	3,400	lf	5.00	17,000
crack sealing	4,720	lf	5.00	23,600
cell caps	3	cy	600.00	1,800
compacted fill	8	cy	25.00	200
Misc. Site Items	1	ls	allow	150,000
beach fill	229,000	cy	10.00	2,290,000
36" drain pipe	2,590	lf	50.00	129,500
conc chevron	82	cy	600.00	49,200
conc walls	45	cy	600.00	27,000
Total Direct Construction Cost				$3,535,800
Indirect Cost	@		15%	530,370
				4,066,170
Profit	@		9%	365,955
				4,432,125
Bond	@		1.5%	66,482
17 Account, Beach Replenishment		Current Contract Cost, Oct 06		$4,498,607
01 Account, Lands & Damage (PCA)		LS		200,000
				4,698,607
30 Account, Plan, Engr.& Design		10%		469,861
				5,168,468
31 Account, Constr. Management		6%		310,108
				5,478,576
CONTINGENCY		20%		1,055,715
				6,534,291
				$6,534,291
				rounded
TOTAL PROJECT COST, FY-07				**$6,530,000**

Table 4.
O&M, Alternative 3

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	1	DATE	26-Jun-06
LOCATION:	Jackson County MS.	SHEET NO.	4	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:		BASIS of ESTIMATE:			Info furnished per PDT Team
Seawall and Channel Repair and Beach with Dunes		FILE NAME:	pascagoula6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seawall and Channel Repair and Beach with Dunes				
Mobilization, Preparatory Work, Demobilization	1	job	allow	200,000
demo exist conc channel	236	cy	50.00	11,800
new conc wall panel	19,800	sf	28.00	554,400
conc cap	40	cy	600.00	24,000
wood piles	660	lf	20.00	13,200
excav	300	cy	18.00	5,400
tie backs	1,320	lf	25.00	33,000
spall repair	1,140	lf	5.00	5,700
joint repair	3,400	lf	5.00	17,000
crack sealing	4,720	lf	5.00	23,600
cell caps	3	cy	600.00	1,800
compacted fill	8	cy	25.00	200
Misc. Site Items	1	ls	allow	150,000
beach fill	270,000	cy	10.00	2,700,000
sand fence	7,700	lf	5.00	38,500
Planting	8	ac	12500	100,000
36' drain pipe	2,590	lf	50.00	129,500
conc chevron	82	cy	600.00	49,200
conc walls	45	cy	600.00	27,000
	Total Direct Construction Cost			$4,084,300
	Indirect Cost	@	15%	612,645
				4,696,945
	Profit	@	9%	422,725
				5,119,670
	Bond	@	1.5%	76,795
17 Account, Beach Replenishment	Current Contract Cost, Oct 06			$5,196,465
	01 Account, Lands & Damage (PCA)		LS	200,000
				5,396,465
	30 Account, Plan, Engr.& Design		10%	539,647
				5,936,112
	31 Account, Constr. Management		6%	356,167
				6,292,278
	CONTINGENCY		20%	1,218,456
				7,510,734
				$7,510,734
				rounded
	TOTAL PROJECT COST, FY-07			**$7,510,000**

Cost Estimates

Estimated costs for the alternative plans are shown in Tables 5 through 8. Note that the elements of Alternative 1 are common to all alternatives as discussed previously. Table 5 summarizes costs, and Tables 6 through 8 provide cost estimate detail. Quantity estimates are based on drawings and/or rudimentary field measurements. These costs include contingencies, costs for engineering and design (E&D), and construction management. The E&D cost for preparation of construction contract plans and specifications (P&S) includes a detailed contract survey and management of the survey contract, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement packages, project engineering and coordination, supervision, technical review, computer costs, and reproduction.

Table 5.
Estimated Costs Summary

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	Summary	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	1	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	Summary	BASIS of ESTIMATE:	info furnished per PDT Team		
		FILE NAME:	pascagoula6-26.xls		

Alt No.	DESCRIPTION	Quantity	Unit		ESTIMATED AMOUNT
1.	***Seawall and Channel Repair***	1	job	xxx	$1,790,000
2.	***Seawall and Channel Repair and Beach***	1	job	xxx	$6,470,000
3.	***Seawall and Channel Repair and Beach with Dunes***	1	job	xxx	$7,460,000

Notes:
Price Level, Oct 06
Unit Cost based on Historical Data, Recent Pricing, & Estimator's Judgment

Table 6.
Alternative 1. Channel and Seawall Repair Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Seawall and Channel Repair***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	pascagoula6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seawall and Channel Repair				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
demo exist conc channel	236	cy	50.00	11,800
new conc wall panel	19,800	sf	28.00	554,400
conc cap	40	cy	600.00	24,000
wood piles	660	lf	20.00	13,200
excav	300	cy	18.00	5,400
tie backs	1,320	lf	25.00	33,000
spall repair	1,140	lf	5.00	5,700
joint repair	3,400	lf	5.00	17,000
crack sealing	4,720	lf	5.00	23,600
cell caps	3	cy	600.00	1,800
compacted fill	8	cy	25.00	200
Misc. Site Items	1	ls	allow	100,000
	Total Direct Construction Cost			$890,100
	Indirect Cost	@	15%	133,515
				1,023,615
	Profit	@	9%	92,125
				1,115,740
	Bond	@	1.5%	16,736
09 Account, Channels & Canals		Current Contract Cost, Oct 06		$1,132,476
	01 Account, Lands & Damage (PCA)		LS	150,000
				1,282,476
	30 Account, Plan, Engr.& Design		10%	128,248
				1,410,724
	31 Account, Constr. Management		6%	84,643
				1,495,368
	CONTINGENCY		20%	299,074
				1,794,441
				$1,794,441
				rounded
	TOTAL PROJECT COST, FY-07			**$1,790,000**

Table 7.
Alternative 2. Seawall and Channel Repair and Beach Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	3	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Seawall and Channel Repair and Beach***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	pascagoula6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seawall and Channel Repair and Beach				
Mobilization, Preparatory Work, Demobilization	1	job	allow	200,000
demo exist conc channel	236	cy	50.00	11,800
new conc wall panel	19,800	sf	28.00	554,400
conc cap	40	cy	600.00	24,000
wood piles	660	lf	20.00	13,200
excav	300	cy	18.00	5,400
tie backs	1,320	lf	25.00	33,000
spall repair	1,140	lf	5.00	5,700
joint repair	3,400	lf	5.00	17,000
crack sealing	4,720	lf	5.00	23,600
cell caps	3	cy	600.00	1,800
compacted fill	8	cy	25.00	200
Misc. Site Items	1	ls	allow	150,000
beach fill	229,000	cy	10.00	2,290,000
36" drain pipe	2,590	lf	50.00	129,500
conc chevron	82	cy	600.00	49,200
conc walls	45	cy	600.00	27,000
	Total Direct Construction Cost			$3,535,800
	Indirect Cost	@	15%	530,370
				4,066,170
	Profit	@	9%	365,955
				4,432,125
	Bond	@	1.5%	66,482
17 Account, Beach Replenishment		Current Contract Cost, Oct 06		$4,498,607
	01 Account, Lands & Damage (PCA)		LS	150,000
				4,648,607
	30 Account, Plan, Engr.& Design		10%	464,861
				5,113,468
	31 Account, Constr. Management		6%	306,808
				5,420,276
	CONTINGENCY		20%	1,054,055
				6,474,331
				$6,474,331
				rounded
	TOTAL PROJECT COST, FY-07			**$6,470,000**

Table 8.
Alternative 3. Seawall and Channel Repair and Beach with Dune Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Pascagoula Repairs	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	4	OF	5
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:		BASIS of ESTIMATE:			Info furnished per PDT Team
Seawall and Channel Repair and Beach with Dunes		FILE NAME:	pascagoula6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Seawall and Channel Repair and Beach with Dunes				
Mobilization, Preparatory Work, Demobilization	1	job	allow	200,000
demo exist conc channel	236	cy	50.00	11,800
new conc wall panel	19,800	sf	28.00	554,400
conc cap	40	cy	600.00	24,000
wood piles	660	lf	20.00	13,200
excav	300	cy	18.00	5,400
tie backs	1,320	lf	25.00	33,000
spall repair	1,140	lf	5.00	5,700
joint repair	3,400	lf	5.00	17,000
crack sealing	4,720	lf	5.00	23,600
cell caps	3	cy	600.00	1,800
compacted fill	8	cy	25.00	200
Misc. Site Items	1	ls	allow	150,000
beach fill	270,000	cy	10.00	2,700,000
sand fence	8,470	lf	5.00	42,350
Planting	8	ac	12500	100,000
36" drain pipe	2,590	lf	50.00	129,500
conc chevron	82	cy	600.00	49,200
conc walls	45	cy	600.00	27,000
	Total Direct Construction Cost			$4,088,150
	Indirect Cost	@	15%	613,223
				4,701,373
	Profit	@	9%	423,124
				5,124,496
	Bond	@	1.5%	76,867
17 Account, Beach Replenishment		Current Contract Cost, Oct 06		$5,201,363
	01 Account, Lands & Damage (PCA)	LS		150,000
				5,351,363
	30 Account, Plan, Engr.& Design	10%		535,136
				5,886,500
	31 Account, Constr. Management	6%		353,190
				6,239,690
	CONTINGENCY	20%		1,217,938
				7,457,628
				$7,457,628
				rounded
	TOTAL PROJECT COST, FY-07			**$7,460,000**

Schedule and Design for Construction

A typical schedule for preparation of P&S through construction is shown in Table 9.

