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Mississippi Coastal Improvements Program (MsCIP) Hancock, Harrison, and Jackson Counties, Mississippi

Comprehensive Plan and Integrated Programmatic Environmental Impact Statement

Volume 5 - Appendix E: Engineering

MsCIP

THE GREAT SEAL OF THE STATE OF MISSISSIPPI

FOREWORD

This document is one of a number of technical appendices to the Mississippi Coastal Improvements Program (MsCIP) Comprehensive Plan and Integrated Feasibility Report and Environmental Impact Statement.

The Mississippi Coastal Improvements Program (MsCIP) Comprehensive Plan Integrated Feasibility Report and Environmental Impact Statement provides systems-based solutions and recommendations that address: hurricane and storm damage reduction, ecosystem restoration and fish and wildlife preservation, reduction of damaging saltwater intrusion, and reduction of coastal erosion. The recommendations contained in the Main Report/EIS also provide measures that aid in: greater coastal environmental and societal resiliency, regional economic re-development, and measures to reduce long-term risk to the public and property, as a consequence of hurricanes and coastal storms. The recommendations cover a comprehensive package of projects and activities, that treat the environment, wildlife, and people, as an integrated system that requires a multi-tiered and phased approach to recovery and risk reduction, irrespective of implementation authority or agency.

The MsCIP Study Area

The purpose of the Comprehensive Plan Report is to present, to the Congress of the United States, the second of two packages of recommendations (i.e., the first being the “interim” recommendations funded in May 2007, and this “final” response, as directed by the Congress), directed at recovery of vital water and related land resources damaged by the hurricanes of 2005, and development of recommendations for long-term risk reduction and community and environmental resiliency, within the three-county, approximately 70 mile-long coastal zone, including Mississippi Sound and its barrier islands, of the State of Mississippi.

This appendix, the Main Report/EIS, and all other appendices and supporting documentation, were subject to Independent Technical Review (ITR) and an External Peer Review (EPR). Both review processes will have been conducted in accordance with the Corps "Peer Review of Decision Documents" process, has been reviewed by Corps staff outside the originating office, conducted by a Regional and national team of experts in the field, and coordinated by the National Center of Expertise in Hurricane and Storm Damage Protection, North Atlantic Division, U.S. Army Corps of Engineers.

The report presents background on the counties that comprise the Mississippi coastline most severely impacted by the Hurricanes of 2005, their pre-hurricane conditions, a summary of the effects of the 2005 hurricane season, problem areas identified by stakeholders and residents of the study area, a summary of the approach used in analyzing problems and developing recommendations directed at assisting the people of the State of Mississippi in recovery, recommended actions and projects that would assist in the recovery of the physical and human environments, and identification of further studies and immediate actions most needed in a comprehensive plan of improvements for developing a truly resilient future for coastal Mississippi.

This appendix contains detailed technical information used in the analysis of existing and future without-project conditions, in the development of problem-solving measures, and in the analysis, evaluation, comparison, screening, and selection of alternative plans, currently presented as tentatively-selected recommendations contained in the Main Report/EIS.

Each appendix functions as a complete technical document, but is meant to support one particular aspect of the feasibility study process. However, because of the complexity of the plan formulation process used in this planning study, the information contained herein should not be used without parallel consideration and integration of all other appendices, and the Main Report/EIS that summarizes all findings and recommendations.

This appendix, The Engineering Appendix, contains detailed supporting data and technical information on the many engineering options that were considered as possible measures that could be used in the Comprehensive Plan. Each option can be used as a stand-alone measure or in combination with other engineering options, environmental measures or non-structural programs in the development of alternatives for the Comprehensive Plan.

EXECUTIVE SUMMARY

Hurricanes are commonly recurring hazards for 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. Development along the Mississippi coastline with relatively low elevations in many areas has created a landscape that is highly susceptible to storm damage. Two bays that divide the coastlines of the three counties also aggravate the potential for inland flooding due to storm surge. The influence that landfall location for hurricanes may impart on storm surge is based on physical reasons and dictates why western Mississippi might register higher stages for a given hurricane than elsewhere along the Mississippi Coast. While the central coast of Mississippi has the highest topography, major hurricanes such as Camille in 1969 and Katrina in 2005 still produced surges that devastated this highly developed area. The area that was completely inundated due the storm surge associated with Hurricane Katrina is shown in Figure ES-1. Approximately half of the coast of Mississippi including all of Harrison County has man-made beaches with high-value real estate immediately landward of the beaches. Essentially all of the structures facing the Mississippi Sound were completely destroyed in Katrina.

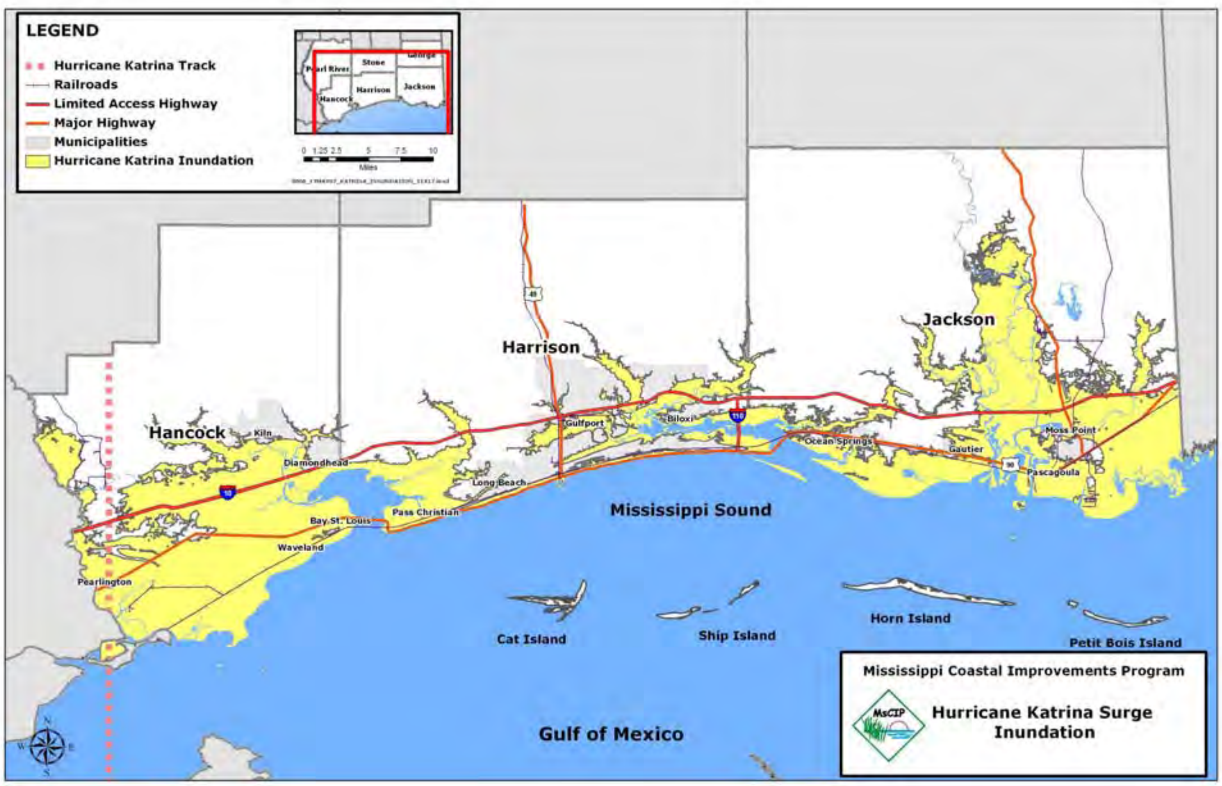


Figure ES-1. Inundated Areas of Coastal Mississippi from Hurricane Katrina Storm Surge

The Mississippi coast and its offshore chain of barrier islands 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. Studies also show that all of the barrier islands are losing surface area due to erosion caused by a number of factors including the impacts of major storms.

Sea level rise and land surface subsidence have been taken into account as part of this study and is reported as “relative sea level rise” which accounts for both as a single value. The Intergovernmental Panel for Climate Change (IPCC) ‘high’ values were selected for evaluating project performance as the ‘higher than observed rate’ versus those predicted using EPA and NRC methods because the IPCC values are more recent and more widely (globally) used. In a subtle departure from USACE guidance, relative sea level rise values based on IPCC ‘expected’ (also referred to as ‘medium’ and ‘central value’) eustatic sea level rise predictions were adopted in lieu of rise computed using extrapolated historic rates because most experts believe that the rate of sea level rise will increase in this century and extrapolated historic rise assumes past relative sea level rise rates will persist.

With the task of developing a comprehensive hurricane damage reduction plan for the coast of Mississippi, several issues had to be considered. First, it had to be technically feasible. The storm damage reduction system must be designed such that it would be effective and at the same time not destroy what it was supposed to help protect? It had to be reliable so when needed, it would do the job it was designed for. It also needed to be cost effective. This system also had to be integrated into other storm reduction concepts such as non-structural solutions and buy-out programs. It must also include re-establishing some wetland areas as environmental components of the plan. The development along the coast had some areas that were not contiguous to highly developed areas like found in Harrison County where the entire coastline is densely developed. These outlying areas will require individual means for any storm damage reduction. Almost any project along a coastline has environmental concerns and this is true in Mississippi. In Jackson County, the Pascagoula River system separates the city of Pascagoula from most of the coast to the west. This river system with its vast marshes areas is one of the last major free-flowing rivers in the southeast and is home to an endangered fish species. In the western portion of the state, extensive marshes create other concerns along with the Pearl River that separates Mississippi from Louisiana. Other technical issues also made working in this river problematic.

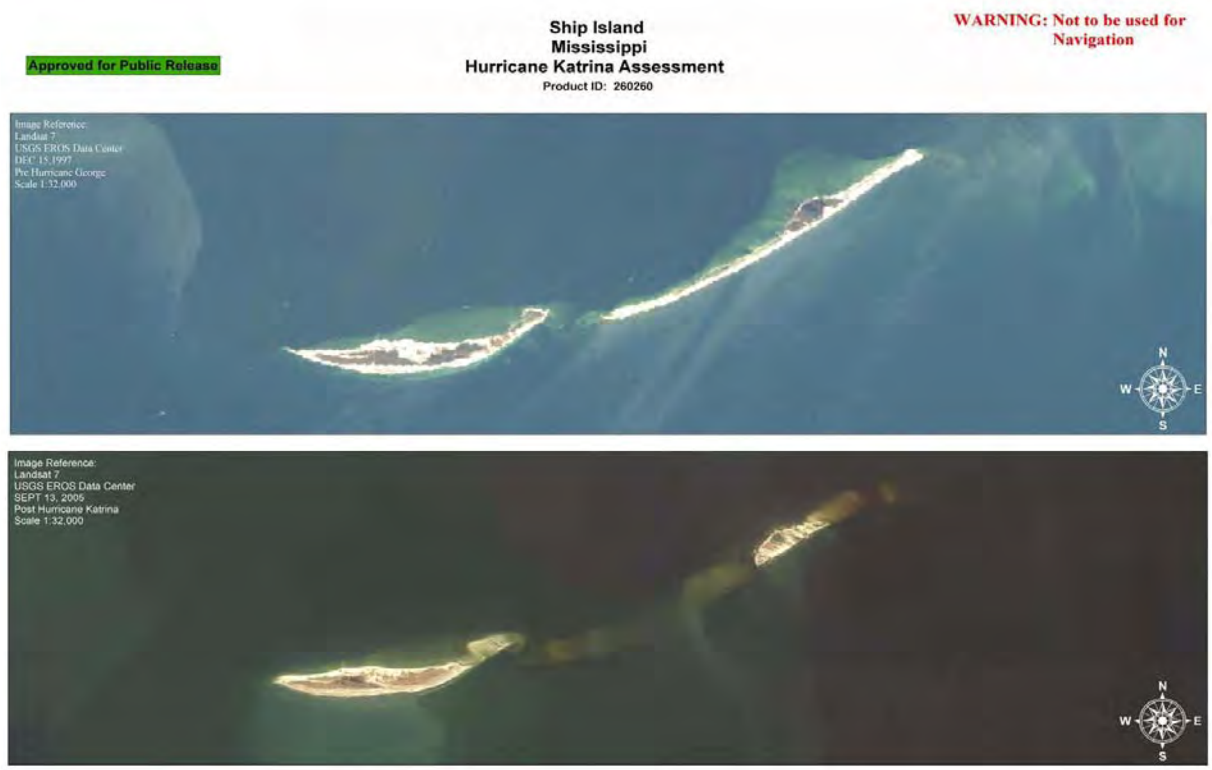
Review of the coastline in Mississippi using aerial photographs, topographic maps, LIDAR surveys, and storm inundation data revealed that natural topography could play a major role in forming storm barriers. Other features such as the offshore barrier islands, extensive beaches in many areas, and existing beach-front roadways were also realized as having a role in formulating a storm defense system. An existing railway track crosses the entire state near the coast and in the typical fashion of railways, these tracks follow high ground. This same general alignment was judged to be favorable for any type of inland barrier.

Review of the inundation maps from Katrina also revealed the extensive low-lying areas associated with two bays that extend inland from the coast. It was apparent that any storm protection systems would have to consider these as breaks in the line. Closing off rivers and bays with surge gates has been used in Europe to protect inland areas and these type structures have been considered for Mississippi.

During planning sessions with the project delivery team, a structural “Lines of Defense” concept was drafted that started with the offshore barrier islands and progressed inland to what could be considered the worst possible scenario with a extremely large hurricane, even worse than Katrina. Research identified numerous methods that have been developed to provide protection from storm surge. Along with the traditional methods of levee or structural seawall construction, many other types of protection were reviewed. These included inflatable barriers, concrete sidewalks or roadways that could be hydraulically rotated upwards to form a seawall, sliding panel gates, offshore breakwaters, and many types of surge barriers to close off the bays. The lines would also provide increasing levels of protection as you transgressed inland. It was understood that some lines would not provide protection from large storms. It was also evident that several areas of the coast could not be included in continuous line of defense and would be either placed in a ring levee system or designated to a non-structural solution.

From the planning session came five conceptual lines of defense. The general concept for this plan was made in a project team meeting that included engineers, environmentalists, planners, and geologists. Information from along the coastline was gathered that included large scale aerial photography, topographic maps, navigation maps, and a large collection of pre and post-Katrina photographs.

The first apparent feature to be considered was the offshore barrier islands that had been included in the Mississippi Governor's Hurricane Recovery Plan. Designated as Line of Defense (LOD) 1, the barrier islands have been eroded by numerous storms. In 1969, Hurricane Camille caused extensive erosion on the islands and created a large breach in Ship Island, (see Figure ES-2). This breach began to heal from the east as the littoral drift of sand added land mass to the west end of East Ship Island. This large scale breaching occurred again during Katrina, eroding away all the sand that had collected over the previous 35 years since Hurricane Camille. The post-Camille shoreline of Ship Island was documented by the Mississippi Department of Environmental Quality. After Katrina, it was widely expressed that if the islands had been in a pre-Camille condition, the storm surge would have been much less along the mainland coast. This scenario was modeled to help predict what effects the islands play in storm reduction. There are a total of seven different options included in this report covering a wide range of possible ways to mitigate erosion of the islands.



Source – United States Geological Survey

Figure ES-2. Before and After. The aerial photograph on top shows the islands in 1997 prior to Hurricane George in 1998. The bottom photograph shows the same view of the eroded condition of East and West Ship Island after Hurricane Katrina. Prior to a breach during Hurricane Camille, Ship Island was a single island, although the island has been breached prior to Camille.

The beaches (manmade in the 1950s) that extend along much of the coast were also considered as a feature that could be modified to provide some level of protection by construction of dunes on the beaches. Other projects are underway to improve some of the beaches and proposed projects would construct small dunes on most of the beaches. Improving on these features by adding higher dunes and/or dune vegetation was designated as LOD-2. These would not provide protection from large storms, but would be beneficial for smaller storms and would provide recreational and environmental benefits. Each of the three counties has beaches that fit this scenario for adding dunes. For each county, 11 options were considered for adding some measure of dune creation. Most of the options have versions that included adding vegetation and sand fencing as well as dunes without these features. Eight of the options in each county have the dune placed against roadways that parallel the beaches with the assumption that these roadways would be elevated as a separate measure. Each of these options have a dune crest elevation less than the adjacent roadway (possibly raised in the future under LOD-3 options) to prevent sand from constantly being blown onto the road. A photo of the existing condition of the beaches and roads in Harrison County is shown in figure ES-3. These options have some value as protection for the road, but more value as an ecological benefit. Two other options include a stand-alone dune out on the beach that could provide some level of surge defense along with ecological benefits. Each county also has an option with a wide sand berm fully planted with sea oats, the preferred vegetation to help stabilize dunes. This option will allow the sea oats to trap wind-blown sand and naturally build a dune with time. The dune options in all three counties total 33 different measures that could be considered.