Table 9.
Typical Schedule for P&S

Draft P&S	8 weeks after start
ITR/BCOE review	3 weeks after draft P&S
Final P&S/RTA	3 weeks after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	1 year after NTP

UPPER BAYOU CASOTTE

General

The hurricanes of 2005 caused damage to drainage ways by blowing trees and other debris into these areas and by deposition of sediment in many areas of Jackson County, MS. Some canals and drainage ways were affected. This document provides information regarding damage to two of the drainage ways flowing into Bayou Casotte or Pt Aux Chennes Bay. Rough order-of-magnitude cost estimates for restoring the capacity of these water courses is also presented.

Location

A general location map of the study area is shown below. The area is in the city of Moss Point near the intersection of Hwy 63 and US Hwy 90. The area is relatively flat, with some small interconnecting ditches apparently draining different directions. The drainage ways of interest are shown below.

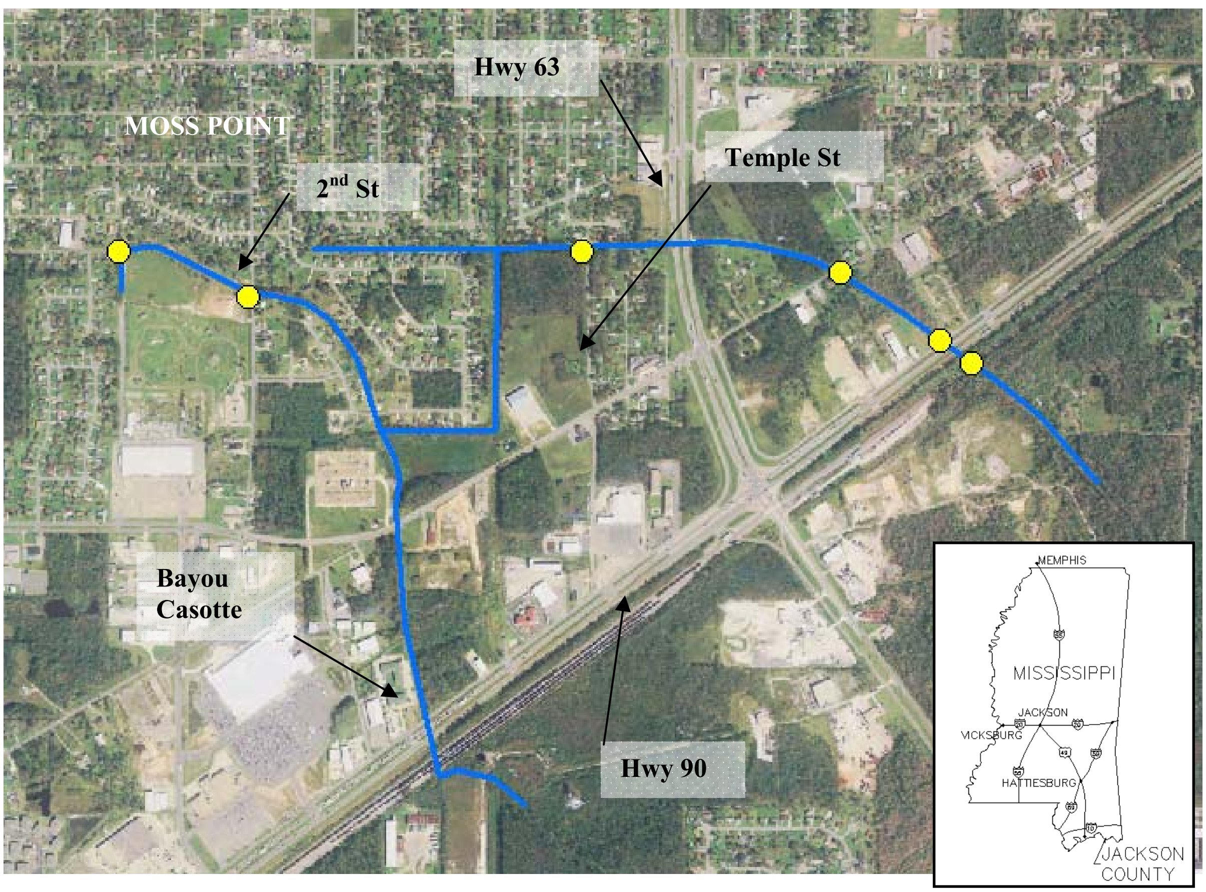


Figure 1. Upper Bayou Casotte

Existing Conditions

Sediment, growth and debris have resulted in clogged ditches in the area. Photos are shown below.

Figure 2. Upper Bayou Casotte – Temple St

Figure 3. Upper Bayou Casotte – Hwy 63

Figure 4. Upper Bayou Casotte – 2nd Avenue Debris at Culvert

The drainage way varies in width from approximately 9 ft–15 ft. with an average of approximately 12 ft. The length is approximately 2.71 miles. An engineer representing Hancock County did not know the degree of shoaling due to the 2005 hurricane season. There appears to be significant debris in the drainage way, especially at some of the culverts. USGS quad sheets indicate that the elevation of the subdivisions is between elevation 15 ft and 20 ft NGVD. The drainage way apparently partially flows into Bangs Lake and Pt Aux Chennes Bay, and partially into Bayou Casotte through interconnected drainage ways. Long term relief would consist of flow efficiency improvement in the lower part of the drainage system.

Coastal/Hydraulics

Hurricane Katrina inundation limits provided by FEMA are estimated approximately at elevation 15 ft NGVD as shown below.

Figure 5. Upper Bayou Casotte -Inundation Contours

Additional data is provided in a report to FEMA by URS Group, Inc., titled “Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report,” 16 January 2006, as well as in a report by FEMA titled “Draft Report, Hurricane Katrina Flood Frequency Analysis,” dated September 2005. Results are summarized below.

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b) (Tables 1 and 2 and Figure 6). The flood frequency analysis only represents conditions at and near the gage.

Table 1.
Selected Tidal Gage Stations from NOAA

Station ID	Name	Latitude	Longitude	Begin Year	End Year
8729840	Pensacola, Pensacola Bay, FL	30.40 N	87.21 W	1924	2005
8735180	Dauphin Island, Mobile Bay, AL	30.25 N	88.08 W	1967	2005
8747766	Waveland, Mississippi Sound, MS	30.28 N	89.37 W	1979	2005
8761724	Grand Isle, East Point, LA	29.26 N	89.96 W	1972	2005

Table 2.
Selected Tidal Gages from USGS/USACE

Name	Latitude	Longitude	Begin Year	End Year
Back Bay Biloxi at Biloxi, MS	30.40 N	88.84 W	1882	1998
Pascagoula River at Pascagoula, MS	30.37 N	88.56 W	1940	1998

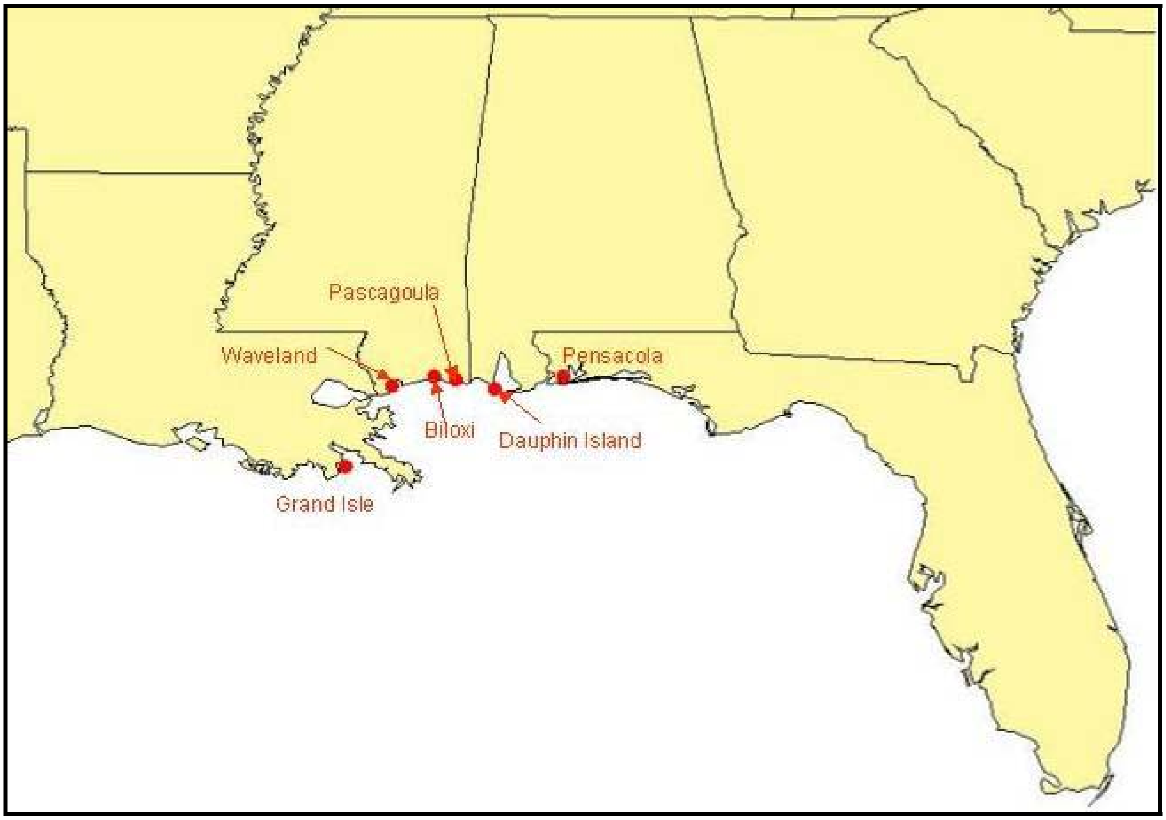


Figure 6. Tidal Gage Locations in the Area Impacted by Hurricane Katrina

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.
- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.

- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.
- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet.
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet.
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet.

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 3. As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 3.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations Location Katrina Elevation

Location	Katrina Elevation (Ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
Dauphin Island, AL	5.81	20
Pensacola, FL	6.07	50

A stage-frequency curve developed by the Corps of Engineers for the Biloxi gage is shown below. The gage shows stage 24 to have a return frequency of 100 years compared to the FEMA table which shows a return interval of >10,000 years.

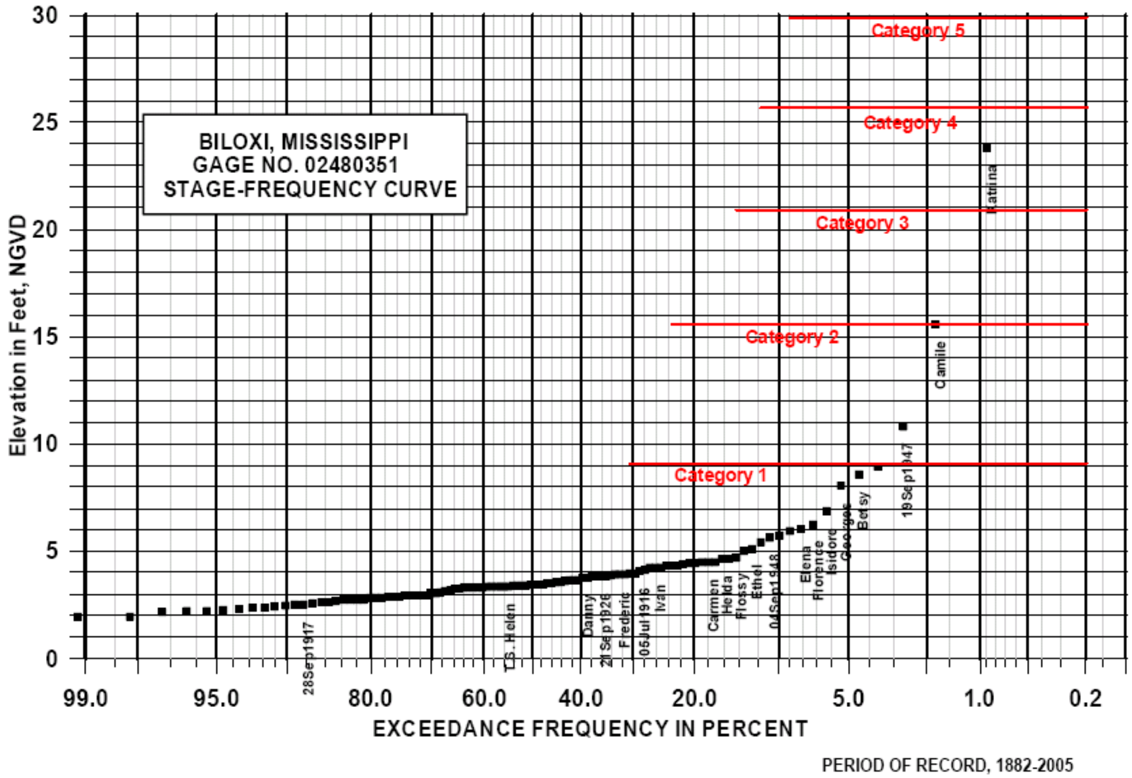


Figure 7. Stage Frequency Curve

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at the site are unknown. Subsurface conditions are assumed to be similar to those at the closest available geotechnical borings. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to this site is the Mississippi Department of Geology boring identified as JK9. This boring is located approximately 6500 feet east of the site. Sample descriptions and grain size data for the upper 10 feet of this boring are summarized in the table below.

Table 4.
Mississippi Department of Geology Boring JK9 (upper 10 feet)

Depth	Description	% Grvl	% Sand	%Silt/Clay
0’ 0” – 1’ 6”	muddy fine sand	0.0	76.0	15.1 / 8.9
2’ 6” – 4’-0”	clayey fine sand	0.0	76.6	7.8 / 15.6
5’ 0” – 7’ 0”	clayey fine sand	0.0	57.7	10.2 / 32.1
7’ 0” – 8’ 6”	fine sandy clay	0.0	35.5	20.6 / 43.9
8’ 6” – 10’ 0”	fine sandy mud	0.0	30.6	42.2 / 27.2

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects, and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

- ***Alternative 1.*** Sediment Removal (2ft). This alternative is a short term alternative that would consist of removing approximately 2 ft of sediment over an average width of 15 ft and length of 2.71 miles, as shown in Figures 2 and 8. There appears to be significant debris in the drainage way, especially at some of the culverts, which would also have to be removed to facilitate removal of the sediment. Because several culvert locations appeared to be significantly clogged with growth and debris, the results of this work is expected to have significant benefit.

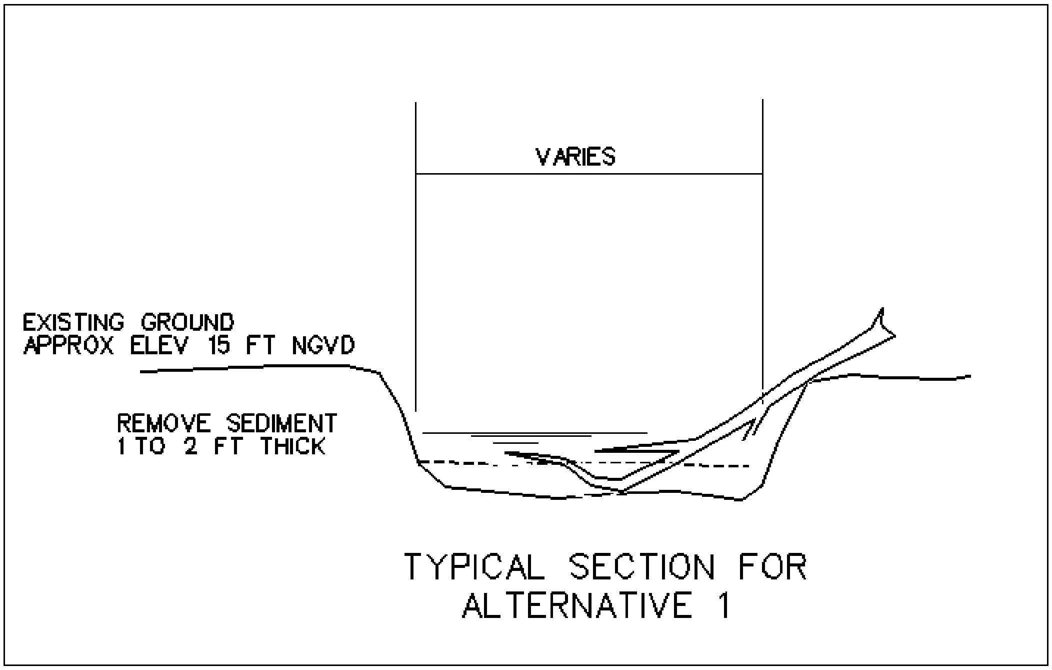


Figure 8. Alternative 1. Sediment Removal

- ***Alternative 2.*** Sediment Removal (1ft). This alternative is the same as Alternative 1 except that only 1 foot of sediment would be removed. No additional drawing is provided. This alternative would result in slightly less benefits.

Construction Procedures and Water Control Plan

Construction would be done by using marsh buggy type back-hoe or other mechanical excavation equipment and dump trucks. Material could be stockpiled to drain and hauled to a land fill area, since some debris is involved. If a marsh buggy equipment is used, water control would not be a problem.

Project Security

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions

Operations and Maintenance

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Shoaling is expected to be minimal except in the event of a rare hurricane event.

Table 6.
Upper Bayou Casotte - Alternative 1 Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Upper Bayou Cassotte	ITEM NO.	1	DATE	26-Jun-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Upper Bayou Cassotte Removal 2 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	upper bayou cassotte6-26.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Upper Bayou Cassotte Removal 2 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
excavation	15,900	cy	20.00	318,000
clearing	10	ac	5000.00	50,000
grassing	10	ac	5000.00	50,000
Misc. Site Items	1	ls	allow	50,000
Total Direct Construction Cost				$568,000
Indirect Cost	@		**15%**	85,200
				653,200
Profit	@		**9%**	58,788
				711,988
Bond	@		**1.5%**	10,680
09 Account, Channels & Canals			**Current Contract Cost, Oct 06**	$722,668
01 Account, Lands & Damage (PCA)			**LS**	262,500
				985,168
30 Account, Plan, Engr.& Design			**8%**	78,813
				1,063,981
31 Account, Constr. Management			**6%**	63,839
				1,127,820
CONTINGENCY			**20%**	173,064
				1,300,884
				$1,300,884
				rounded
TOTAL PROJECT COST, FY-07				**$1,300,000**

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 7.
Upper Bayou Casotte – Alt. 1– Maintenance Cost Estimate.