Figure ES-3. 2007 photograph of Biloxi Beach showing the existing beach berm and the adjacent seawall and roadway.

As mentioned above, another existing condition along much of the coast is roadways that coincide with the beaches. It was envisioned that raising these roadways would have minimal environmental impact and provide the first hardened barrier to surge damage. These roadways, while not continuous along the coast, were designated as LOD-3. The new road elevations would not be as high as to act as a seawall for very large storms, but like LOD-2, they would be beneficial for smaller, more frequent storms. While different elevations were initially considered for the roadways, the technical difficulty of raising the roads over six feet was realized. This is due to the numerous intersecting roads, driveways, and parking areas that could not be constructed without extreme grades. The existing beachfront roads in Hancock and Jackson have a typical grade elevation of 5.0 (NAVD88) and the general grade elevation for US 90 in Harrison County is 10.0 (NAVD88) although it varies from elevation 7.0 to 16.0 (NAVD88) depending on the exact location. With the existing road elevations, a top elevation of 11.0 (NAVD88) was selected for study in Hancock and Jackson County and a top elevation of 16.0 (NAVD88) was selected for study in Harrison County for a total of three options. It was also recognized that LOD-3 would require that a barrier be placed at the mouths of the bays to be effective against back-flooding.

Some areas of the coast were not associated with beaches or existing roadways that allowed for a continuous defense line. When including environmental and/or technical reasons, these areas could only be viewed as stand-alone projects such as ring levees. These areas included five communities in Jackson County and one in Hancock County. For discussion purposes, these were also included in LOD-3. Each of the conceptual ring levees have been evaluated for construction at two elevations, 20.0 and 30.0 (NAVD88). The costs also included interior drainage, pumping stations, gates for roadways and overtopping protection. Some sites also have one or more alternate alignments. The alternate alignments were selected to lessen the impacts on wetlands, lessen the intensity of wave action or to decrease the construction costs versus adding non-structural solution areas. With all ring levee elevations and alternate alignments, there are 24 different options for further consideration.

Further inland, an existing railroad grade provided a levee-like barrier to storm surge from Katrina in some areas, (see Figure ES-4). This railway extends all the way across the State crossing both St. Louis Bay and Biloxi Bay. In Harrison County, the railway parallels the coastline just a few blocks inland. Using a parallel, high-ground alignment as the railway system, an inland barrier was envisioned that could be constructed to such an elevation as to protect from a large storm surge, even larger than Katrina. Like LOD-3, this system would require that the bays be closed off with barriers from surge to be effective. As LOD-4, this barrier was studied at elevations up to the maximum storm surge or maximum possible intensity (MPI) storm that could be predicted based on simulated hurricane events. These selected elevations are 20.0, 30.0 and 40.0 (NAVD88). Possible options for LOD-4 include omitting the surge barrier across St. Louis Bay. This would require that LOD-4 be terminated o the east side of the bay. An alternate alignment to satisfy this option was selected at Menge Avenue in Pass Christian where the LOD-4 levee could be extended northward to higher ground. This option would also leave the town of Bay St. Louis without any type of surge protection. If this alternate alignment is used, Bay St. Louis hurricane defenses could be included as a ring levee with an option under LOD-3. Many alignments for project termination on the western and eastern sides of the state were considered before one that was selected, mostly due to technical and environmental reasons. This system would not cross the Pearl River on the western side of the state nor the Pascagoula River in Jackson County. Including all the different elevations and alignments for LOD-4, there are a total of 22 options including the six options for the surge gates.

Figure ES-4. The CSX Railway parallels the coast and its embankment acted as a low levee-like storm surge barrier in some areas.

As maximum protection from the largest storm surge event, the limits of surge predicted from the MPI event was transposed to maps. This location of this line was shifted as refinements were made in the storm surge modeling. While actually a non-structural measure, it was designated as LOD-5. It would be an area north of any potential surge damage that would be recommended to local governments for location of critical infrastructure such as hospitals and emergency facilities.

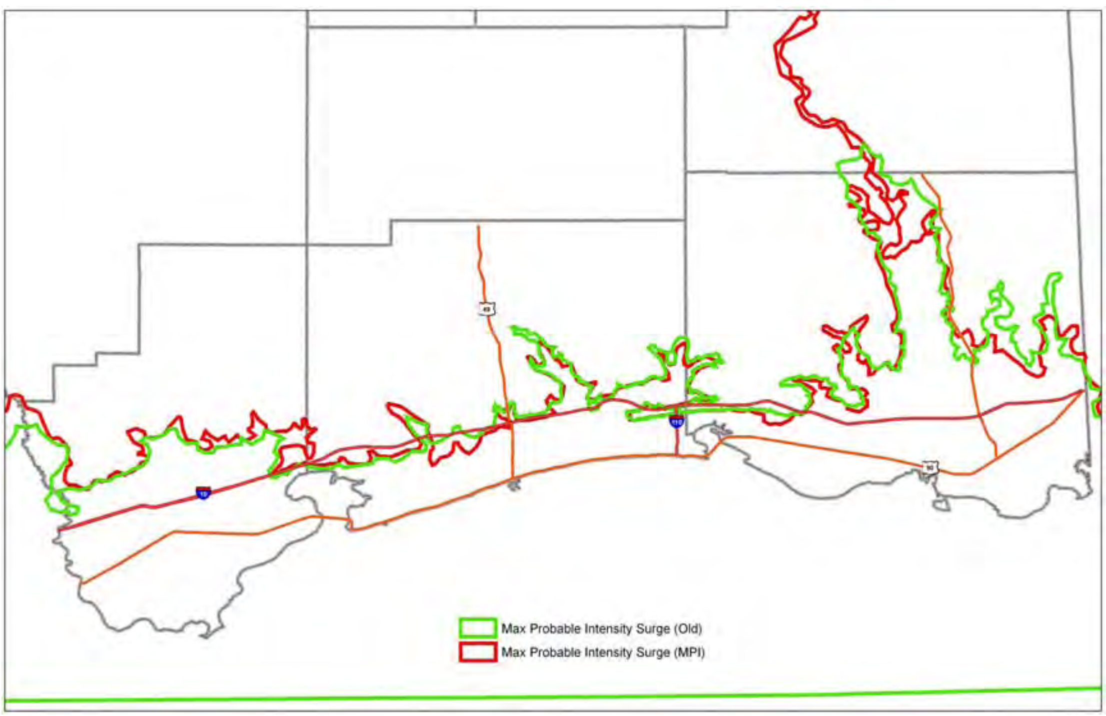


Figure ES-5. The surge limits of a computer simulated Maximum Possible Intensity hurricane based on early data and later refined modeling efforts

To proceed with initial cost estimates, various components of the structural options were conceptually designed to the selected elevations described in previous paragraphs. The initial elevations selected for each component of the lines of defense are assumed to bracket a wide range of potential storms with corresponding surge elevations. Using these preliminary designs, rough order of magnitude cost estimates were completed for each of the structural options. These cost estimates can used to develop cost curves for future use to estimate rough estimates after final design elevations are selected. With these cost curves, future studies can also select varied levels of protection based on risk assessments as well as taking into account future estimates of sea level rise.

At this phase of the plan formulation process, there were no assessments made for HTRW investigations nor remediation costs based on the vast number of properties potentially involved and the uncertainties associated with project footprints. Also, the cost of escalation will be addressed as projects are selected to proceed to feasibility level of design. The identification of a major HTRW site within a project footprint could certainly have a cost impact, but none are known to exist at this time. Likewise, depending on the time that a project is funded for further study to feasibility level, the effects of escalation could be a major factor based on fuel costs or other items that can change drastically outside the usual inflation rate.

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Figures

Tables

PART 1. GENERAL

1.1 Guidance

1.1.1 Engineer Regulations

- ER 1105-2-101, "Planning - Risk Analysis for Flood Damage Reduction", 3 January 2006
- ER 1110-1-12, "Engineering and Design - Quality Management", 1 June 1993
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- EM 1110-1-1804, “Engineering and Design - Geotechnical Investigations”, 1 January 2001
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1.2 History of Tropical Cyclones

1.2.1 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.

1.2.2 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.

1.2.3 Results

A chronological listing of all known Hurricanes to affect Mississippi from 1711 to 2005 is given in Table 1.2-1. The resultant time series is shown in Figure 1.2-1. 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.

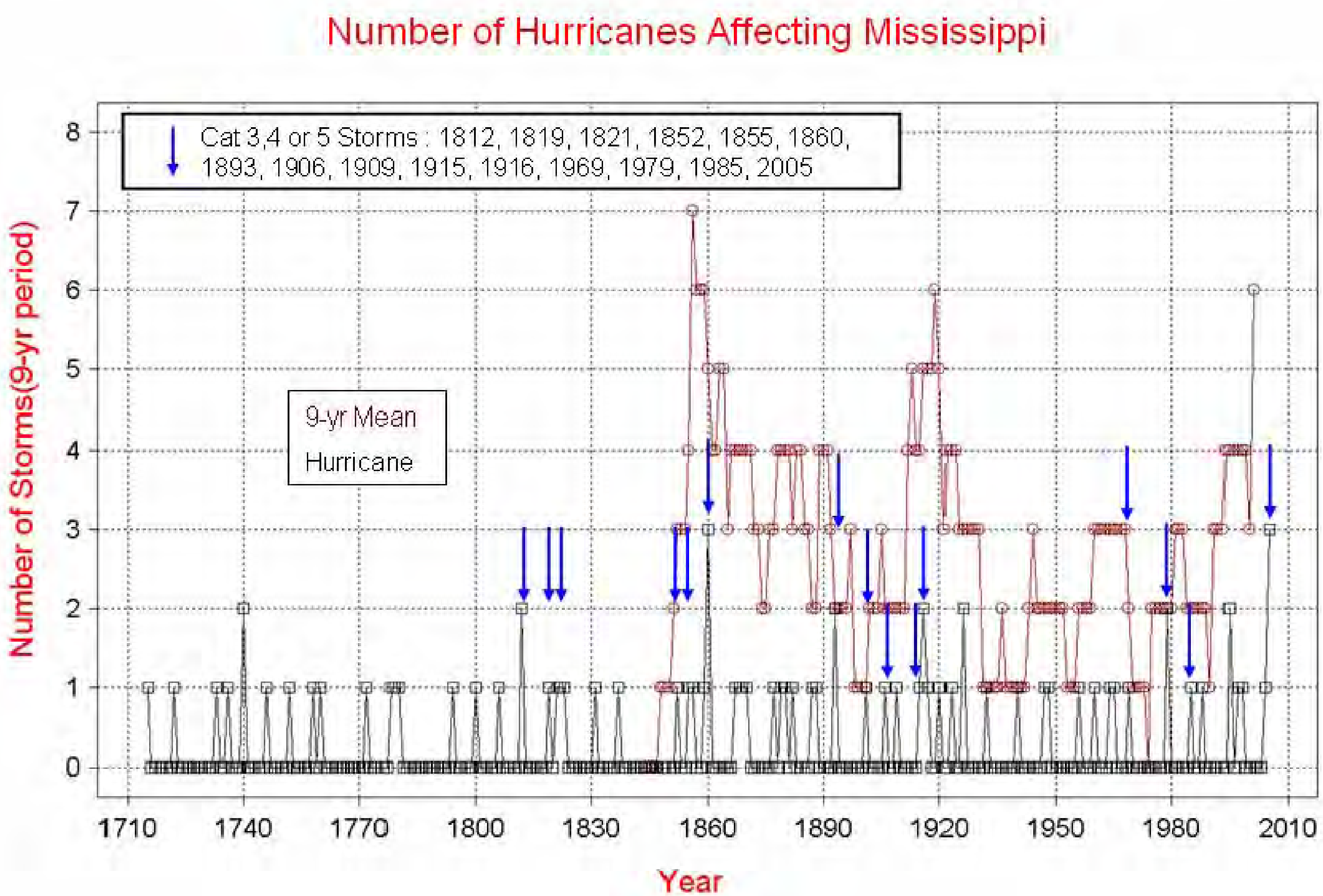


Figure 1.2-1. Hurricanes that Have Affected Mississippi

Table 1.2-1.
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
1901 Aug. 15	Gulfport	1
1906 Sept. 27	Pascagoula	3
1909 Sept. 20	New Orleans	3

Table 1.2-1.
Hurricanes Affecting Mississippi Coast (1715-2005) (continued)

Year	Landfall	Estimated Storm Category at Landfall
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

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.

1.2.4 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.

1.2.5 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.

1.3 Tide Gage Stage-Frequency Analysis

The annual percent chance exceedance stage relationship, referred to as the ‘stage-frequency curve,’ is the single most important descriptor of a community’s flood risk. The relationship describes the annual probability, expressed in percent, of a given stage (i.e. water surface elevation) being equaled or exceeded and is relied heavily upon for purposes of the National Flood Insurance Program, for the development and evaluation of flood damage reduction measures, for understanding and communicating annual and long-term risk, amongst others.

Historically, tide gage data have been used almost exclusively to describe the entirety of a given stage-frequency curve in a given coastal area. The shortcoming of this approach is that it tends to mask the true risk in the vicinity of the gage. The reasons for this are many, but perhaps the most important is related to the observation that, while the occurrence of strong hurricanes in a given coastal region is not probabilistically rare, the probability of a particular gage site taking a direct hit from one of those strong hurricanes is more rare. A more accurate representation of the true risk for severe hurricanes then can only be obtained over a long period of meteorological and water level observations (a century is not long enough) or through refined statistical analysis of storms and effects modeling efforts.

Present needs have required that a great deal of effort be placed on developing statistical methods and modeling approaches to improve our present understanding of severe hurricane risk. A Risk Assessment Group, led by scientists at the Engineer Research and Development Center (ERDC) in Vicksburg, MS, was assembled in the aftermath of Hurricane Katrina to develop such statistical and modeling methods (Ref. 1) for the Gulf of Mexico region, and those methods have been used for this program (ERDC modeling efforts are described in Chapter 2). Those efforts were focused on what might be called an extreme storm subset of the tropical storm/hurricane population. While their products and findings are many, one of their most important products was the development of 4% (1 in 25), 2% (1 in 50), 1% (1 in 100), 0.2% (1 in 500), and 0.1% (1 in 1000) annual chance stage exceedance estimates for numerous locations in the vicinity of coastal Mississippi. These estimates, combined with probabilistic analysis results of historic observed tide levels, were joined to create composite (i.e. consisting of both observed data and hydrodynamic modeling results) stage-frequency curves for planning subunits in coastal Mississippi. These in turn were used for a host of MsCIP design and evaluation efforts.

This chapter describes the available historic tide stage data and the development of that data into stage-frequency curves. The curves were compared to an historic stage-frequency curve and to ERDC model data at the location of the gage sites are displayed.

1.3.1 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. Gage locations are shown in Figure 1.3-1. 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.

Water levels recorded at the gage sites are collected in a stilling well to minimize effects from wave height and wave run-up. In cases where the tide gage was destroyed or malfunctioned, the maximum water level was obtained from 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 are accurate to +/- 0.1 foot. There is limited quality control of the tide data.

Figure 1.3-1. Mobile District Tide Gage Network

When a hurricane is forecast to strike the Gulf Coast, CESAM personnel are dispatched to remove recorded data from coastal gages and ensure that the gages are working properly. 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 dependent on the accuracy of the hurricane path and surge forecast.

Two gages were removed in Mississippi and one in Alabama on 28 August 2005, one day before the projected H. Katrina 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 CESAM gages were destroyed and 2 gages were damaged by the hurricane. Figure 1.3-2 shows the status of the gages shortly after H. Katrina.

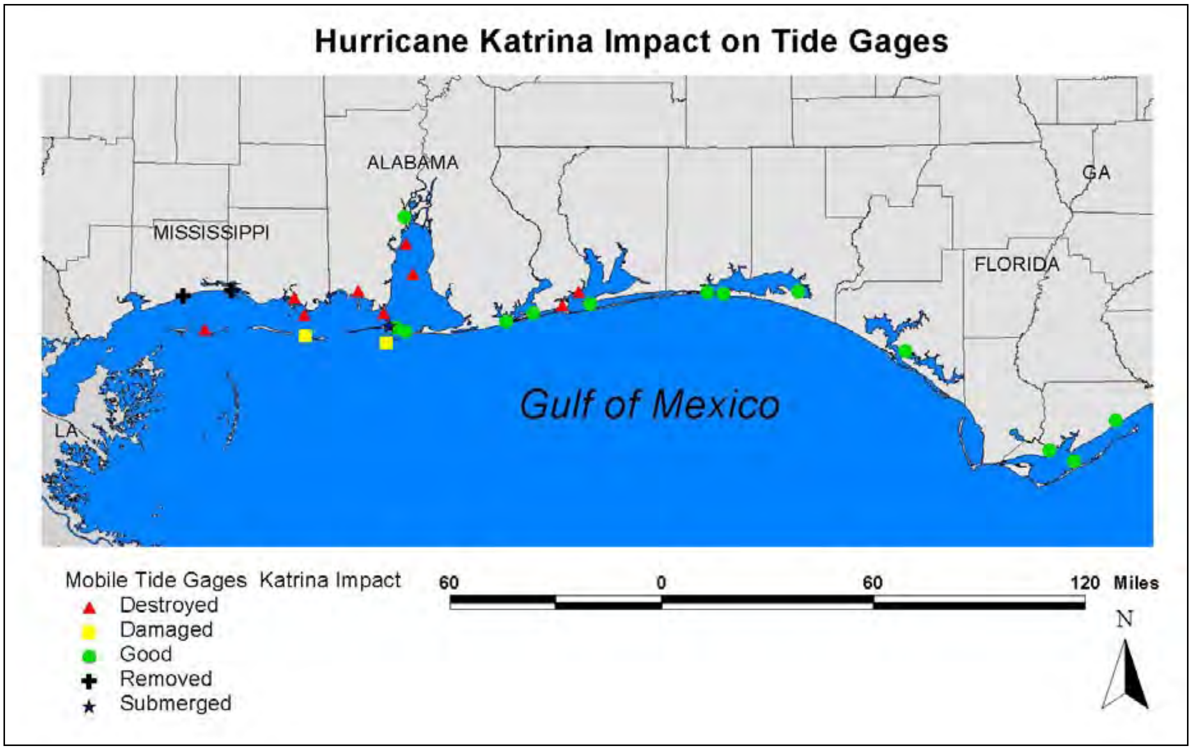


Figure 1.3-2. Hurricane Katrina Impact on Tide Gages

There are 7 active CESAM tide gages along the Mississippi Coast gages as shown in Figure 1.3-3.

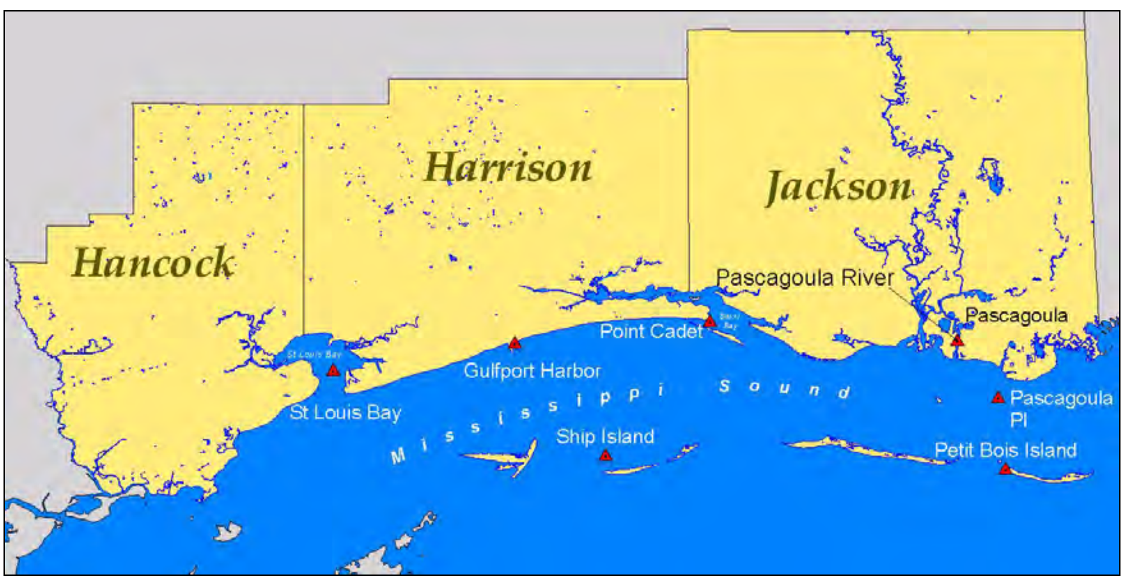


Figure 1.3-3. CESAM Mississippi Coast Tide Gages

1.3.2 Methodology

EM 1110-2-1415 (Ref. 2) recommends using graphical analysis for stage (elevation) frequency computations. 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 (Ref. 3). The median plotting position formula was selected to derive probabilistic plotting positions 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 months of recorded data; in most cases this is due to a gage malfunction or damage from a storm event. The data set includes the effects of subsidence and sea level rise and no attempts have been made to adjust the data to account for these factors. Of these, subsidence is more important in that it affects the datum of the gage and thus the absolute water surface elevation estimate. Future analysis by this office will research the necessary adjustments. Each of the three 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 documented 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. The historic record of annual maximum stages is shown in Table 1.3-1 and presented graphically in Figures 1.3-4 through 1.3-6.

1.3.2.1 Presentation of Data

Table 1.3-1.
Mississippi Coast Historic Annual Stages at Mobile District Tide Gages

		Gulfport (1963)			Pascagoula (1940)			Biloxi (1882)		
Storm	Date	Gage Height, ft.	ft. NAVD		Gage Height, ft.	ft. NAVD		Gage Height, ft.	ft. NAVD	
Sep 1882	9/10/1882								2.42	
27Sep1906	1906-Sep-27								6.05	
20Sep1909	1909-Sep-20							10.43	4.48	
12Aug1911	1911-Aug-12								4.49	
14Sep1912	1912-Sep-14								3.51	
29Sep1915	1915-Sep-29		9.13	1					9.05	
05Jul1916	1916-Jul-05								4.20	
28Sep1917	1917-Sep-28							8.61	2.66	
21Sep1920	1920-Sep-21								5.57	

		Gulfport (1963)			Pascagoula (1940)			Biloxi (1882)		
Storm	**Date**	**Gage Height, ft.**	**ft. NAVD**		**Gage Height, ft.**	**ft. NAVD**		**Gage Height, ft.**	**ft. NAVD**	
15Oct1923	1923-Oct-15							11.96	6.01	7
21Sep1926	1926-Sep-21		6.13	1					3.95	
Sep 1932	1932-Sep							9.16	3.21	
Oct 1932	1932-Oct							9.33	3.38	
July 1933	1933-Jul							9.16	3.21	
Sep 1933	1933-Sep							9.74	3.79	
Jun 1934	1934-Jun							8.98	3.03	
T.S. Jun 1939	1939-Jun							9.05	3.10	
26Sep1939	1939-Sep-29							9.5	3.55	
	1940-Aug-06					3.71		10.4	4.45	
12Sep1941	1941-Sep-12					3.38		9.52	3.57	
06Sep1945	1945-Sep-06						5	9.1	3.15	
	1947-Sep-08					2.68				6
19Sep1947	1947-Sep-19		14.13	1		7.48	2,6	16.88	10.93	2,6
04Sep1948	1948-Sep-04		6.13	1		4.08			5.73	
	1949-Sep-04					3.98			4.59	
Baker	1950-Aug-30					3.73			3.66	
Barbara	1954-Jul-29					2.43		9.1	3.15	
Brenda	1955-Aug-01					3.18			4.00	
26Aug1955	1955-Aug-26		6.13	1		2.83			3.67	
	1956-Jun-13					3.48		10.78	4.83	
Flossy	1956-Sep-24		4.13	1		3.18		9.39	3.44	
Audrey	1957-Jun-27					3.36			3.75	
T.S Ester	1957-Sep-18		6.63	1		2.63			4.77	
Ethel	1960-Sep-15		5.13	1		4.58			5.25	
Helda	1964-Oct-04	5.14	4.27			4.13			4.76	
Betsy	1965-Sep-09		10.83	2,7		6.48		14.64	8.69	
Debbie	1965-Sep-29	6.8	3.93			2.92				6
Camille	1969-Aug-17		19.81	2	11.37	11.33	2		15.69	2
Felice	1970-Sep-15	3.01	3.14		2.43	2.39		8.94	2.99	
Fern	1971-Sep-05	2.68	2.54		2.37	2.33				
Edith	1971-Sep-16	3.35	3.21		2.08	2.04			3.63	
Carmen	1974-Sep-08	4.95	4.81		3.98	3.94			4.60	
Babe	1977-Sep-06	3.9	3.76				5			5
Bob	1979-Jul-11		6.13			4.63			5.75	
Frederic	1979-Sep-12		3.43			5.86			4.03	
Elena	1985-Sep-02		5.56			5.58			6.16	
Juan	1985-Oct-28		6.63			5.39			5.96	
Bonnie	1986-Jun-23		2.73			2.45			2.83	
Gilbert	1988-Sep-08		4.90			3.10			4.06	
Florence	1988-Sep-10		4.67			3.11			6.39	
Chantal	1989-Jul-31		3.13			2.31			3.48	
Andrew	1992-Aug-26		4.02			3.18			3.90	
TS Dean	1995-Jul-28		3.70			2.83			3.52	
Erin	1995-Aug-04		2.68			2.84			3.04	

		Gulfport (1963)			Pascagoula (1940)			Biloxi (1882)		
Storm	**Date**	**Gage Height, ft.**	**ft. NAVD**		**Gage Height, ft.**	**ft. NAVD**		**Gage Height, ft.**	**ft. NAVD**	
Opal	1995-Oct-04		3.05			2.65				3
Josephine	1996-Oct-05		3.47			2.74			3.47	
Danny	1997-Jul-19		4.25			2.98			3.87	
Earl	1998-Sep-02		3.30			3.16		3.52	3.00	
Georges	1998-Sep-28		7.18			8.44	2		8.18	
T.S. Helen	2000-Nov-24		3.75			3.08			3.48	
T.S. Allison	2001-Jun-11		4.56			3.98				5
T.D. Edward	2002-Sep-06		4.13		4.09	3.45			3.57	
T.S. Hanna	2002-Sep-14	5.14	4.65		4.64	4.00			4.16	
Isidore	2002-Sep-26	8.26	7.77			5.83			6.99	
Lili	2002-Oct-04	3.79	3.30			3.96			4.88	
T.S. Bill	2003-Jul-10	4.6	4.11			3.41			4.12	
Ivan	2004-Sep-16	5.28	4.79			6.80	4		4.36	
T.S. Matthew	2004-Oct-10	4.88	4.39		3.66	3.02		4.32	3.80	
T.S. Cindy	2005-Jul-06	6.16	5.67			5.83			5.97	
Dennis	2005-Jul-10	3.63	3.14			3.33			2.99	
Katrina	2005-Aug-29		24.30	4		16.68	2		23.93	4
Storm Count			45			51			65	