PROJECT:	Coastal MS Study, Upper Bayou Cassotte	ITEM NO.	1	DATE	21-Apr-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Upper Bayou Cassotte Removal 2 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m upper bayou cassotte4-22.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Upper Bayou Cassotte Removal 2 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
excavation	15,900	cy	20.00	318,000
clearing	10	ac	5000.00	50,000
grassing	10	ac	5000.00	50,000
Misc. Site Items	1	ls	allow	50,000
Total Direct Construction Cost				$568,000
Indirect Cost	@		**15%**	85,200
				653,200
Profit	@		**9%**	58,788
				711,988
Bond	@		**1.5%**	10,680
Current Contract Cost, Oct 06				$722,668
01 Account, Lands & Damage (PCA)			**LS**	0
				722,668
30 Account, Plan, Engr.& Design			**10%**	72,267
				794,935
31 Account, Constr. Management			**6%**	47,696
				842,631
CONTINGENCY			**20%**	168,526
				1,011,157
ESCALATION, FY-07			**1%**	10,112
				$1,021,268
				rounded
TOTAL PROJECT COST, FY-07				**$1,020,000**

Table 8.
Upper Bayou Casotte - Alternative 2 Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Coastal MS Study, Upper Bayou Cassotte
LOCATION: Hancock County MS.
WORK ITEM: ***Upper Bayou Cassotte Removal 1 Ft.***

ITEM NO. 1 DATE 26-Jun-06
SHEET NO. 2 OF 3
PREPARED: Parmer CHECKED: Ellsworth
BASIS of ESTIMATE: info furnished per PDT Team
FILE NAME: upper bayou cassotte6-26.xls

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Upper Bayou Cassotte Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
excavation	8,000	cy	20.00	160,000
clearing	10	ac	5000.00	50,000
grassing	10	ac	5000.00	50,000
Misc. Site Items	1	ls	allow	50,000
	Total Direct Construction Cost			$410,000
	Indirect Cost	@	**15%**	61,500
				471,500
	Profit	@	**9%**	42,435
				513,935
	Bond	@	**1.5%**	7,709
09 Account, Channels & Canals	**Current Contract Cost, Oct 06**			$521,644
	01 Account, Lands & Damage (PCA)		**LS**	262,500
				784,144
	30 Account, Plan, Engr.& Design		**8%**	62,732
				846,876
	31 Account, Constr. Management		**6%**	50,813
				897,688
	CONTINGENCY		**20%**	127,038
				1,024,726
				$1,024,726
				rounded
	TOTAL PROJECT COST, FY-07			**$1,020,000**

It is estimated that maintenance clearing will be required every 25 years at the cost shown below.

Table 9.
Upper Bayou Casotte – Alt. 2– Maintenance Cost Estimate.

PROJECT:	Coastal MS Study, Upper Bayou Cassotte	ITEM NO.	1	DATE	21-Apr-06
LOCATION:	Hancock County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Upper Bayou Cassotte Removal 1 Ft.***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m upper bayou cassotte4-22.xls		

D E S C R I P T I O N	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Upper Bayou Cassotte Removal 1 Ft.				
Mobilization, Preparatory Work, Demobilization	1	job	allow	100,000
excavation	8,000	cy	20.00	160,000
clearing	10	ac	5000.00	50,000
grassing	10	ac	5000.00	50,000
Misc. Site Items	1	ls	allow	50,000
Total Direct Construction Cost				$410,000
Indirect Cost	@		**15%**	61,500
				471,500
Profit	@		**9%**	42,435
				513,935
Bond	@		**1.5%**	7,709
Current Contract Cost, Oct 06				$521,644
01 Account, Lands & Damage (PCA)			**LS**	0
				521,644
30 Account, Plan, Engr.& Design			**10%**	52,164
				573,808
31 Account, Constr. Management			**6%**	34,429
				608,237
CONTINGENCY			**20%**	121,647
				729,884
ESCALATION, FY-07			**1%**	7,299
				$737,183
				rounded
TOTAL PROJECT COST, FY-07				**$740,000**

Table 10.
Typical Schedule for P&S

Draft P&S	3 months after start
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
Open bids	30 days after advertise
Award	30 days after open bids
NTP	3 weeks after award
Complete construction	4 months after NTP

References

Hurricane Katrina Rapid Response Mississippi Coastal & Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report, FEMA (URS Group, Inc.), 16 January 2006.

Draft Report, Hurricane Katrina Flood Frequency Analysis, FEMA, September 2005.

NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b).

FRANKLIN CREEK FLOODWAY

General

Flooding along a tributary of Franklin Creek in Jackson County, Mississippi has been a chronic problem for years. Highway construction has interrupted overland flow resulting in ponding which floods the area of Pecan, Mississippi. This report addresses a proposed flooding solution. Rough order-of-magnitude cost estimates for restoring the capacity of these water courses is also presented.

Location

General location maps of the study area are shown below. The area is near the communities of Orange Grove and Pecan Mississippi, near the Alabama – Mississippi state line. Franklin Creek and Franklin Creek Tributary flow into the Escatawpa River which flows into the Pascagoula River. The community is approximately 4 miles inland from Mississippi Sound at Grand Bay.