1

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

2

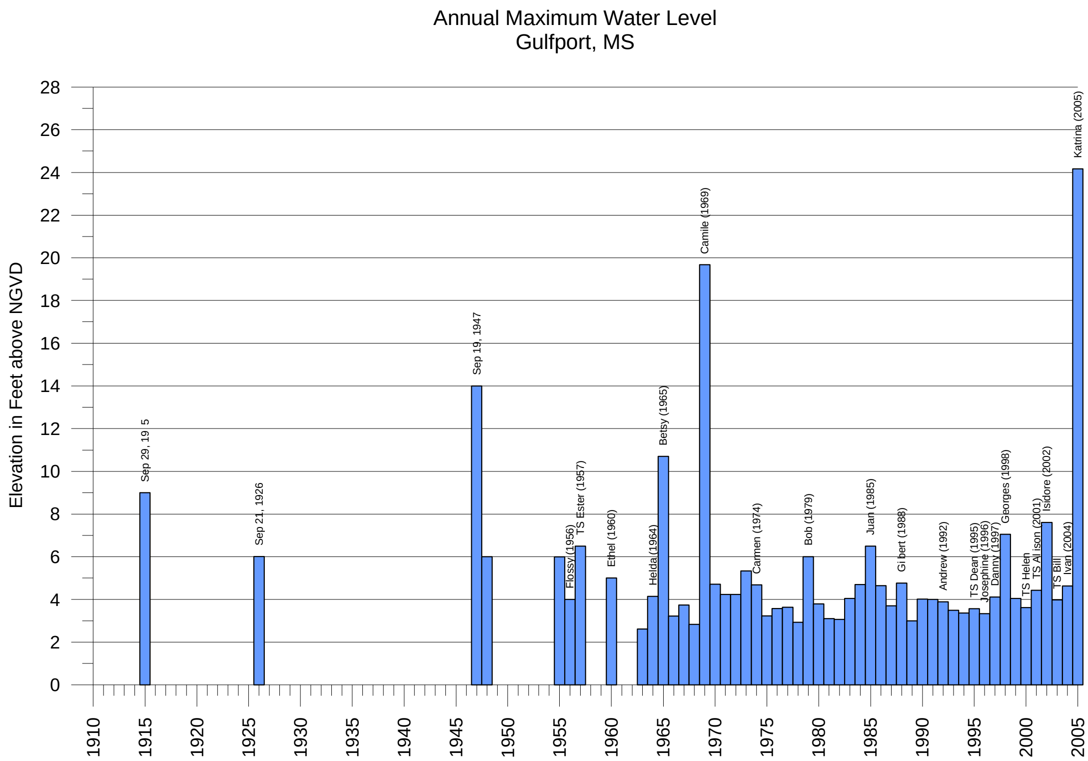


Figure 1.3-4. Gulfport, MS Annual Maximum Water Level

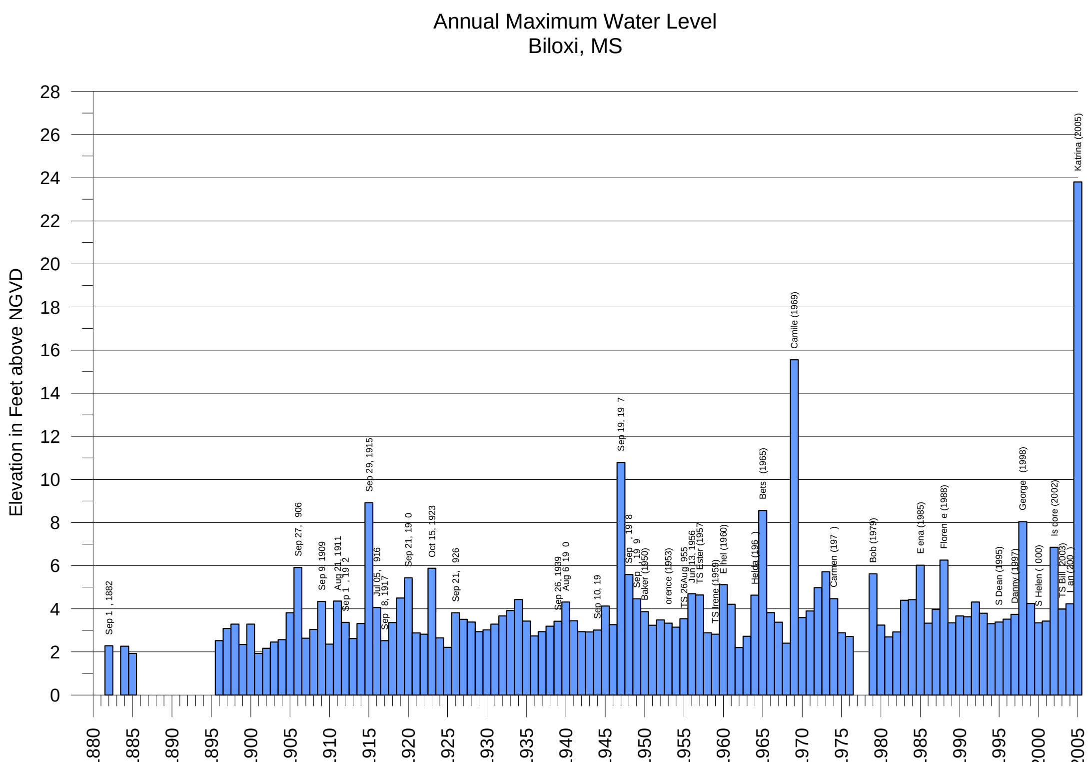


Figure 1.3-5. Biloxi, MS Annual Maximum Water Level

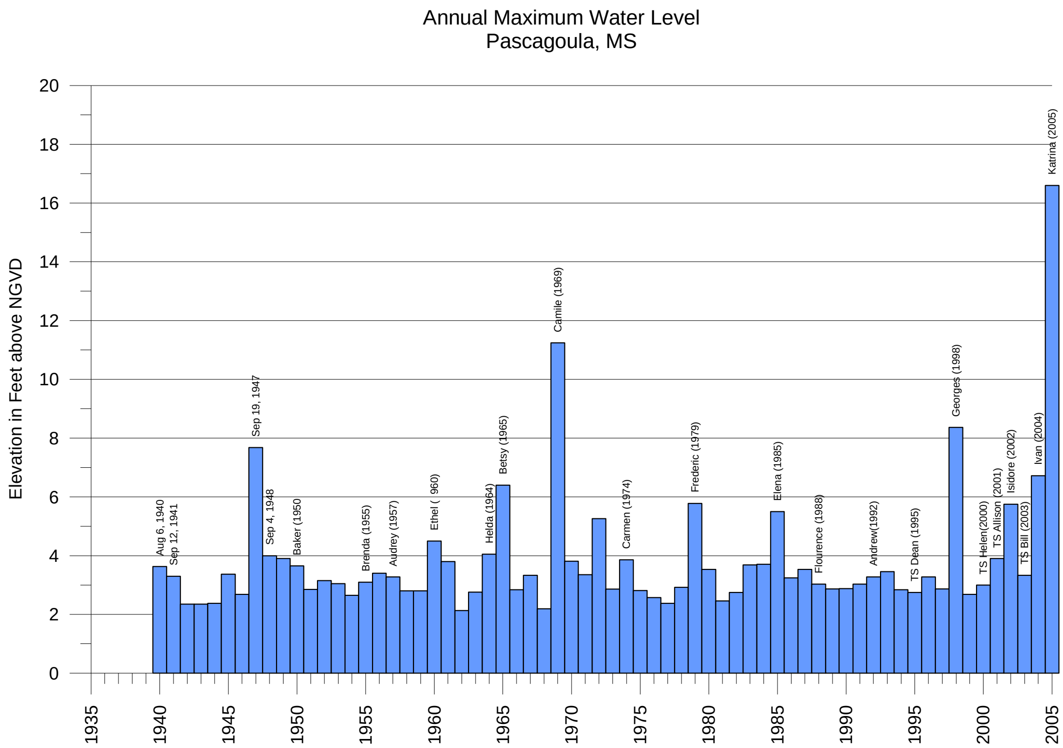


Figure 1.3-6. Pascagoula, MS Annual Maximum Water Level

1.3.3 Results

1.3.3.1 Graphical Stage-Frequency Analysis

A graphically fit (by eye) curve was drawn through the median plotting positions of the historic data for each gage site. Results for selected annual probabilities of occurrence are shown in Table 1.3-2. Comprehensive results are shown in tabular format with observed data in Tables 1.3-3 through 1.3-5. The computed Weibull plotting position is shown in those tables for reference only. Figures 1.3-7 through 1.3-9 show results presented graphically against an historic stage frequency curve. The historic curve (shown in red on the figures) was developed to represent the entire Mississippi Coast and published in a Mississippi Coast hurricane survey published by Mobile District in 1965 (Ref. 4). The hurricane survey curve was developed based on observed tidal data. That curve pre-dates some of the most intense surge-producing hurricanes to have struck the vicinity of Mississippi in the modern record: H. Betsy (1965), H. Camille (1969), H. Georges (1998), and H. Katrina (2005). The result is that, in the 40 years of record, one's impression of what the 1 in 100 chance annual stage might be according to these methods has increased dramatically, and at Gulfport that stage has nearly doubled. This observation reinforces the idea that the length of period of record is an important consideration, and that just a few historically significant events can dramatically impact the risk picture. Similarly, the tabulated results in Table 1.3-2 clearly show the influence that landfall location may impart on the stage frequency curve. While there are physical reasons why western Mississippi might register higher stages for a given hurricane than elsewhere along the Mississippi Coast, if H. Camille and H. Katrina landed more centrally there, the stage-frequency relationship would likely have been somewhat more uniform for low annual chance events at the three gages. This also demonstrates the need to combine gage data with statistical and modeling efforts to improve stage-frequency estimates.

Table 1.3-2.
Results from Graphical Frequency Analysis

Annual Percent Chance Exceedance	Pascagoula Stage	Biloxi Stage	Gulfport Stage
50	3.3	3.7	4.3
20	4.0	4.5	6.1
10	6.0	5.7	6.9
5	7.9	7.6	9.4
2	12.5	12.6	18.8
1	17.1	19.1	23.1

Period of record: Pascagoula 1916-2005, Biloxi 1882-2005, Gulfport 1941-2005.

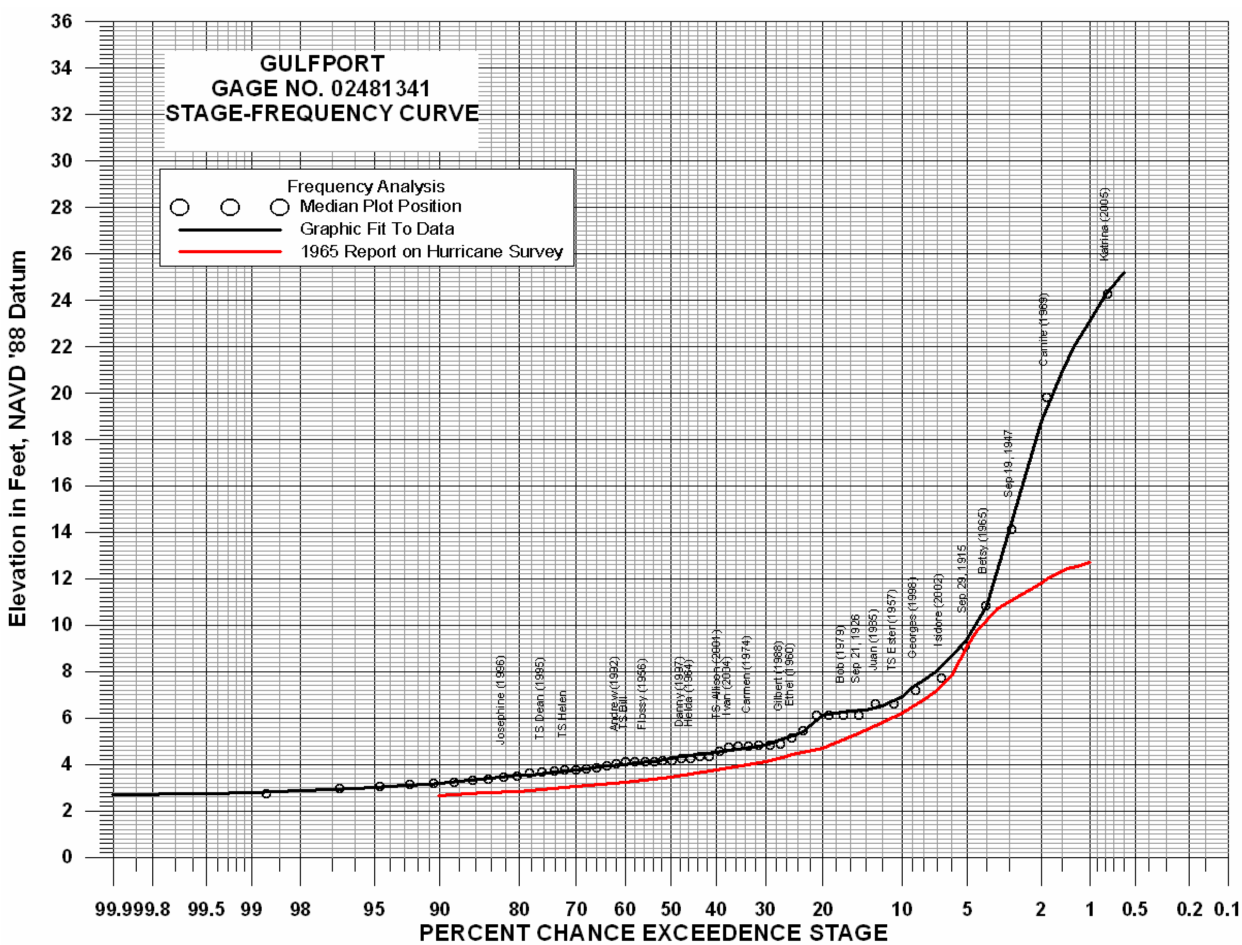


Figure 1.3-7. Gulfport, MS Frequency Curve

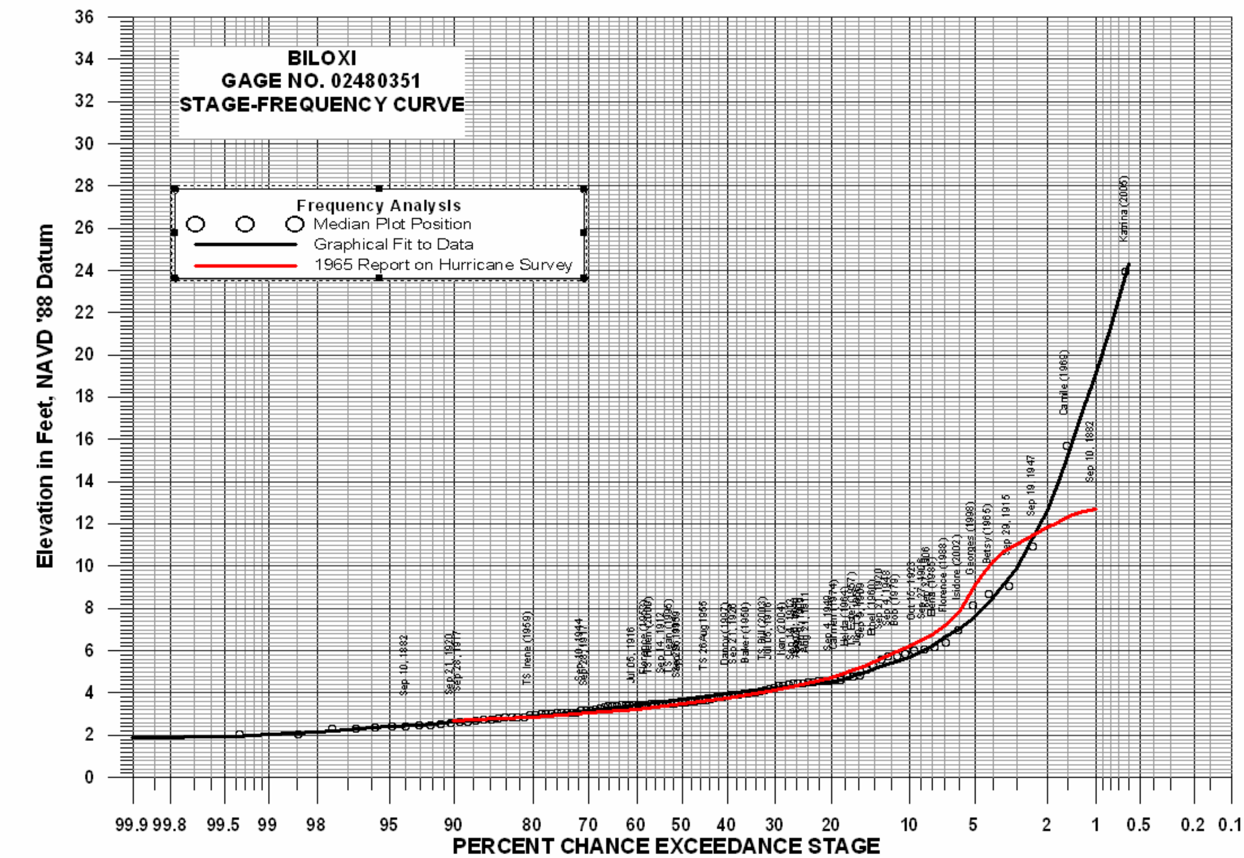


Figure 1.3-8. Biloxi, MS Frequency Curves

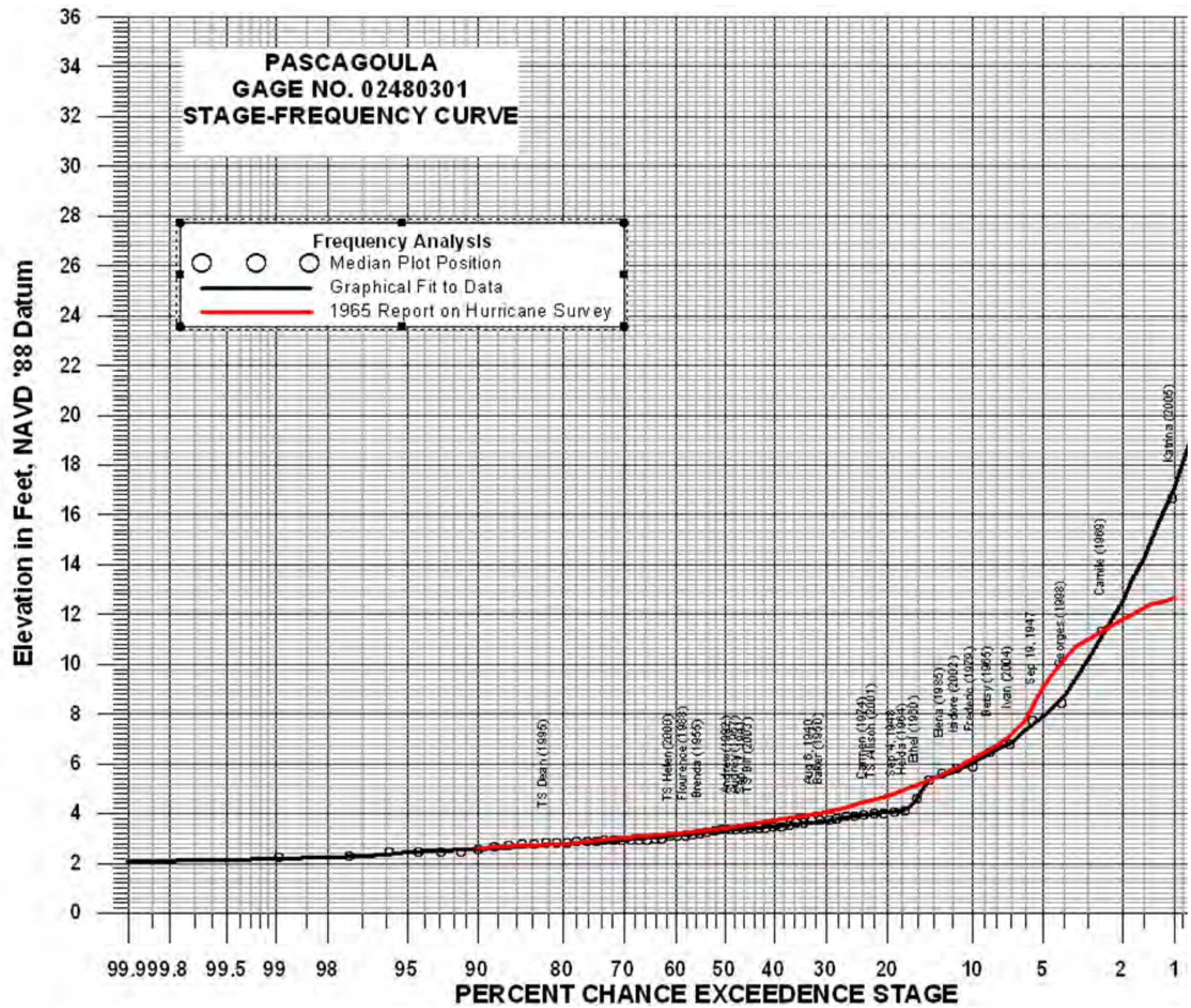


Figure 1.3-9. Pascagoula, MS Frequency Curve

Table 1.3-3.
Gulfport, MS Annual Peaks

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	24.30	1	1.09	0.77	Katrina (2005)
1969	19.81	2	2.17	1.86	Camille (1969)
1947	14.13	3	3.26	2.95	Sep 19, 1947
1965	10.83	4	4.35	4.05	Betsy (1965)
1915	9.13	5	5.43	5.14	Sep 29, 1915
2002	7.74	6	6.99	6.71	Isidore (2002)
1998	7.18	7	9.03	8.76	Georges (1998)
1957	6.63	8	11.06	10.8	TS Ester (1957)
1985	6.63	9	13.09	12.85	Juan (1985)
1926	6.14	10	15.12	14.89	Sep 21, 1926
1948	6.13	12	19.19	18.98	
1979	6.13	11	17.16	16.94	Bob (1979)

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1955	6.12	13	21.22	21.03	
1973	5.46	14	23.25	23.08	
1960	5.13	15	25.28	25.12	Ethel (1960)
1988	4.90	16	27.32	27.17	Gilbert (1988)
1970	4.85	17	29.35	29.21	
1984	4.83	18	31.38	31.26	
1974	4.81	19	33.41	33.3	Carmen (1974)
1986	4.78	20	35.44	35.35	
2004	4.76	21	37.48	37.39	Ivan (2004)
2001	4.56	22	39.51	39.44	TS Allison (2001)
1971	4.36	23	41.54	41.49	
1972	4.36	24	43.57	43.53	
1964	4.27	25	45.6	45.58	Helda (1964)
1997	4.25	26	47.64	47.62	Danny (1997)
1983	4.18	27	49.67	49.67	
1999	4.18	28	51.7	51.71	
1990	4.14	29	53.73	53.76	
1956	4.13	30	55.77	55.8	Flossy (1956)
1991	4.13	31	57.8	57.85	
2003	4.11	32	59.83	59.89	TS Bill
1992	4.02	33	61.86	61.94	Andrew (1992)
1980	3.93	34	63.89	63.99	
1967	3.87	35	65.93	66.03	
1987	3.83	36	67.96	68.08	
1977	3.76	37	69.99	70.12	
2000	3.75	38	72.02	72.17	TS Helen
1976	3.71	39	74.05	74.21	
1995	3.70	40	76.09	76.26	TS Dean (1995)
1993	3.62	41	78.12	78.3	
1994	3.49	42	80.15	80.35	
1996	3.47	43	82.18	82.39	Josephine (1996)
1975	3.36	44	84.22	84.44	
1966	3.35	45	86.25	86.49	
1981	3.23	46	88.28	88.53	
1982	3.20	47	90.31	90.58	
1989	3.13	48	92.34	92.62	
1978	3.06	49	94.38	94.67	
1968	2.96	50	96.41	96.71	
1963	2.75	51	98.44	98.76	

Table 1.3-4.
Biloxi, MS Annual Peaks

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	23.93	1	0.89	0.63	Katrina (2005)
1969	15.69	2	1.79	1.53	Camille (1969)
1947	10.93	3	2.68	2.42	Sep 19, 1947
1915	9.05	4	3.57	3.32	Sep 29, 1915
1965	8.69	5	4.46	4.22	Betsy (1965)
1998	8.18	6	5.36	5.12	Georges (1998)
2002	6.99	7	6.25	6.01	Isidore (2002)
1988	6.39	8	7.14	6.91	Florence (1988)
1985	6.16	9	8.04	7.81	Elena (1985)
1906	6.05	10	8.93	8.71	Sep 27, 1906
1923	6.01	11	9.82	9.61	Oct 15, 1923
1973	5.85	12	10.71	10.50	
1979	5.75	13	11.61	11.40	Bob (1979)
1948	5.73	14	12.50	12.30	Sep 4, 1948
1920	5.57	15	13.39	13.20	Sep 21, 1920
1960	5.25	16	14.29	14.09	Ethel (1960)
1972	5.12	17	15.18	14.99	
1956	4.83	18	16.07	15.89	Jun 13, 1956
1957	4.77	19	16.96	16.79	TS Ester (1957)
1964	4.76	20	17.86	17.68	Helda (1964)
1919	4.64	21	18.75	18.58	
1974	4.60	22	19.64	19.48	Carmen (1974)
1949	4.59	23	20.54	20.38	Sep 4, 1949
1934	4.57	24	21.43	21.27	
1984	4.56	25	22.32	22.17	
1983	4.53	26	23.21	23.07	
1911	4.49	27	24.11	23.97	Aug 21, 1911
1909	4.48	28	25.00	24.87	Sep 9, 1909
1940	4.45	29	25.89	25.76	Aug 6, 1940
1992	4.45	30	26.79	26.66	
1999	4.38	31	27.68	27.56	
2004	4.36	32	28.57	28.46	Ivan (2004)
1961	4.34	33	29.46	29.35	
1945	4.26	34	30.36	30.25	
1916	4.20	35	31.25	31.15	Jul 05, 1916
2003	4.12	36	32.14	32.05	TS Bill (2003)
1987	4.10	37	33.04	32.94	
1933	4.05	38	33.93	33.84	
1971	4.03	39	34.82	34.74	
1950	4.00	40	35.71	35.64	Baker (1950)
1966	3.96	41	36.61	36.54	
1905	3.95	42	37.50	37.43	
1926	3.95	43	38.39	38.33	Sep 21, 1926

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1993	3.93	44	39.29	39.23	
1997	3.87	45	40.18	40.13	Danny (1997)
1932	3.80	46	41.07	41.02	
1990	3.80	47	41.96	41.92	
1991	3.76	48	42.86	42.82	
1970	3.72	49	43.75	43.72	
1955	3.67	50	44.64	44.61	TS 26Aug1955
1996	3.66	51	45.54	45.51	
1927	3.65	52	46.43	46.41	
1952	3.61	53	47.32	47.31	
1941	3.58	54	48.21	48.20	
1935	3.56	55	49.11	49.10	
2001	3.56	56	50.00	50.00	
1939	3.55	57	50.89	50.90	Sep 26, 1939
1928	3.52	58	51.79	51.80	
1995	3.52	59	52.68	52.69	TS Dean (1995)
1912	3.51	61	54.46	54.49	Sep 14, 1912
1967	3.51	60	53.57	53.59	
1918	3.50	62	55.36	55.39	
1989	3.48	63	56.25	56.28	
2000	3.48	64	57.14	57.18	TS Helen (2000)
1953	3.47	65	58.04	58.08	Florence (1953)
1986	3.47	66	58.93	58.98	
1914	3.45	67	59.82	59.87	
1994	3.44	68	60.71	60.77	
1898	3.42	70	62.50	62.57	
1900	3.42	71	63.39	63.46	
1931	3.42	69	61.61	61.67	
1946	3.40	72	64.29	64.36	
1980	3.38	73	65.18	65.26	
1951	3.37	74	66.07	66.16	
1938	3.33	75	66.96	67.06	
1954	3.28	76	67.86	67.95	
1897	3.23	77	68.75	68.85	
1908	3.17	78	69.64	69.75	
1930	3.16	79	70.54	70.65	
1944	3.15	80	71.43	71.54	Sep 10, 1944
1929	3.07	81	72.32	72.44	
1937	3.07	82	73.21	73.34	
1942	3.07	83	74.11	74.24	
1943	3.05	84	75.00	75.13	
1982	3.05	85	75.89	76.03	
1921	3.02	88	78.57	78.73	
1958	3.02	86	76.79	76.93	
1975	3.02	87	77.68	77.83	
1922	2.96	89	79.46	79.62	

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1959	2.95	90	80.36	80.52	TS Irene (1959)
1936	2.87	91	81.25	81.42	
1963	2.86	92	82.14	82.32	
1976	2.85	93	83.04	83.21	
1981	2.83	94	83.93	84.11	
1924	2.79	95	84.82	85.01	
1907	2.77	96	85.71	85.91	
1913	2.75	97	86.61	86.80	
1904	2.70	98	87.50	87.70	
1896	2.66	99	88.39	88.60	
1917	2.66	100	89.29	89.50	Sep 28, 1917
1903	2.59	101	90.18	90.39	
1968	2.54	102	91.07	91.29	
1910	2.50	103	91.96	92.19	
1899	2.48	104	92.86	93.09	
1882	2.42	105	93.75	93.99	Sep 10, 1882
1884	2.40	106	94.64	94.88	
1925	2.35	107	95.54	95.78	
1962	2.34	108	96.43	96.68	
1902	2.30	109	97.32	97.58	
1885	2.07	110	98.21	98.47	
1901	2.07	111	99.11	99.37	

1

Table 1.3-5.
Pascagoula, MS Annual Peaks

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
2005	16.69	1	1.49	1.05	Katrina (2005)
1969	11.33	2	2.99	2.56	Camille (1969)
1998	8.45	3	4.48	4.07	Georges (1998)
1947	7.77	4	5.97	5.57	Sep 19, 1947
2004	6.81	5	7.46	7.08	Ivan (2004)
1965	6.49	6	8.96	8.58	Betsy (1965)
1979	5.87	7	10.45	10.09	Frederic (1979)
2002	5.84	8	11.94	11.60	Isidore (2002)
1985	5.59	9	13.43	13.10	Elena (1985)
1972	5.35	10	14.93	14.61	
1960	4.59	11	16.42	16.11	Ethel (1960)
1964	4.14	12	17.91	17.62	Helda (1964)
1948	4.09	13	19.40	19.13	Sep 4, 1948
1949	3.99	14	20.90	20.63	
2001	3.99	15	22.39	22.14	TS Allison (2001)
1974	3.95	16	23.88	23.64	Carmen (1974)
1970	3.90	17	25.37	25.15	
1961	3.89	18	26.87	26.66	
1984	3.80	19	28.36	28.16	
1983	3.77	20	29.85	29.67	
1950	3.74	21	31.34	31.17	Baker (1950)
1940	3.72	22	32.84	32.68	Aug 6, 1940
1980	3.62	23	34.33	34.19	
1987	3.62	24	35.82	35.69	
1993	3.54	25	37.31	37.20	
1956	3.49	26	38.81	38.70	
1945	3.46	27	40.30	40.21	
1971	3.44	28	41.79	41.72	
1967	3.42	29	43.28	43.22	
2003	3.42	30	44.78	44.73	TS Bill (2003)
1941	3.39	31	46.27	46.23	Sep 12, 1941
1957	3.37	32	47.76	47.74	Audrey (1957)
1992	3.37	33	49.25	49.25	Andrew(1992)
1996	3.37	34	50.75	50.75	
1986	3.33	35	52.24	52.26	
1952	3.24	36	53.73	53.77	
1955	3.19	37	55.22	55.27	Brenda (1955)
1953	3.14	38	56.72	56.78	
1988	3.12	39	58.21	58.28	Florence (1988)
1991	3.12	40	59.70	59.79	
2000	3.09	41	61.19	61.30	TS Helen(2000)
1978	3.01	42	62.69	62.80	
1990	2.97	43	64.18	64.31	
1989	2.96	44	65.67	65.81	
1973	2.95	46	68.66	68.83	
1951	2.94	47	70.15	70.33	

Year	Gage Height ft. NAVD	Rank	Weibull Plotting Position (FFA)	Median Plotting Position (FFA)	Storm
1966	2.93	48	71.64	71.84	
1994	2.93	49	73.13	73.34	
1975	2.90	50	74.63	74.85	
1958	2.89	51	76.12	76.36	
1959	2.89	52	77.61	77.86	
1963	2.85	53	79.10	79.37	
1982	2.84	54	80.60	80.87	
1995	2.84	55	82.09	82.38	TS Dean (1995)
1946	2.77	56	83.58	83.89	
1999	2.77	57	85.07	85.39	
1954	2.74	58	86.57	86.90	
1976	2.66	59	88.06	88.40	
1981	2.55	60	89.55	89.91	
1944	2.47	61	91.04	91.42	
1977	2.47	62	92.54	92.92	
1954	2.74	58	86.57	86.90	
1976	2.66	59	88.06	88.40	
1981	2.55	60	89.55	89.91	
1944	2.47	61	91.04	91.42	

1.3.3.2 Composite Stage-Frequency Curves

As mentioned in Section 1.3, these probabilistic graphical analysis results were joined with hydrodynamic and statistical model results to create composite stage-frequency curves used for a host of MsCIP design and evaluation efforts as discussed throughout this report. This section presents ERDC modeling results at the location of the USACE gages with those results obtained by probabilistic analysis of gage data and shows how they were combined to form composite stage-frequency curves.

Figure 1.3-10 shows stage-frequency components obtained through probabilistic analysis of historic gage data at Gulfport with ERDC results for the same location. ERDC results were obtained from the results of hydrodynamic modeling of severe storm events and statistical analysis of hydrodynamic model output as described in Chapter 2. These results are referred to as ‘synthetic,’ as they were not explicitly developed from observed data, and represent the best estimate of stage for a given annual chance of occurrence. Uncertainty bands[1] for these best estimates were computed and are used in the analyses supporting the MsCIP program. Figure 1.3-11 shows the joined, or composite, stage frequency curves with uncertainty at 2 standard deviations. The curves were joined graphically. This figure was obtained from the HEC-FDA model, in which one hundred feet has been added to stage for computational purposes; the data are otherwise consistent. Similar figures are presented as Figures 1.3-12 through 1.3-15 for both the Biloxi and Pascagoula gage locations.

A more detailed discussion on the development and adaptation of composite stage-frequency information to the flood damage evaluation purpose is provided in section 2.16.

[1] Uncertainty computations are discussed in sections 2.9 and 2.16.

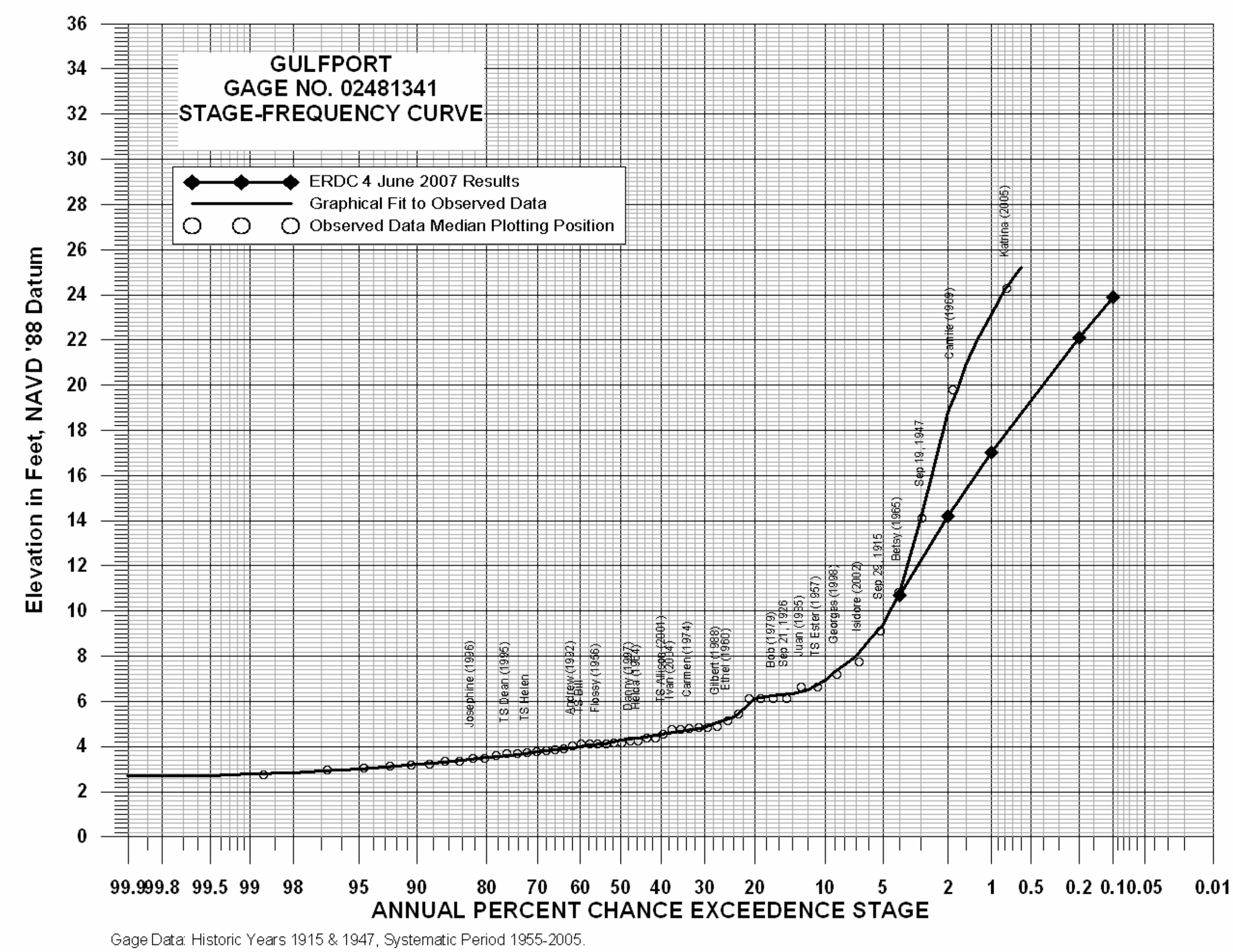


Figure 1.3-10. Graphical and Synthetic Stage-Frequency Curve Components at Gulfport