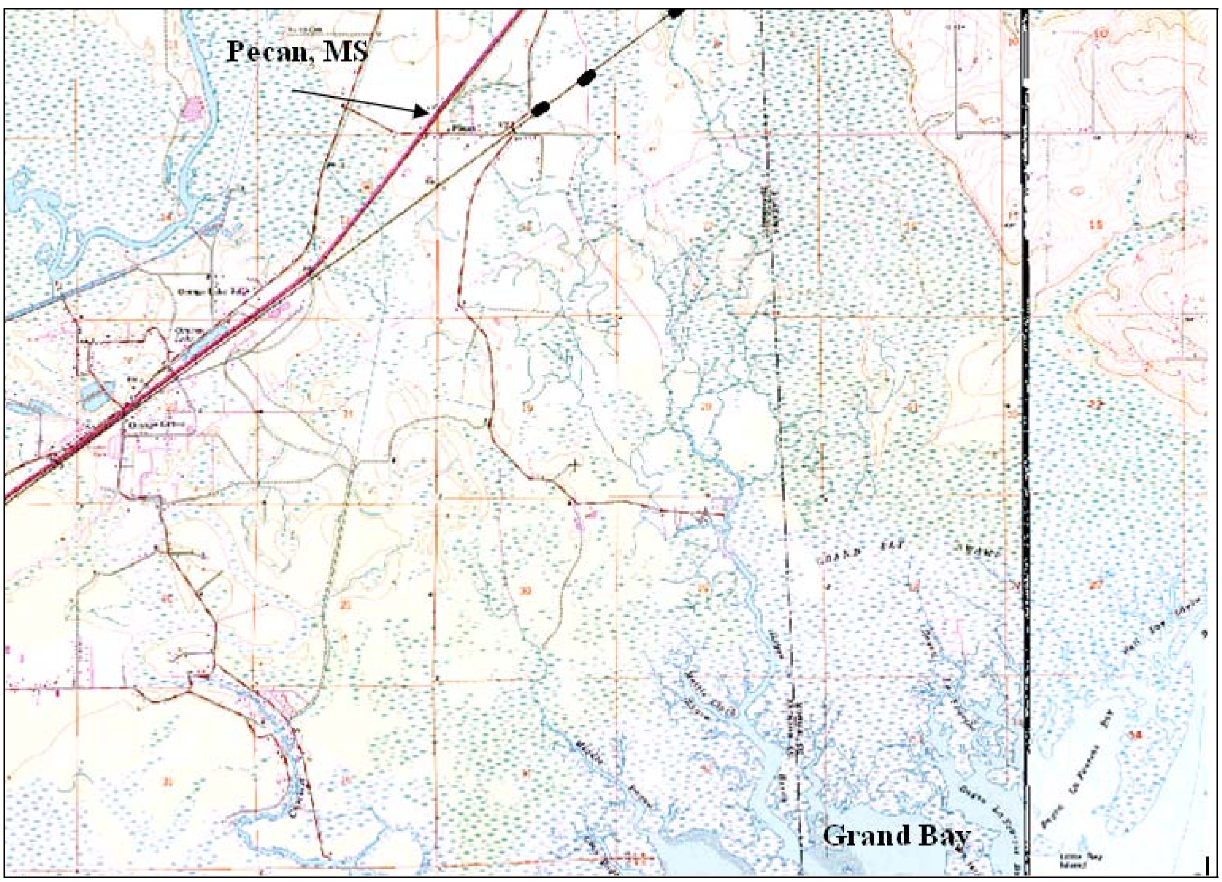


Figure 1. Vicinity Map

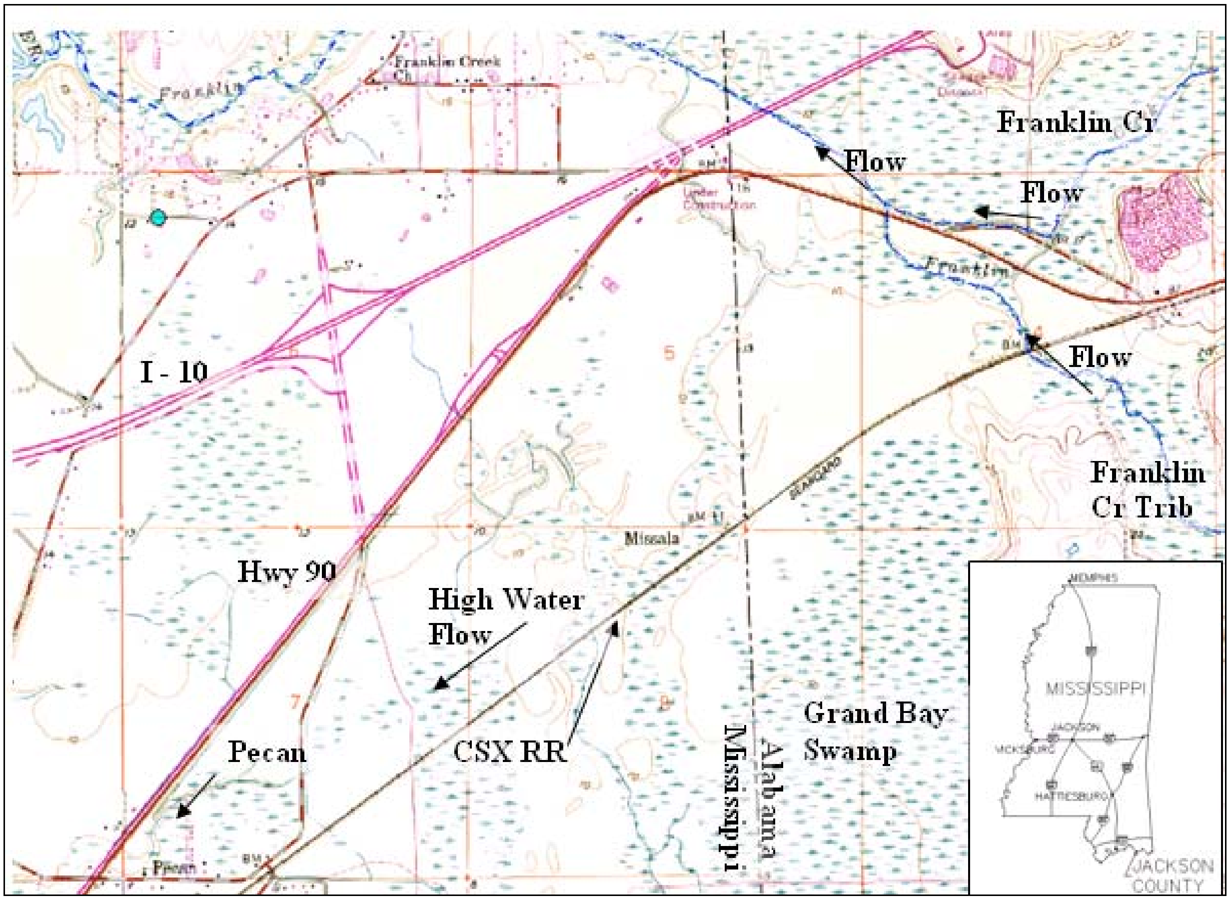


Figure 2. Location Map

Existing Condition

Franklin Creek Tributary intersects Franklin Creek above the CSX RR and Hwy 90, flooding the area bounded by Hwy 90 and the railroad, including Pecan, MS. This occurs especially during high water on the Pascagoula River and the Escatawpa River. Prior to 1950 and construction of the railroad, high flow from Franklin Creek and Franklin Creek Tributary could spill into a swamp and flow in a south-westerly into Grand Bay. Since construction of Hwy 90 and the railroad, water from the creek and tributary cannot easily flow in the original overbanks along the low flow path to the Escatawpa River or to the south to Grand Bay. Although during the 1950's, the low flow channel of Franklin Creek was relocated by the Alabama State Highway along the north side of the old highway, during high water, some of the Franklin Creek water still continues to southward, where it meets the Franklin Creek Tributary and then flows along the north side of the railroad to Pecan, MS.

Coastal/Hydraulics

Hurricane Katrina reached elevation 14.2 ft NGVD at Pecan, MS as shown below. The ground elevation at Pecan is between elevation 5 and 10 ft NGVD.

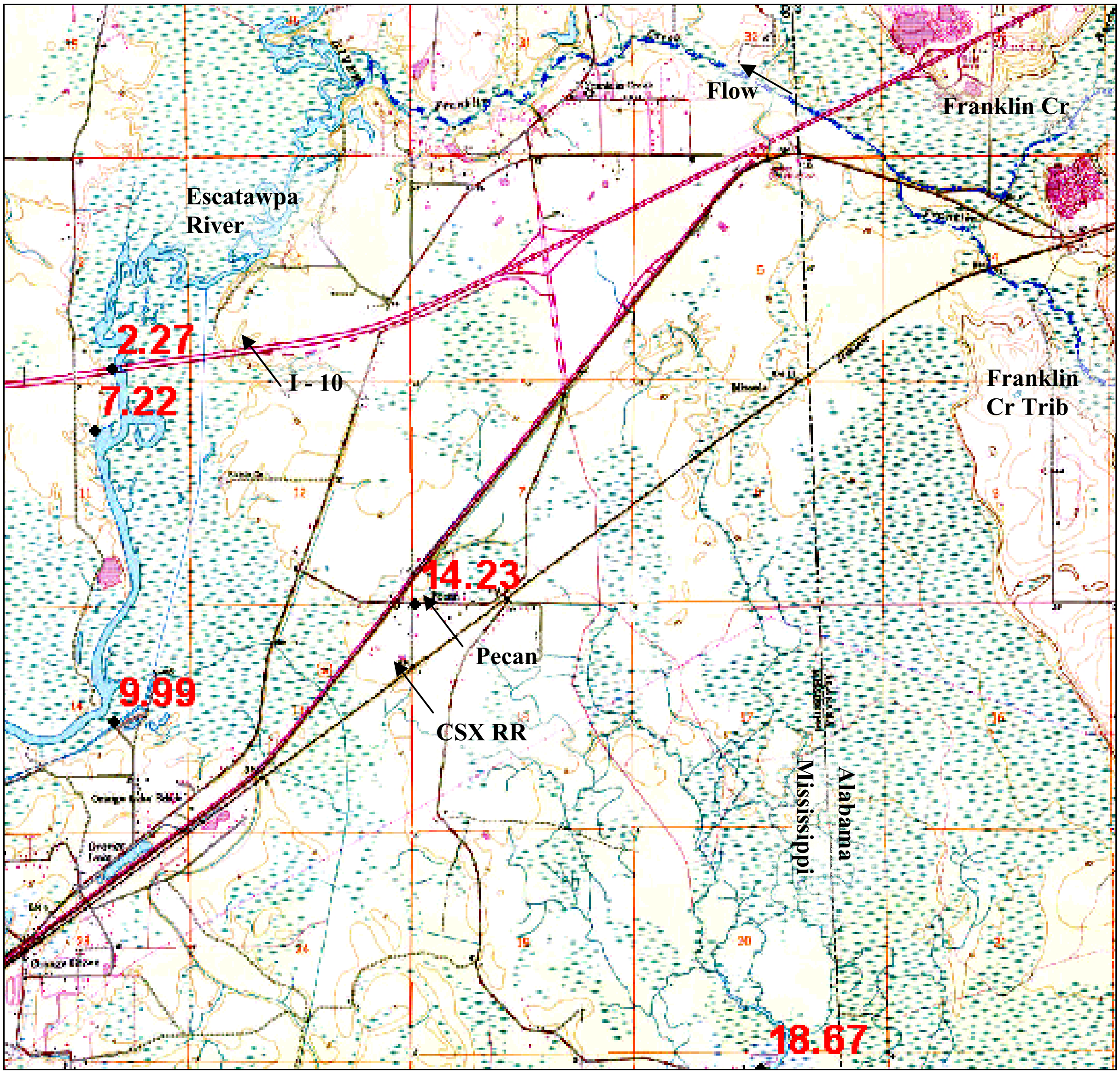


Figure 3. Hurricane Katrina High Water Marks

Additional data is provided in a report to FEMA by URS Group, Inc., titled "Hurricane Katrina Rapid Response Mississippi Coastal and Riverine High Water Mark (CHWM, RHWM) Collection, Draft Report," 16 January 2006, as well as in a report by FEMA titled "Draft Report, Hurricane Katrina Flood Frequency Analysis," dated September 2005. Results are summarized below.

While the best data available was used at the time of the flood frequency analysis, the reference data had limitations. Some stations were damaged or destroyed or malfunctioned during Hurricane Katrina and did not record the peak stage. Another limitation was that gages with long records of data are sparsely distributed. These gages provided useful records of a long sequence of historic storm surge peak heights. Where a useful gage record was available but the gage had failed during Hurricane Katrina, the analysis was based on the closest supplemental HWM data from NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b) (Tables 1 and 2 and Figure 6). The flood frequency analysis only represents conditions at and near the gage.