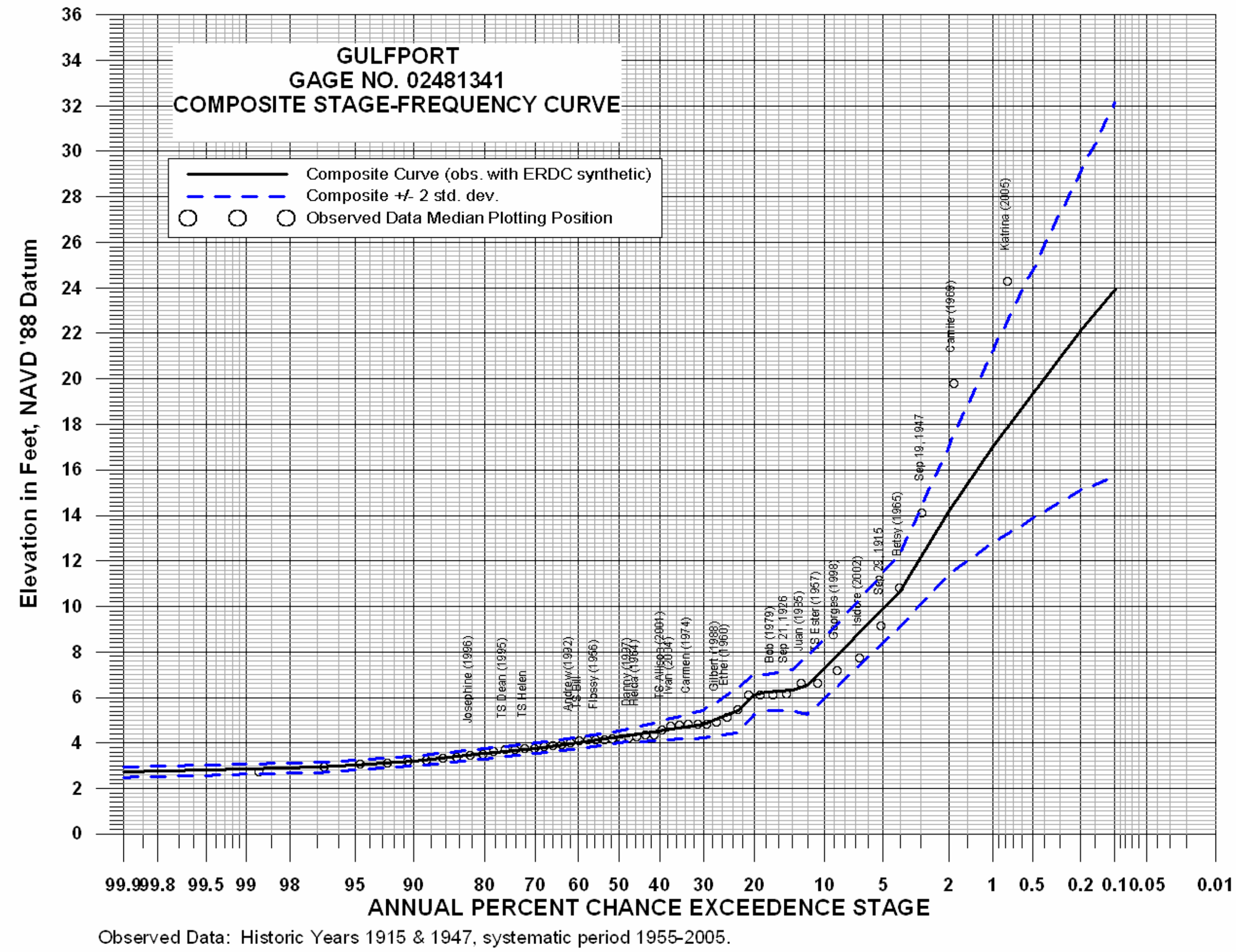


Figure 1.3-11. Composite Stage-Frequency Curve, Gulfport

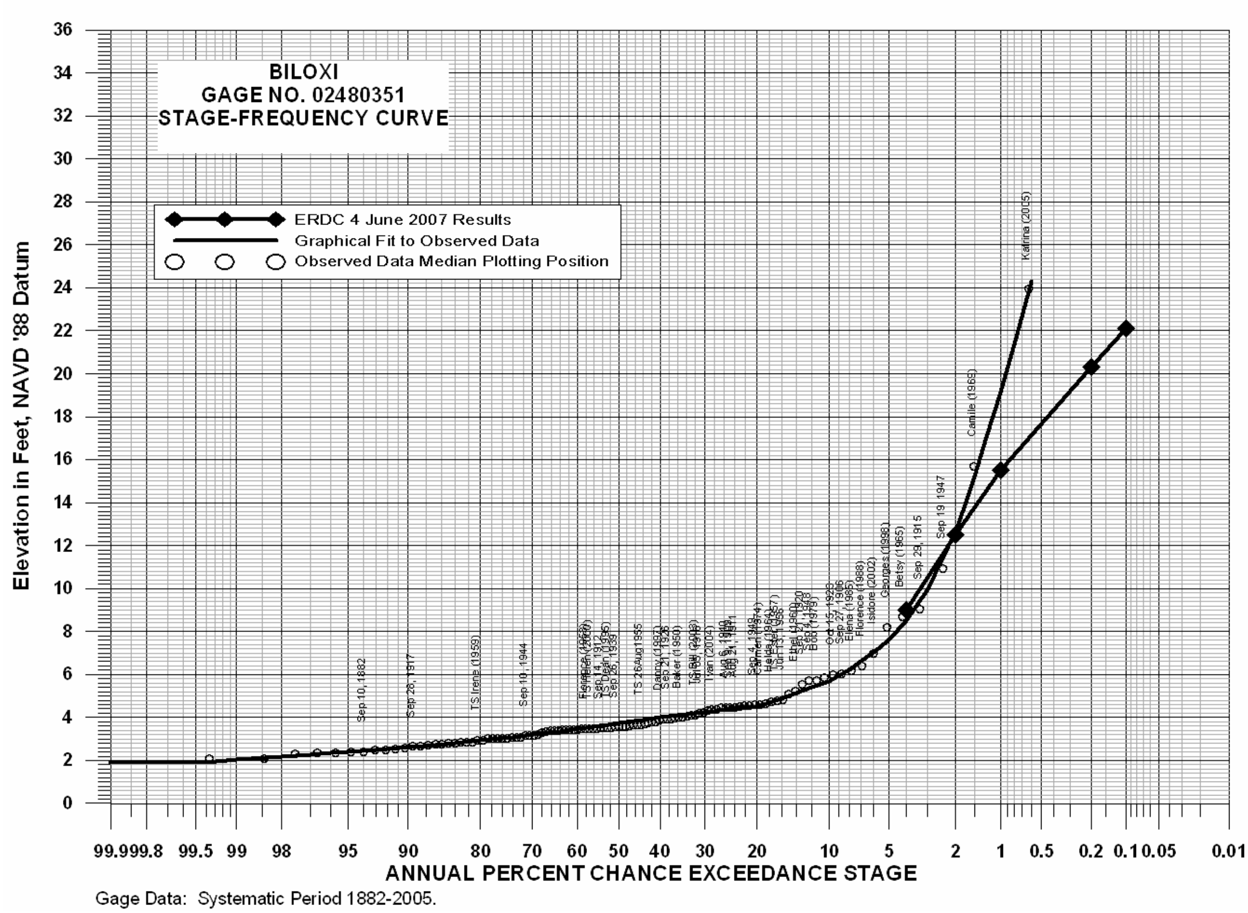


Figure 1.3-12. Graphical and Synthetic Stage-Frequency Curve Components at Biloxi

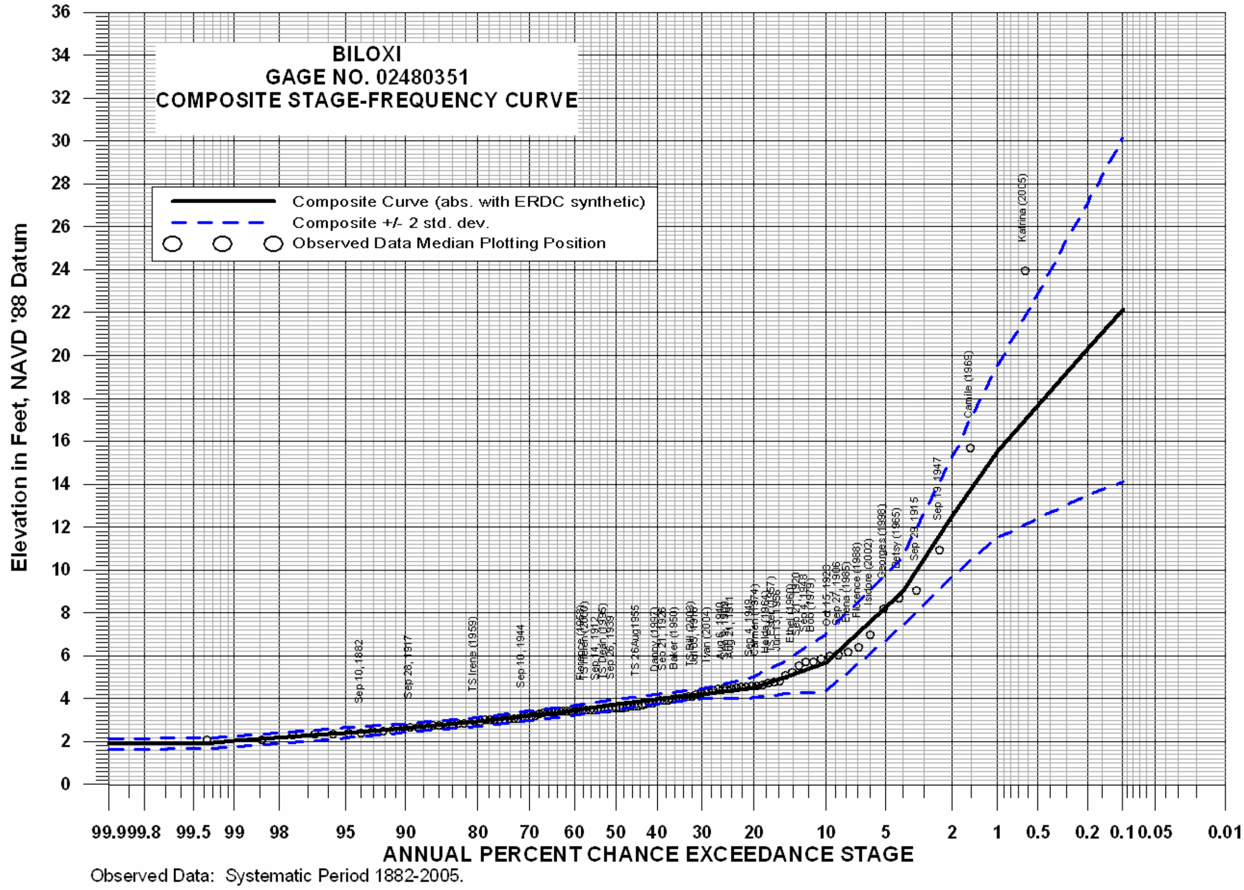
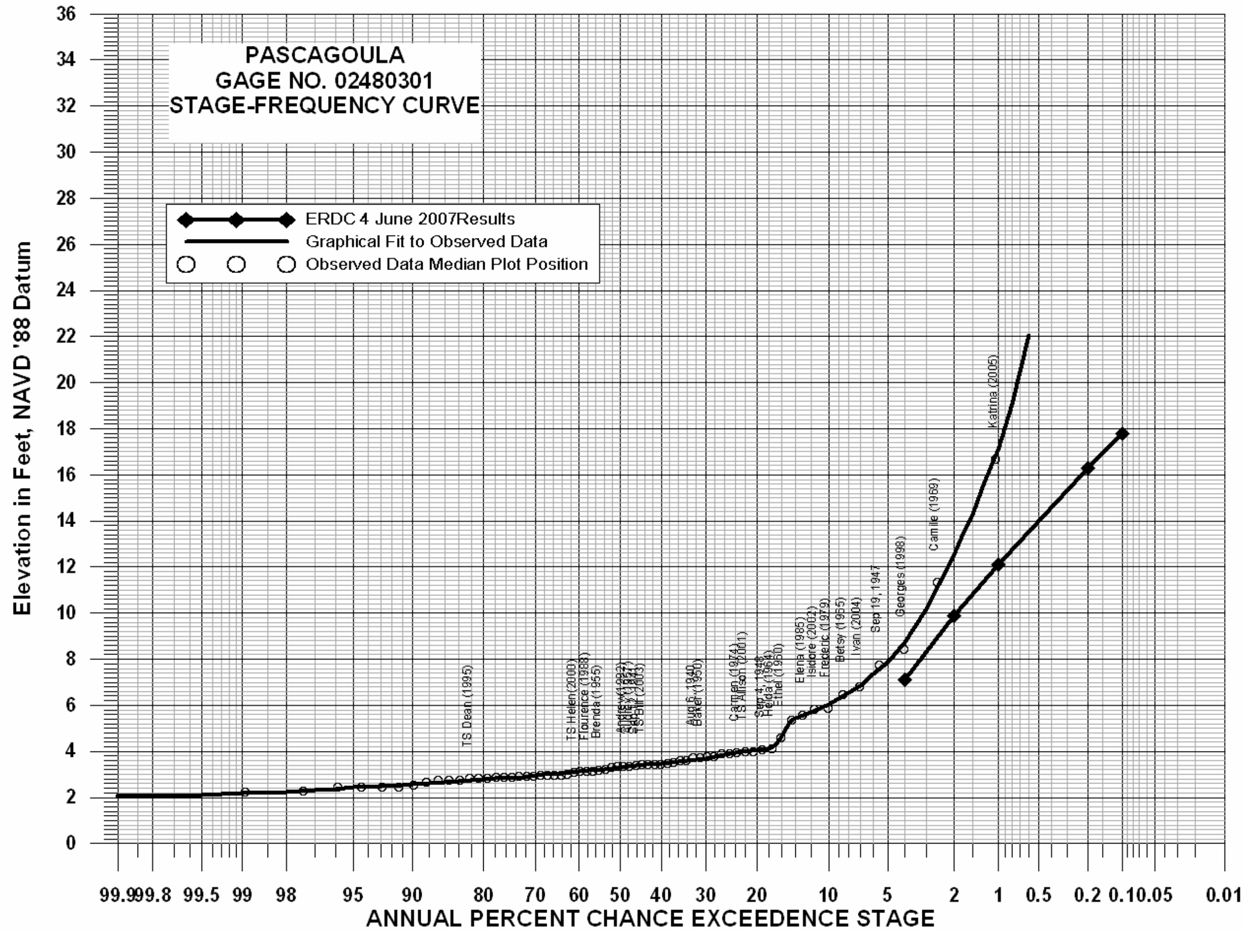


Figure 1.3-13. Composite Stage-Frequency Curve, Biloxi



Gage Data: Period of Record 1941-2005.

Figure 1.3-14. Graphical and Synthetic Stage-Frequency Curve Components at Pascagoula

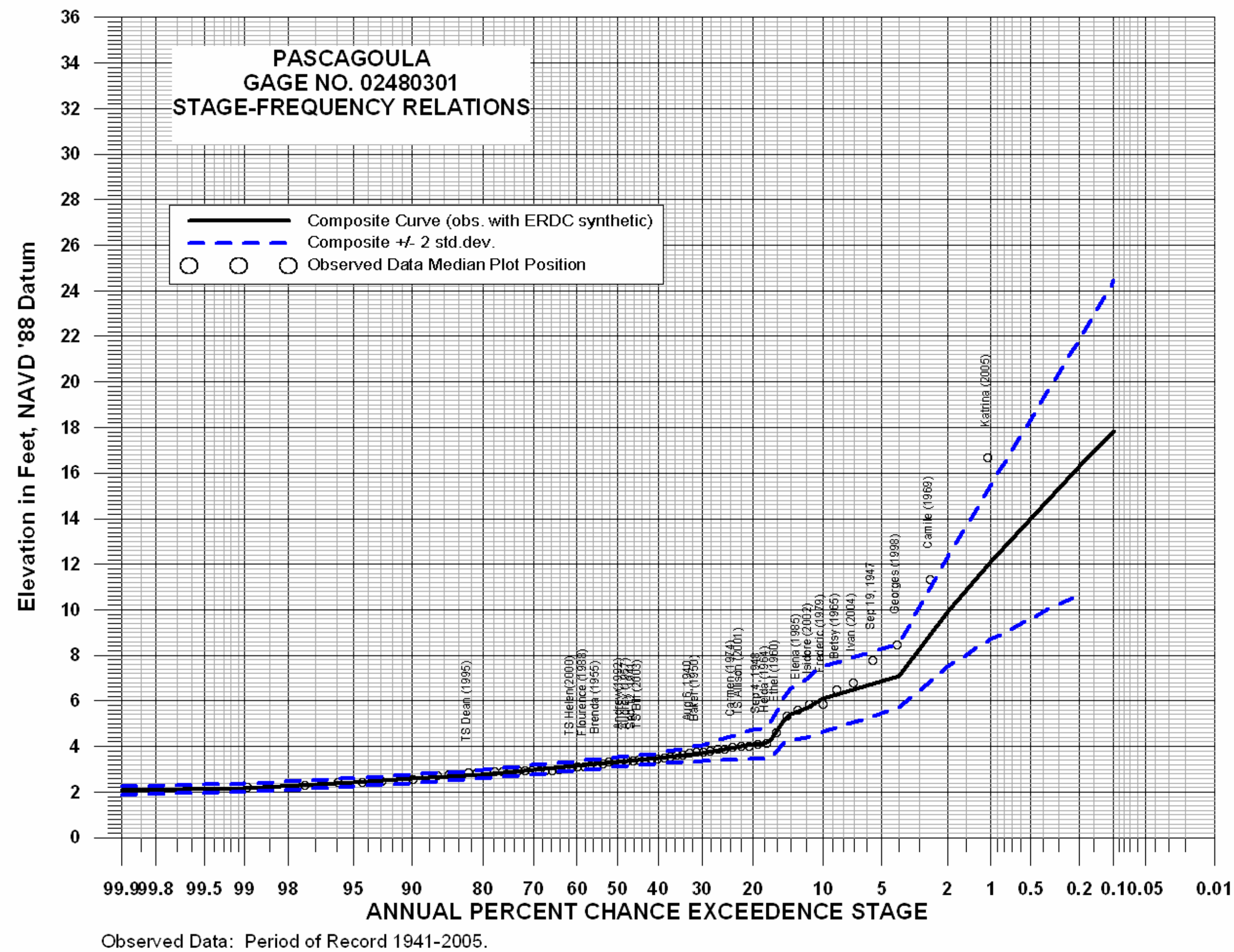


Figure 1.3-15. Composite Stage-Frequency Curve, Pascagoula

1.3.4 References

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1.4 Typical Wind, Wave, Water Level, Current, and Sediment Transport Conditions

The Mississippi Sound extends from Mobile Bay, Alabama, to the east to Lake Borgne, 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. The Sound has a mean depth of 10 ft Mean Low Water (MLW) and more than 99% of it is shallower than 20 ft MLW. The Sound extends about nine miles north to south from the Mississippi mainland coastline to a series of low, typically sandy barrier islands on the edge of the coastal shelf which marks the Gulf of Mexico.

1.4.1 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 May through October hurricane conditions. Hurricane wind fields and their effects on storm surge and waves are an area of particular concern for this study and are discussed at length in Chapter 2 of this appendix.

1.4.2 Waves

Wave intensity of the Mississippi Sound is typically 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 with periods of five to eight seconds. Hurricane and storm conditions, and strong winter cold fronts can produce significant surges and much larger wave conditions at the coast and barrier islands. Wave phenomena due to hurricanes are discussed in detail in Chapter 2 of this appendix.

1.4.3 Tides

The mean tidal range near the Mississippi Sound shoreline is approximately 1.5 ft. Although the tidal range caused by astronomical forces is relatively small, atmospheric pressure variation and, particularly, 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. The tidal variation in the Mississippi Sound and adjacent waters is typically diurnal (one high tide and one low tide daily) though mixed tides (two high tides and two low tides) occur a few days out of the month. The average tide cycle is 24.8 hours which is slightly less than one lunar day. Mobile District has a long tide level monitoring history in Mississippi as discussed in section 1.3. The long period of record provides for an interpretation as to the relative rate of sea level rise as discussed in section 1.6.

1.4.4 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.

1.4.5 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, such as outfalls, port facilities, and sand enclosures along the Mississippi mainland coastline that 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 border direct silt and clay into the sound. Salinity-induced flocculation of these very fine 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 its characteristically brownish appearance.

1.5 Geologic Setting and General Geophysical Investigations

1.5.1 Geologic Setting and Physiography

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 an 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 aged 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 sea level changes associated with global glacial action 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 with replenishment to this system coming from the east associated with sediments from the Apalachicola River that contribute to the barrier islands in northwest Florida westward into Alabama along Dauphin Island. A predominant wave action from the southeast creates 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 In Mississippi. The transgressive-regressive sequence has reworked sand and other sediments along the coast that 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 1.5-1, (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.

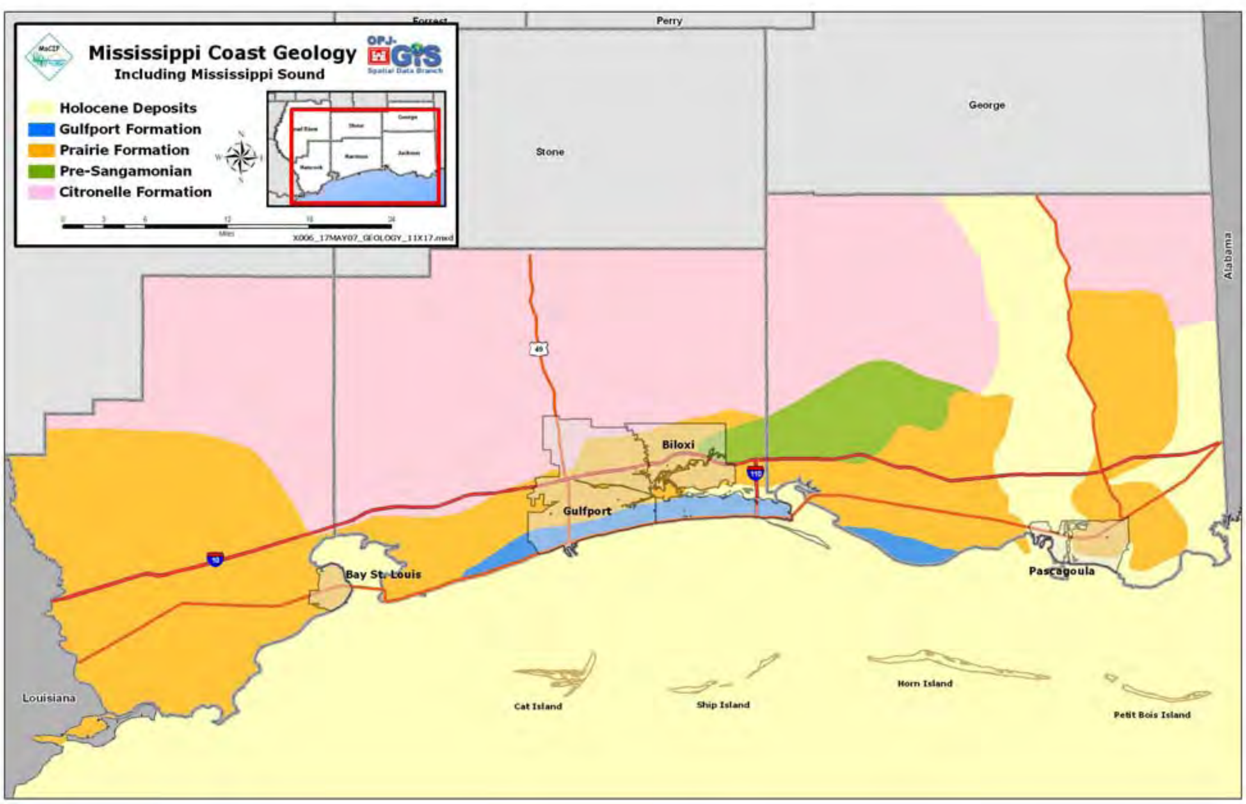

Figure 1.5-1. Generalized Geologic Map of Coastal Mississippi (After Otvos, 1997)

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.

The northern portions of the three counties contain limited outcrops of the Miocene aged Pascagoula/Hattiesburg formation. This formation contains inter-bedded 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.

Collectively, the formations that outcrop within the study area provide vast quantities of useful construction material that includes high quality sand, sandy clays and clay. The nature of the various options discussed in this document will require all of these types of materials and the availability of these materials commercially throughout the area will benefit any project costs. Other than limited locations that fall within river channels or the bay bottoms, the geologic formations are expected to provide good foundation conditions. The areas within the river channels and bay bottoms will require deep geotechnical exploration to define local conditions, however the presence of major highway bridges and train trestles indicate that suitable deep foundations can be designed.

The study area is located within the East Gulf Coastal Plain physiographic province. There are two major physiographic regions in the Mississippi coastal region. The Gulf Coast Flatwoods form an irregular belt through the southern half of the three-county region. This belt consists mainly of wet lowlands and poorly drained depressions, with some higher, adequately-drained areas. The second physiographic region, the Southern Lower Coastal Plain, is rolling and gently undulating, interior uplands. Elevations range from sea level along the coast in Hancock, Harrison, and Jackson Counties to about 420 feet above sea level. The slope of the land surface is generally oriented to the south. The area is underlain by a thick sequence of sedimentary deposits dipping to the south and west.

1.5.2 Historical Offshore Sampling and Geophysical Exploration

Historical Offshore Sampling and Geophysical Exploration - To support any nourishment of sand along the mainland and on the barrier islands, extensive deposits of beach quality sand will be required. The sand will have several physical requirements that include color, grain size, and particle shape. 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. Another source of data exists with the US Geological Survey office located in St. Petersburg, Florida. Vast amounts of acoustic profiles are contained within their files in analog format. (Oral communication, Flocks, 2006) These profiles include the areas within Mississippi Sound, around the barrier islands, and southward out into the Gulf.

Extensive additional information is also stored in archives at the United States Geological Survey, but not in a user friendly format. These records include thousands of miles of acoustic profiles that exist as analog data recorded on scrolls. Through cooperation with the Mineral Management Service, efforts are underway to have these records transferred to a digital format that can be incorporated into a GIS type database. Of particular interest to this study is the St. Bernard Shoals that lie about 45 miles south of the barrier islands. St. Bernard Shoals is now a series of submerged barrier islands that existed when the sea levels were much lower. It is believed that large quantities of high quality sand exists in the Shoals that could be used for the restoration of beaches and dunes both on the barrier islands and the mainland beaches.

1.5.3 Proposed Offshore Geophysical Exploration

Proposed Offshore Geophysical Exploration - Additional acoustic profiling is proposed for off-shore areas within Mississippi Sound and in some areas south of the barrier islands. These surveys will help identify sand deposits that could be used or re-nourishment of the islands and to provide data on the shallow strata between the islands. Some of the area is within the boundaries of the Gulf Islands National Seashore and work within these boundaries must be approved by the National Park Service. 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 1.5-2.

Figure 1.5-2. 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.

1.5.4 Tectonic and Seismic Considerations

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 produces no significant seismic events, associated displacements must be considered even if very small. Computed subsidence of 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). These rates are the subject of much discussion among various agencies due to the fact that the primary benchmarks may not be stable thus influencing the results any surveys. The need to update the benchmarks to provide accurate elevation data is recognized by the National Geodetic Survey. Mississippi's subsidence has been factored into the relative sea level rise based on over eighty years of observation at three tide gauges along the coastline, Gulfport, Biloxi and Pascagoula. The relative sea level rise is based on both actual changes in sea level and any subsidence combined into a single value. This change would be what the casual observer would notice over time along the coast. The relative sea level values will be considered in all designs.

1.5.5 On-shore Borrow Areas

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 1.5-3. 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 individual site reserves may be less than one million cubic yards, but collectively contain vast quantities of material. 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 1.5-1, 1.5-2, and 1.5-3, respectively.

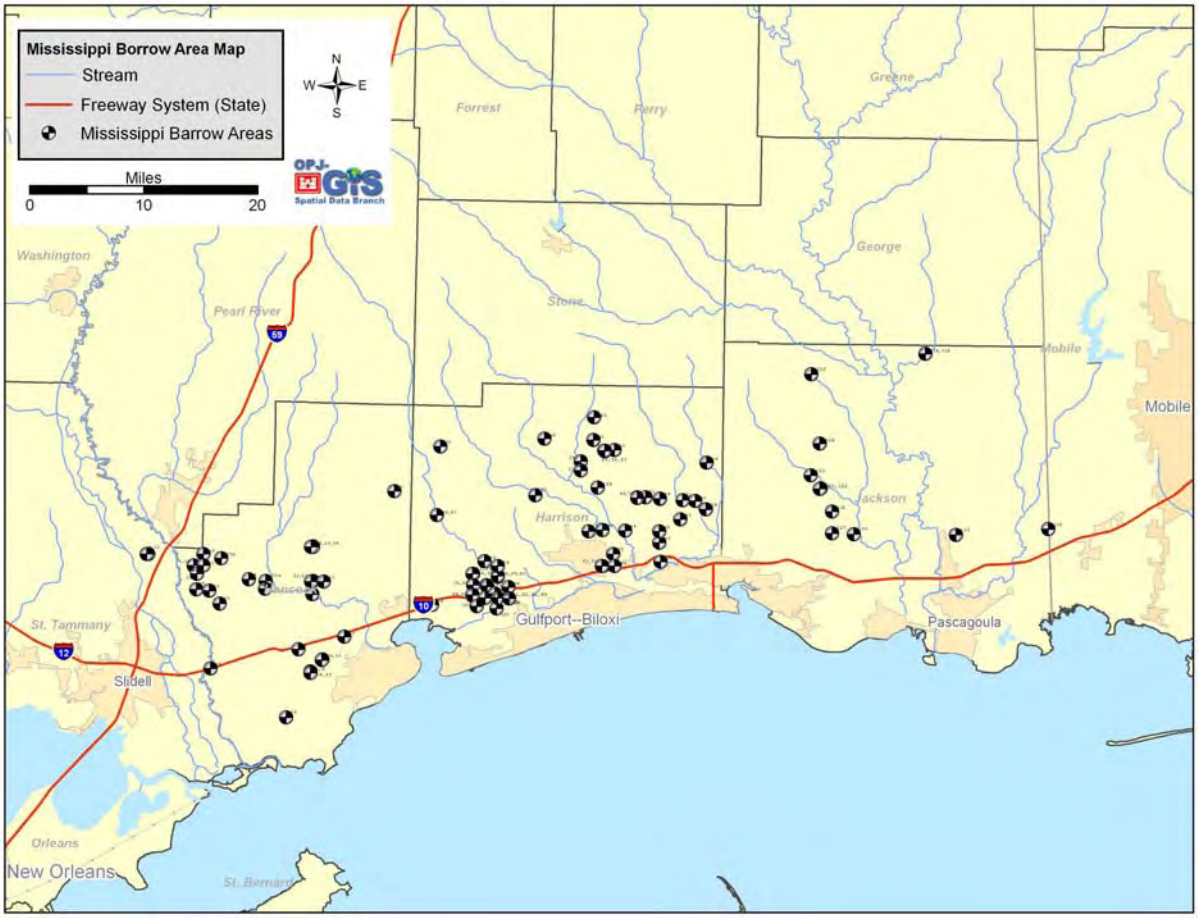

Figure 1.5-3. Location of Permitted Mining (Borrow) Operations in Coastal Mississippi Counties

Table 1.5-1.
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 1.5-2.
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

Table 1.5-2.
Permitted Borrow Areas in Harrison County (continued)

County	Operator	Permit #	Permitted Acres	Material
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
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 1.5-3.
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
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 as interim projects and will require sand 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 offshore 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.

1.5.6 Offshore Borrow Areas

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. Within the Seven Point Hurricane Recovery Strategy developed by the Governor of the State of Mississippi, one is restoring the barrier islands of the coast of Mississippi to a pre-hurricane Camille footprint. This is addressed in this appendix as Option A under the Barrier Islands. This will involve establishing islands of a size similar to a pre-Camille condition with allowances made for migration of the islands over time. 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.

At the present time, four areas have been selected for acoustic profiling near the barrier islands to assist in identifying potentially useful deposits of sand. An initial quantity of 66,000,000 cubic yards of sand has been estimated for use on the barrier islands as the quantity of sand for restortation to a pre-Camille footprint as described above and would be the target for this survey. During hurricane Katrina, the breach of Ship Island was widened to approximately three to four miles. This breaching also occurred during Hurricanes Fredrick and Camille with a low sand spit 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 the islands were restored to the pre-Camille footprint, the restoration of Ship Island will be the largest single project requiring up to 30,000,000 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 and bringing sand dunes to at least elevation 20 feet (NAVD 88) 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 1.5-2. 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.

Prior studies of the St. Bernard Shoals (Oral Communication, USGS, 2006) are probably the best source of the sand, although additional studies and sampling will be required to ensure the sediments meet the quantity and quality requirements. St. Bernard Shoals are a series of submerged barrier islands located south of the existing islands (see Figure 1.5-4) and are believed to contain substantial quantities of high quality sand, more than enough to supply the quantity needed for any use at the barrier islands. The US Geological Survey is presently compiling historical data on offshore sand deposits that will include the St. Bernard Shoals area. This study will also include some sampling of selected areas.

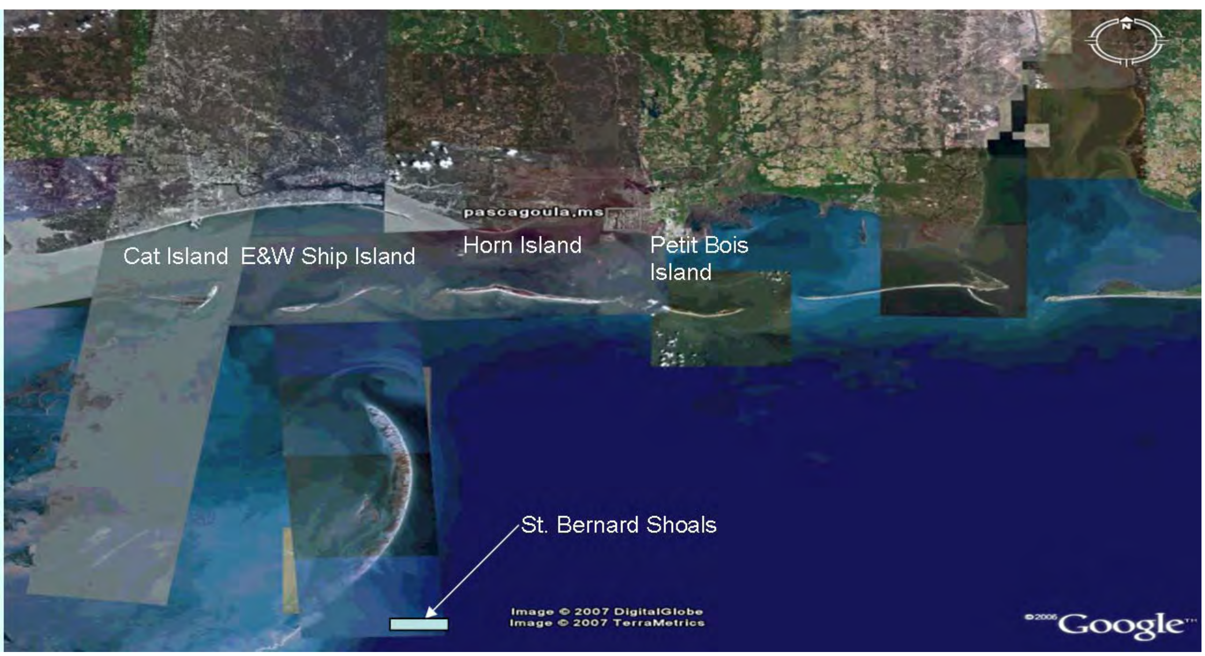

Figure 1.5-4. Map Showing the Location of St. Bernard Shoals

1.5.7 Inland River System Sand (Dredged Material)

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. Figure 1.5-5 shows an aerial view of one of the sites. 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. Information on available sand on these two river systems is shown in Tables 1.5-4 and 1.5-5. 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 1.5-10 shows the relationship of these disposal areas to the project sites along the Mississippi coast. The range of haul distances (by water) to the barrier islands western extent varies from 163 to 500 miles. Material from these sites could easily be transported by barge down the river system for use along the beaches. The cost to store this type of dredged material is high and it has recently been estimated that removing the sand from the existing disposal areas would save the Government over $100,000,000 at today's cost.