Table 1.
Selected Tidal Gage Stations from NOAA

Station ID	Name	Latitude	Longitude	Begin Year	End Year
8729840	Pensacola, Pensacola Bay, FL	30.40 N	87.21 W	1924	2005
8735180	Dauphin Island, Mobile Bay, AL	30.25 N	88.08 W	1967	2005
8747766	Waveland, Mississippi Sound, MS	30.28 N	89.37 W	1979	2005
8761724	Grand Isle, East Point, LA	29.26 N	89.96 W	1972	2005

Table 2.
Selected Tidal Gages from USGS/USACE

Name	Latitude	Longitude	Begin Year	End Year
Back Bay Biloxi at Biloxi, MS	30.40 N	88.84 W	1882	1998
Pascagoula River at Pascagoula, MS	30.37 N	88.56 W	1940	1998

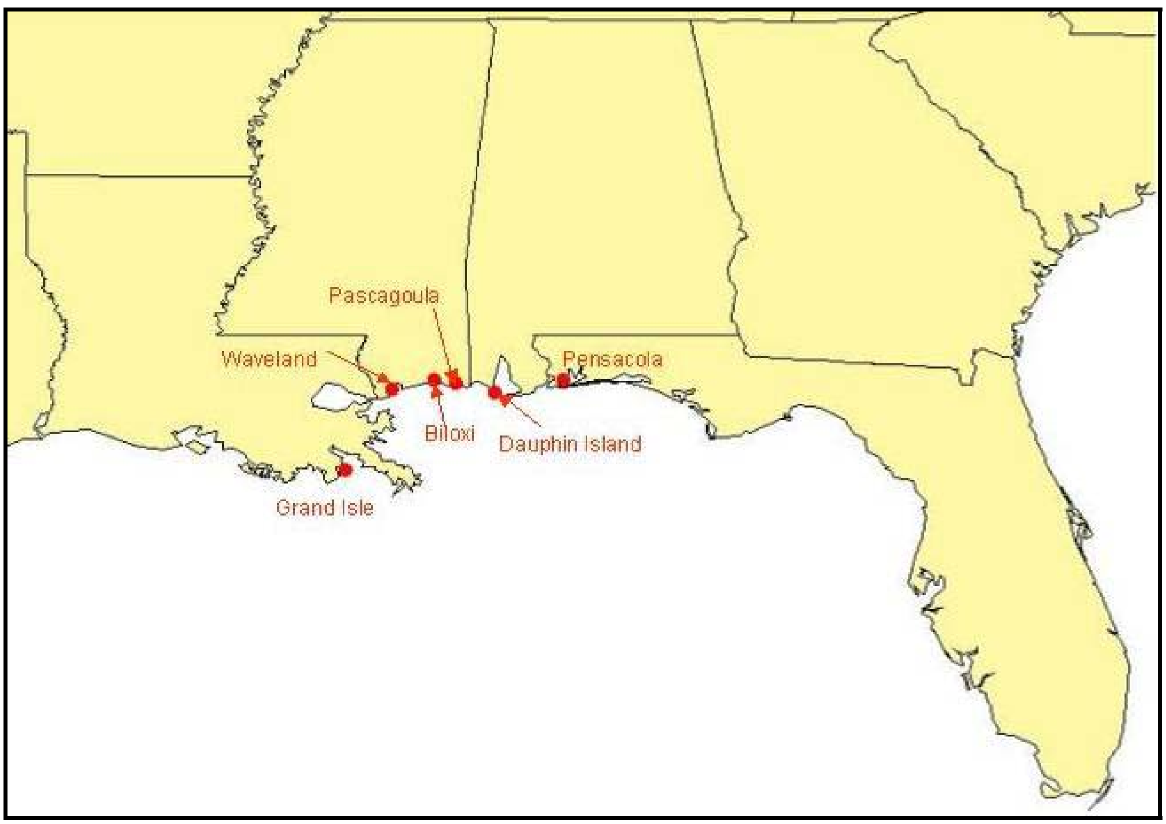


Figure 4. Tidal Gage Locations in the Area Impacted by Hurricane Katrina

The historical data were analyzed using seven different methods to estimate the elevation of various frequency events. The log-Pearson Type III results were considered the most applicable. Following is a summary of the results from the September 2005 report, which does not consider FEMA-surveyed high water mark information.

- At Biloxi, the 100-year elevation is 15.7 feet and the 500-year elevation is 28.7 feet. Therefore, the Hurricane Katrina elevation of 24 feet is estimated to be about a 250-year event at Biloxi, MS.
- At Pascagoula, the 100-year elevation is 11.9 feet and Katrina was 13 feet. Katrina is estimated to be about a 125-year event at Pascagoula, MS.
- At Waveland, the 100-year elevation is 17.6 feet and Katrina was 23 feet. The 200-year event is 22.8 feet (see Appendix D); therefore, Katrina is estimated to be about a 200-year event at Waveland. Note that the Katrina elevation of 23 feet was estimated from four high water marks obtained by USGS at a location north of Waveland near the intersection of I-10 and SR 43. It is possible that Katrina was higher than 23 feet at Waveland. The elevations of high water marks flagged at Waveland have not yet been determined.
- At Dauphin Island, the 100-year event is 7.5 feet and Katrina was 5.81 feet. The 50-year event is 6 feet; Katrina was about a 50-year event at Dauphin Island, AL.
- At Pensacola, the 100-year event is 7.3 feet and Katrina was 6.07 feet. The 50-year event is in the range of 5.8 feet, so Katrina is estimated to be about a 50-year event at Pensacola, FL.
- At Grand Isle, the recorder malfunctioned at an elevation of 5.17 feet, so the peak elevation of Katrina is not available. Therefore, no assessment of the frequency is provided.

The standard error, or 68-percent confidence limits, was determined for the 100-year elevation for the three Mississippi stations to give some estimate of the uncertainty in the flood elevations for the log-Pearson Type III results. Similar estimates could be made for the other stations. The lower and upper 68-percent confidence limits are listed below. The interpretation is that there is a 68-percent chance that the 100-year elevation is between the lower and upper 68-percent confidence limits.

- Waveland, 100-year elevation = 17.6 feet, lower limit = 10.4 feet, upper limit = 29.8 feet.
- Biloxi, 100-year elevation = 15.7 feet, lower limit = 11.4 feet, upper limit = 21.6 feet
- Pascagoula, 100-year elevation = 11.9 feet, lower limit = 8.3 feet, upper limit = 17.0 feet.

A summary of the flood frequencies for Hurricane Katrina based on the effective FEMA elevations can be found in Table 3 As can be seen, the estimated recurrence interval of Hurricane Katrina is unreasonably large for the three Mississippi stations, implying that the FEMA effective flood elevations are likely too low.

Table 3.
Flood Frequencies for Hurricane Katrina Based on Effective FEMA Flood Elevations Location Katrina Elevation

Location	Katrina Elevation (ft)	Estimated Frequency (Years)
Waveland, MS	23	>10,000
Biloxi, MS	24	>10,000
Pascagoula, MS	13	1,000
DauphinIsland, AL	5.81	20
Pensacola, FL	6.07	50

A stage-frequency curve developed by the Corps of Engineers for the Biloxi gage is shown below. The gage shows stage 24 to have a return frequency of 100 years compared to the FEMA table which shows a return interval of >10,000 years.

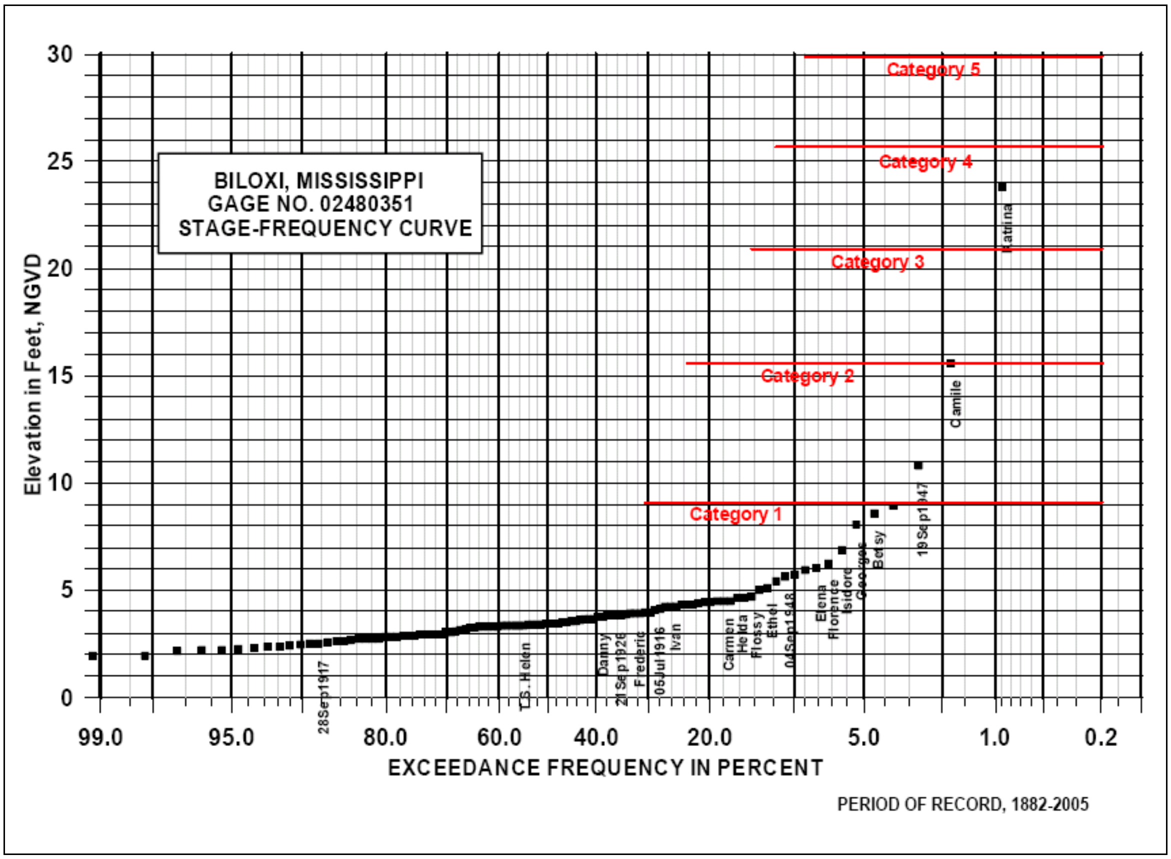


Figure 5. Stage-Frequency for Biloxi Gage

Geotechnical

Subsurface investigation has not been conducted for this project and subsurface conditions at this site are unknown. Subsurface conditions are assumed to be similar to those at the closest available geotechnical boring. A review was made of USACE Mobile District and Mississippi Department of Geology GIS subsurface information in Jackson County. The closest geotechnical boring to these sites is the Mississippi Department of Geology boring identified as JK9. This boring is located approximately 25000 feet west of the proposed excavation at the site. Sample descriptions and grain size data for the upper 10 feet of this boring are summarized in the table below.

Table 4.
Mississippi Department of Geology Boring JK9 Mississippi Department of Geology Boring JK9 (upper 10 feet)

Depth	Description	% Grvl	% Sand	%Silt/Clay
0′ 0″ – 1′ 6″	muddy fine sand	0.0	76.0	15.1 / 8.9
2′ 6″ – 4′ 0″	clayey fine sand	0.0	76.6	7.8 / 15.6
5′ 0″ – 7′ 0″	clayey fine sand	0.0	57.7	10.2 / 32.1
7′ 0″ – 8′ 6″	fine sandy clay	0.0	35.5	20.6 / 43.9
8′ 6″ – 10′ 0″	fine sandy mud	0.0	30.6	42.2 / 27.2

HTRW

Site inspections are currently being conducted by the U. S. Army Corps of Engineers, Mobile District, Environmental and Hazardous and Toxic Waste and Support Section, at and adjacent to the various proposed Coastal Mississippi Projects. These assessments are being conducted per the requirements of Engineer Regulation (ER) 1165-2-132 entitled, “Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects,” and the American Society of Testing and Materials Standard E 1527.

Inspections are being conducted to determine the presence or evidence of landfills, surface areas unable to support vegetation, visible sheens of petroleum product, nearby contaminated industrial facilities, or any type of visible indication that HTRW concerns exist that may impact the proposed projects.

Site inspections of adjacent properties, reviews of historic aerial photographs, and on site interviews are also being conducted to determine if HTRW concerns impact any of the proposed project areas.

Additionally, environmental database record searches are being conducted to determine if they reveal any evidence of HTRW concerns within or adjacent to the areas of the proposed projects.

Based on the findings of the HTRW site assessment, any specific or unusual environmental concerns that are identified that would affect the construction of the proposed project will need to be addressed appropriately.

It should be noted that all surficial environmental evaluations made during the above described site visits are limited due to the fact that subsurface conditions were not field investigated as part of the HTRW assessment and may differ from the conditions implied by the surficial observations.

These proposed project areas have been severely impacted by hurricane driven storm water and winds. The potential for contamination resulting from the deposition of chemicals or petroleum products from hurricane damaged area businesses and industrial operations exist. Any such chemicals or petroleum products would likely have found their way to area canals, creeks, rivers and drainage ways.

Prior to removal, sediment from these drainage ways would need to be tested for contamination before being placed in designated disposal areas.

Alternatives

Alternatives available for improving the condition are listed below.

- ***Alternative 1: High Flow Diversion.*** This alternative is a short term alternative that would consist of removing approximately 3-5 ft of material over an area of 7.4 acres as shown in Figure 5. The project would allow high flow from Franklin Creek Tributary to spill into the adjacent swamp draining into Grand Bay, thus reducing the flow entering Franklin Creek.

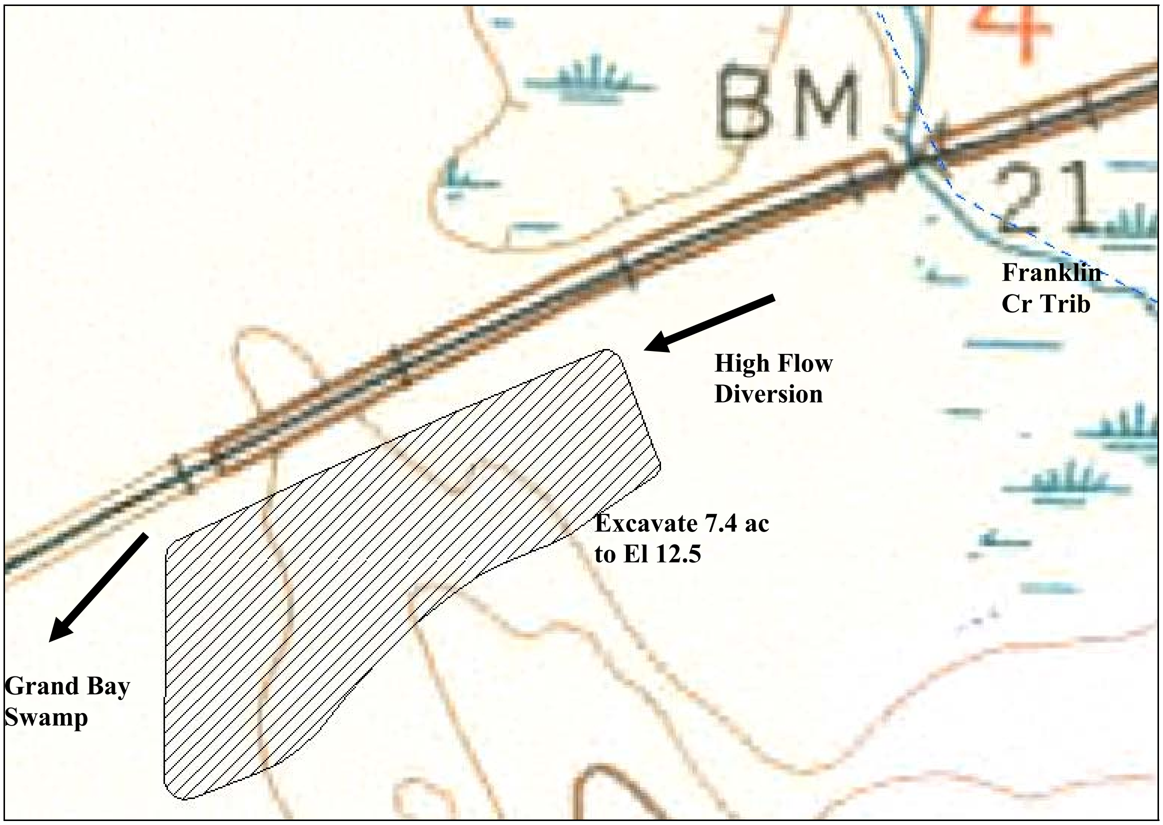


Figure 7. High Flow Diversion Plan

This alternative is anticipated to result in moderate benefits.

- ***Alternative 2: Bridge Relief Openings.*** This alternative would consist of constructing additional railroad bridge relief openings to allow high water on the north side of the railroad to move southward towards Grand Bay. The elevation of the railroad is approximately 25 ft NGVD and the ground elevation is approximately 10 ft NGVD, so the bridge would be approximately 15 ft high. There would be four bridges, each 300 ft in length for a total length of 1200 ft. The location of the bridges is shown below.

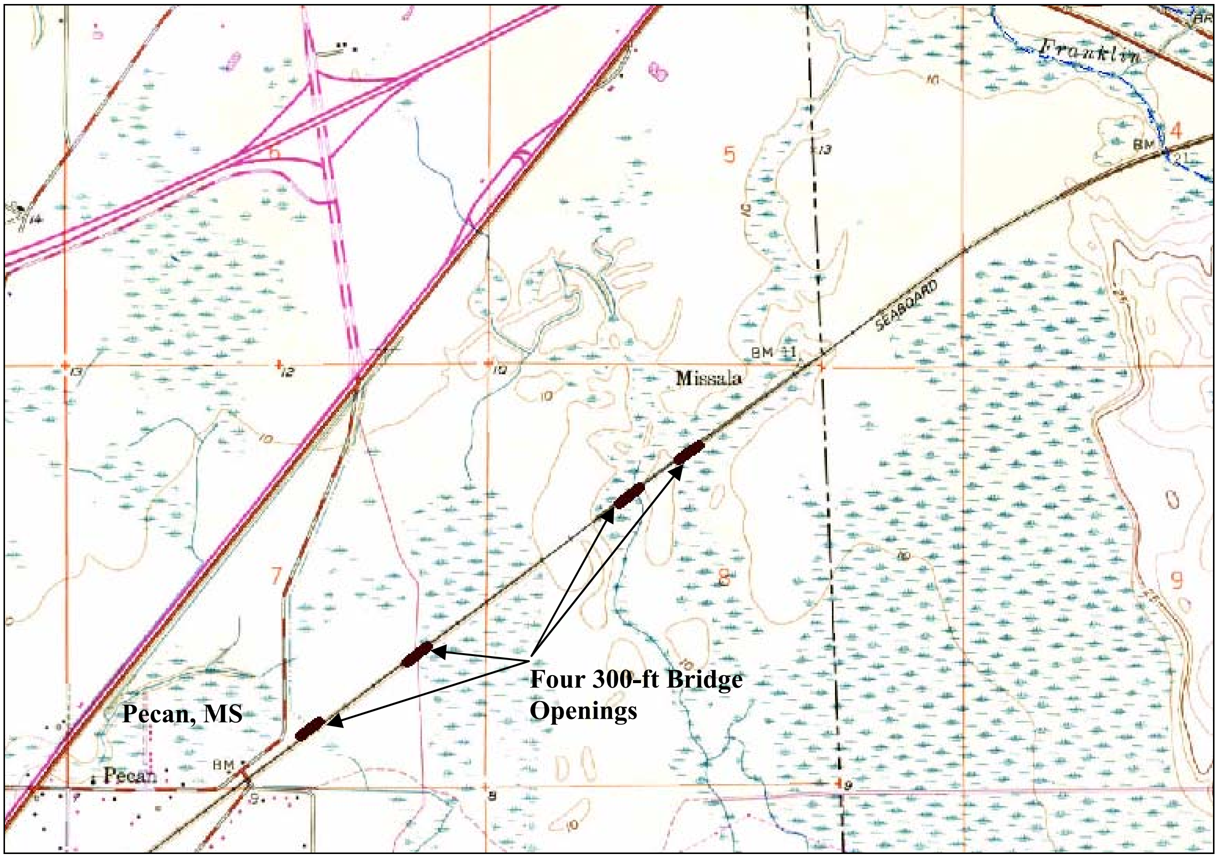


Figure 8. Franklin Creek Alternative 2

The bridges would be standard timber piling bridges with a top of rail to low steel distance of approximately 4 ft thick. A typical timber railroad bridge in another part of the country is shown below.

Figure 9. Franklin Creek - Alternative 2 - Typical Timber Railroad Bridge

This alternative is anticipated to result in moderate benefits.

- ***Alternative 3: Buyout.*** The above structural solutions have the risk of not being able to guarantee a high level of protection to the community of Pecan because the community is very low-lying, and very flat. Existing channels are inadequate to drain the community, even in the event of upstream drainage re-routing (Alternative 1). The structural alternatives examined will also not prevent inundation from high storm surges, which leaves the community at some risk from certain events.

A non-structural plan of purchase and removal that cost less than any structural solution providing the same level of protection would provide permanent relief.

The implementation of Alternative 1 could be a part of the buyout plan and would still serve to re-route flows into Grand Bay Marsh, providing for restoration of overland flow into that area.

Construction Procedures and Water Control Plan—Alternative 1

Construction would be done by using mechanical excavation equipment and dump trucks. Material could be hauled to a land fill area. No water control would be required.

Project Security—Alternative 1

This project will not incorporate any components that might be considered targets for terrorist or other attack and should require no added measures for protection against such actions.

Operations and Maintenance—Alternative 1

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Shoaling is expected to be minimal except in the event of a rare hurricane event. Maintenance costs are included in this report.

Construction Procedures and Water Control Plan—Alternative 2

Construction would be done by either building the replacement bridges simultaneously to avoid shutdown of the traffic for long periods of time, or by constructing temporary by-pass rail diversions while construction is in progress. Water control is not anticipated to be a problem in either case.

Project Security—Alternative 2

This project incorporates four bridges that might be considered targets for terrorist or other attack. Some additional monitoring of these bridges should be considered.

Operations and Maintenance—Alternative 2

Operation and maintenance activities for this project will be minimal and will include only periodic visual inspection. Maintenance costs are included in this report.

Table 5.
Franklin Creek - Alternative 1 Initial Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Franklin Creek	ITEM NO.	1	DATE	26-Jun-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***High Flow Diversion***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	franklin creek6-26xls		

DESCRIPTION			Quantity	Unit	Unit Price	ESTIMATED AMOUNT
High Flow Diversion						
Mobilization, Preparatory Work, Demobilization			1	job	allow	40,000
Excavation			47,800	cy	10.00	478,000
Clear/Grub (assume 10% c/d debris)			7	ac	6,000.00	42,000
grassing			7	ac	3500.00	24,500
access road 30'x 1 mile			1	ls	30000.00	30,000
Misc. Site Items			1	ls	allow	20,000
	Total Direct Construction Cost					$634,500
	Indirect Cost	@			**15%**	95,175
						729,675
	Profit	@			**9%**	65,671
						795,346
	Bond	@			**1.5%**	11,930
09 Account, Channels and Canals		**Current Contract Cost, Oct 06**				$807,276
	01 Account, Lands & Damage (PCA)				**LS**	225,000
						1,032,276
	30 Account, Plan, Engr.& Design				**10%**	103,228
						1,135,504
	31 Account, Constr. Management				**6%**	68,130
						1,203,634
	CONTINGENCY				**20%**	195,727
						1,399,360
						$1,399,360
						rounded
	TOTAL PROJECT COST, FY-07					**$1,400,000**

Alternative 1—Maintenance Cost Estimate

It is estimated that maintenance clearing will be required every 5 years at the cost shown below.

Table 6.
Alternative 1 Maintenance Cost Estimate

PROJECT: Coastal MS Study, Franklin Creek	ITEM NO.	1	DATE	22-Apr-06
LOCATION: Jackson County MS.	SHEET NO.	2	OF	3
	PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM: ***High Flow Diversion***	BASIS of ESTIMATE:			info furnished per PDT Team
	FILE NAME:	o-m franklin creek4-22.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
High Flow Diversion				
Mobilization, Preparatory Work, Demobilization	1	job	allow	10,000
Clear	7	ac	10,000.00	70,000
Total Direct Construction Cost				$80,000
Indirect Cost	@		**15%**	12,000
				92,000
Profit	@		**9%**	8,280
				100,280
Bond	@		**1.5%**	1,504
Current Contract Cost, Oct 06				$101,784
01 Account, Lands & Damage (PCA)			**LS**	0
				101,784
30 Account, Plan, Engr.& Design			**10%**	10,178
				111,963
31 Account, Constr. Management			**6%**	6,718
				118,680
CONTINGENCY			**20%**	23,736
				142,416
ESCALATION, FY-07			**1%**	1,424
				$143,841
				rounded
TOTAL PROJECT COST, FY-07				**$140,000**

Table 7.
Alternative 2 Initial Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Franklin Creek	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Bridge Modification***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	franklin creek6-26xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Bridge Modification				
Mobilization, Preparatory Work, Demobilization	1	job	allow	300,000
new railroad bridge	1,200	lf	2,000.00	2,400,000
Misc. Site Items	1	ls	allow	150,000
	Total Direct Construction Cost			$2,850,000
	Indirect Cost	@	15%	427,500
				3,277,500
	Profit	@	9%	294,975
				3,572,475
	Bond	@	1.5%	53,587
08 Account, Roads, Railroads & Bridges		Current Contract Cost, Oct 06		$3,626,062
	01 Account, Lands & Damage (PCA)	LS		225,000
				3,851,062
	30 Account, Plan, Engr.& Design	10%		385,106
				4,236,168
	31 Account, Constr. Management	6%		254,170
				4,490,338
	CONTINGENCY	20%		853,068
				5,343,406
				$5,343,406
				rounded
	TOTAL PROJECT COST, FY-07			**$5,340,000**

Alternative 2—Maintenance Cost Estimate

It is estimated that annual maintenance will be required at 3% of the initial cost.

Table 8.
Alternative 2 Maintenance Cost Estimate

PROJECT:	Coastal MS Study, Franklin Creek	ITEM NO.	1	DATE	22-Apr-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Bridge Modification***	BASIS of ESTIMATE:			info furnished per PDT Team
		FILE NAME:	o-m franklin creek4-22.xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Bridge Modification				
assume 3% of initial construction cost of $3,700,000 per year	3,700,000	job	0.03	111,000
				rounded
	TOTAL PROJECT COST, FY-07			**$110,000**

Table 9.
Alternative 3 Initial Cost Estimate

PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Coastal MS Study, Franklin Creek	ITEM NO.	1	DATE	28-Jul-06
LOCATION:	Jackson County MS.	SHEET NO.	2	OF	3
		PREPARED:	Parmer	CHECKED:	Ellsworth
WORK ITEM:	***Buyout***	BASIS of ESTIMATE:			Info furnished per PDT Team
		FILE NAME:	franklin creek6-26xls		

DESCRIPTION	Quantity	Unit	Unit Price	ESTIMATED AMOUNT
Buyout				
Acquire single family houses (1400sf - 1 lot)	24	ea	75000.00	1,800,000
Acquire mobile homes (900sf - 1 lot)	6	ea	50,000.00	300,000
Demolish/Remove houses	24	ea	5000.00	120,000
Demolish/Remove mobile homes	6	ea	1000.00	6,000
Misc. Site Items	1	ls	allow	15,000
01 Account, Lands & Damages	Total Direct Construction Cost			$2,241,000
	Indirect Cost	@	15%	336,150
				2,577,150
	Profit	@	9%	231,944
				2,809,094
	Bond	@	1.5%	42,136
	Current Contract Cost, Oct 06			$2,851,230
	01 Account, Lands & Damage (PCA)		LS	210,000
				3,061,230
	30 Account, Plan, Engr.& Design		8%	244,898
				3,306,128
	31 Account, Constr. Management		6%	198,368
				3,504,496
	CONTINGENCY		20%	658,899
				4,163,395
				$4,163,395
				rounded
	TOTAL PROJECT COST, FY-07			**$4,160,000**

Table 10.
Typical Schedule for P&S

Draft P&S	**3 months after start**
ITR/BCOE review	1 week after draft P&S
Final P&S/RTA	1 week after ITR/BCOE
Advertise	2 weeks after RTA
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NOAA Preliminary Report Hurricane Storm Tide Summary (NOAA, 2005b)