Figure 1.5-5. Sunflower disposal area on the Tombigbee River with large quantities of sand available for use on coastal projects in Mississippi

Table 1.5-4.
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

Table 1.5-4.
BWT Dredge Material Disposal Areas Over 100,000 CY (continued)

Site	River Mile	Acquisition	Access/ Land	Access/ River	Est Material Placed To Date(CY)
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 1.5-5.
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

Table 1.5-5.
TTW Dredge Material Disposal Areas Over 100,000 CY (continued)

Site	River Mile	Acquisition	Access/ Land	Access/ River	Est Material Placed To Date(CY)
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

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.

Review of the documents referenced above indicated that the color issue was not resolved and this would be an important factor in the use of the sand on the barrier island beaches. The methods employed, beaching and agitation with exposure to ultraviolet light, were not considered to be effective in removal of what is suspected to be the basis of the color on the sand grains, amorphous iron oxide more commonly referred to as rust. Hydrogen peroxide is a common household bleaching agent that is effective in oxidation of organic matter, but would not effect iron oxide through chemical removal. The same is true for the effects of ultraviolet light on iron oxide. The idea of using agitation would be the most effective of the methods attempted if the color was a coating on the mineral grains, but the test, as conducted, was not conclusive.

With the renewed interest in the possibility of using the sand as a source of material for the littoral zone associated with the Mississippi barrier islands, the disposal areas warranted further study. Again the color of the sand is a concern that has been raised by the National Park Service who has control of the Mississippi Barrier Islands. This concern has both aesthetic and environmental aspects. Aesthetically, the beaches on the barrier islands are composed of relatively white sand. Numerous studies have indicated that the primary source of this sand is an Appalachian origin probably associated with river systems discharging onto the Continental Shelf of present-day Florida (Stone and Others, 2004). This sand is transported westward from the discharge of the river into the Gulf of Mexico. Transport of this sand along the prevailing littoral current has created the white beaches and barrier islands that extend from the mouth of the river in Florida westward across Alabama to Mississippi.

Looking at the color differences of the sand along this system reveals a definite change as shown in Figure 1.5-7. The sample on the left was taken from sand dredged from the Chattahoochee River which is a major tributary of the Apalachicola River. This sampling location is approximately 150 river miles above the Gulf. The middle sample was taken from Disposal Area 39 on the Apalachicola River approximately 37 river miles above the Gulf. The sample on the right was taken from the south beach of Petit Bois Island in Mississippi. Note the change progressive change in color from brown to tan to white.

Geochemical processes could account for the consistent staining of the sand grains while in the river system. As the sand entered the Gulf's littoral system, changes in the geochemical process would not allow additional staining of the sand and any removal of the coating would allow the underlying sand grain to display its true color. The mechanical process of abrasion would occur both in the river system and the littoral system, but if the iron oxide staining was continuously reoccurring in the river system, the resulting color would remain. As the sand grains entered a different geochemical environment where re-staining did not occur, it would account for the difference where the color was a coating. Review of selected sand samples taken from the Black Warrior–Tombigbee River system disposal areas the reveal the same general color that is characteristic of the Chattahoochee-Apalachicola River system. Figure 1.5-8 is a photograph of five samples that include the same samples used in Figure 1.5-7 plus two additional samples, one from the Black Warrior River and another from the Tombigbee River. Note the similarities in color of the Apalachicola River (fourth from left), the Black Warrior (third from left and marked BWT North Star), and the Lower Princess (second from left, Lower Tombigbee River).

Figure 1.5-6. Littoral zone (white beaches and islands) along Central Gulf Coast extending from Bay County, Florida (top of picture) to Mississippi Barrier Islands (lower left), looking east

Figure 1.5-7. Samples of sand taken from (left to right) Chattahoochee River Mile 150, Disposal Area #39 on the Apalachicola River, and Petit Bois Island

Figure 1.5-8. Samples of sand taken from (left to right) Chattahoochee River Mile 150, Disposal Area #39 on the Apalachicola River, North Star disposal area on the Black Warrior River, Lower Princess disposal area on the Tombigbee River, and Petit Bois Island in Mississippi

Assuming that the previous testing was not effective at removing the iron oxide staining on the sand grains, a different bench-top test was performed. If iron oxide is only a coating on the sand grains and occurs as a stain, abrasion would be effective in the removal. The addition of a week acid would also aid in keeping the iron oxide from re-coating the sand grains as it is being removed. For the experiment, I used a small “rock tumbler” of the type used to polish small stones. Into the chamber of the rock tumbler was added a small quantity of sand obtained from the Lower Princess disposal area on the Tombigbee River, enough water to just cover the sand and a tablespoon of “Zud”. Zud is a household cleaning product that is composed of oxalic acid and abrasives. Oxalic acid is a weak acid commonly used to remove rust stains. Zud contains about 10% oxalic acid and 90% fine abrasives. The tumbling chamber was closed and placed the tumbler. An electric motor spins the chamber which allows the contents to tumble. This process would mimic the process of sand grains being transported along the littoral zone with the sand grains being abraded as they strike each other. In the almost infinite volume of water in the Gulf, any iron stain that was removed would not re-coat the sand, but be diluted away. This process started on 4 October 2007 and concluded 10 October 2007. The tumbler did not run over the included long weekend, but did operate for about 4 days. At the completion of the tumbling process, rinse water was added and decanted several times until the turbidity levels dropped and the fines were removed. The remaining sand was air dried and placed in a clear plastic bag for comparison with sand from the same parent sample. As shown in Figure 1.5-9, the results of the experiment are quite dramatic. The tumbled sand lost most of the tan color and is approaching white. This supports the process that occurs with the tan sand from the Apalachicola River system becoming the white sand so familiar to beach-goers along the central Gulf Coast.

Figure 1.5-9. Samples of sand taken from (left to right) North Star disposal area on the Black Warrior River, Lower Princess disposal area, and "Tumbled Lower Princess disposal area"

Adding the sand into the littoral system along the gulf coast should provide the proper geochemical and mechanical processes to remove the iron staining and provide the quality of sand that is desired as it is transported along the littoral drift zone which contain the Mississippi Barrier Islands. Littoral zone placement will also allow additional sorting by the currents and rounding of the sand grains through continued abrasion during transport. Additional research and testing will be conducted to ensure that these processes will in fact provide sand that is compatible with the existing sand in the barrier island system.

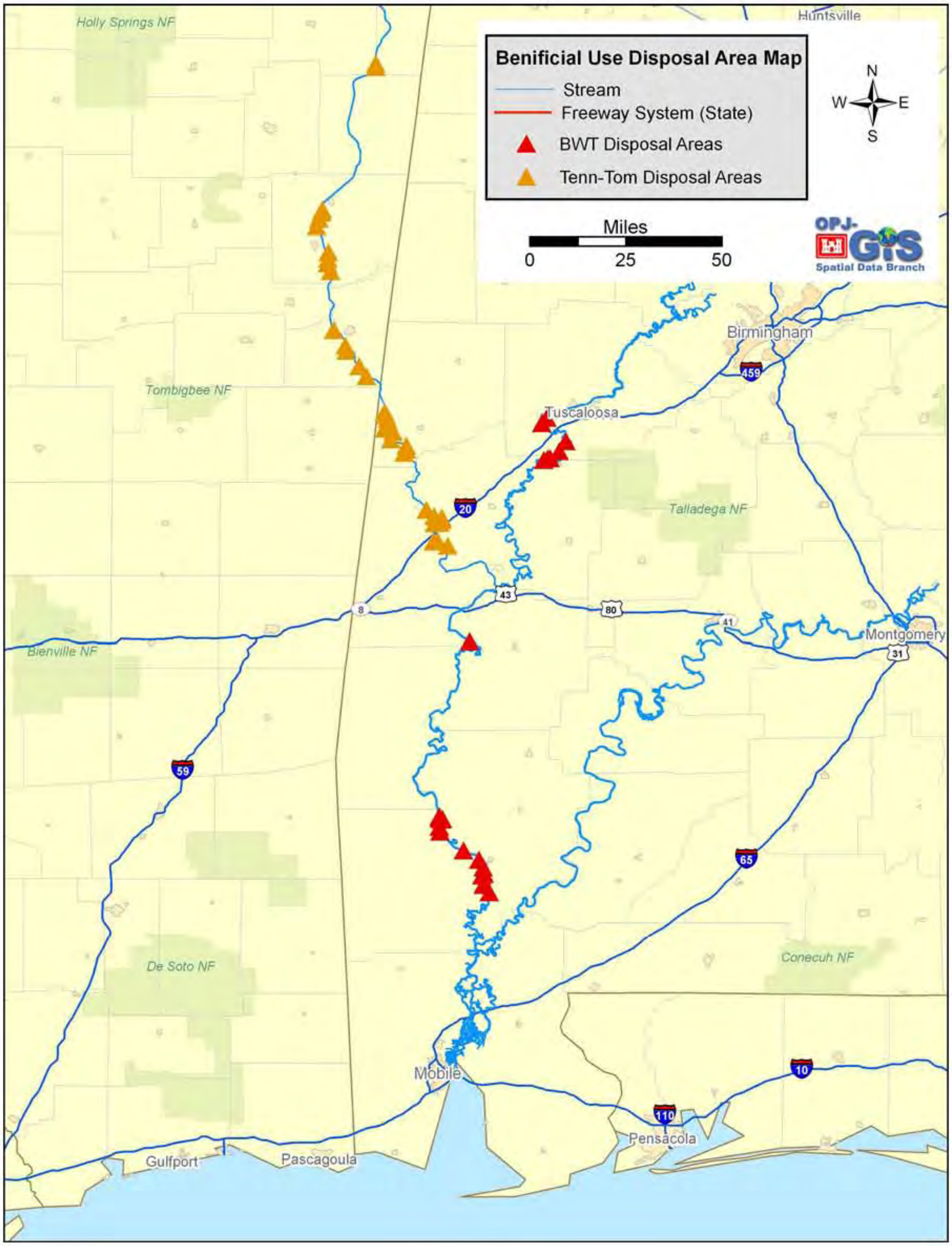

Figure 1.5-10. Inland Disposal Areas that Contain Economic Deposits of Sand

1.5.8 References

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1.6 Sea Level Rise

Systematic long-term tide elevation observations suggest that the elevation of oceanic water bodies is gradually rising and this phenomenon is termed ‘sea level rise.’ The rate of rise is neither constant with time nor uniform over the globe. Present estimates of recent (over about the last 100 years) global average, or eustatic, sea level rise are varied but the average value is about 2 millimeters per year. Sea level is rising due to global warming, and there is uncertainty as to the future rate of sea level rise, how much sea level will rise at any particular location, what the primary drivers of global warming really are, and whether the rate of rise will be relatively constant or accelerate. Regardless of these uncertainties, with 60 percent of the world's population, and 53 percent of the US population, living near the shoreline (Reference 1), sea level rise is a phenomenon which requires society's sustained attention and requires planning with consideration to the needs and protection of future generations.

Sea level rise may be viewed in different ways. 'Eustatic' sea level rise refers to estimates of the rate of sea level rise applied uniformly over the earth's oceanic water bodies. This is an interesting concept and useful for communicating an averaged rate, but because sea level rise is not uniform over the globe, it is not perhaps the most useful concept from a local or regional engineering point of view. Eustatic sea level concepts are usually associated with studies of pre-historic sea level and predictions of future sea level behavior but have been used in the Gulf Coast region in forensic studies of modern coastal subsidence rates (Ref. 2).

'Relative' sea level rise (RSL) at a given location is the change in mean sea level at that location with respect to an observer standing on or near the shoreline. It is determined by fitting a linear relationship to monthly mean or annual mean sea level, either of which is computed from tide gage observations. The slope of the fitted line gives the rate of sea level rise at the location of the tide gage. The computed rate includes the rate of subsidence or uplift of the location upon which the tide gage is founded, and thus the computed RSL rates may be extended locally or regionally to areas with similar geotechnical and tidal conditions.

The National Research Council (NRC) alternatively defines relative sea level change as "the difference between eustatic (global) sea level change and any change in local land elevation" (Ref. 3). This definition is in keeping with the previous interpretation in that local vertical land motion is represented in the change estimate, however, it seems to equate eustatic sea level change to the local absolute sea level component of that change, whereas the previous interpretation makes no such assumption. In practice, the distinction is often ignored because, excepting at the poles where sea level rise would be expected to be higher than an average eustatic value, there are no consistent relationships between eustatic sea level rise and sea level rise at any particular location.

Corps of Engineers Planning Guidance Notebook (Ref. 4) states that potential relative sea level rise should be taken into consideration for coastal or estuarine projects at the feasibility level of study and recommends, given the uncertainty of future sea level rise estimates, preference be given to developing strategies that are robust over the entire range of potential sea level rise rates versus those that perform well only over a limited range of potential sea level rise rates. The guidance states that, at a minimum, project performance would be evaluated based on extrapolation of the observed historic rate and should also consider a higher rate than that historically observed. The guidance specifies, in the absence of more current, definitive information, that Curve 3 of the 1987 NRC study (Ref. 2), a curve presented as a high forecast rate of rise, be used as the eustatic component in estimating the locally 'higher than observed' rate.

It is necessary then to determine (a) the observed historic relative rate of sea level rise along the Mississippi Gulf Coast, (b) the observed and/or forecast rates of subsidence there, and (c) the Curve 3 rate and, if available, other updated, definitive estimate of eustatic sea level rise that may be extended to the Mississippi coast. The following sections describe these determinations.

MsCIP studies are interpreting the guidance as requiring estimates of the magnitude of sea level rise for the expected project life beginning at the base year. Early on in the study, this time period was set at 2005-2100, but has since been revised to 2012 through 2011 (100 years). A number of engineering activities had been well underway or substantially completed by the time the project lifetime window had been revised and as such the relative sea level rise values used for those activities were not revised to the period 2012 to 2100. It will be shown that the difference in selected sea level rise predictions accorded these time windows is small and would not be expected to materially change one's impression of project performance in and of itself.

1.6.1 Mississippi Coast Relative Sea Level Rise

Apparently, no long-term Mississippi coast tide gage records had been used to quantify relative sea level rise since 1947. Mississippi is the only Gulf Coast state for which this is true.

The National Oceanic and Atmospheric Administration (NOAA) is responsible for monitoring, forecasting, and publishing U.S. tide data. In 2001, NOAA published RSL estimates for all of its National Water Level Observation Network (NWLON) tide gage stations with records equaling or exceeding 25 years (Ref 5). Twenty-five years is considered the minimum record from which reasonably reliable sea level rise rates might be determined. There were no NWLON stations in Mississippi meeting this criterion and no RSL estimates were published.

The Permanent Service for Mean Sea Level (PSMSL) was established in 1933 at the Proudman Oceanographic Laboratory, Liverpool, England and collects and interprets sea level data from approximately 2,000 tide stations world-wide. The PSMSL regularly updates RSL estimates for most locations they monitor; one of these is the NOAA station at Bay Waveland Yacht Club (USGS station no. 8747437), though the period of record at that station is considered too short to provide a reliable estimate of RSL.

Mobile District has long-term tide gages at Gulfport (USACE station no. 02481341), Biloxi (02480351), and Pascagoula (02480301). A 1947 report of the Mobile District Engineer submitted to Congress for the Harrison County Beach Erosion Control Study reported, based on 49 years of record at the Biloxi gage, that annual mean stage was "rising gradually and is now approximately 0.3 of a foot higher than at the turn of the century."[2] RSL was computed for these and other stations for present purposes by using the method of least squares to fit a linear relationship to the monthly mean tide level (MTL). Monthly MTL is the average of the daily high and low water observations. The resultant rate was multiplied by 12 to arrive at the average annual RSL rate. Annual MTL values (the average of a calendar year's monthly MTL values) were also fit for comparison to RSL rates computed using monthly data. This method is similar to that employed by NOAA and the PSMSL, though there are differences. Monthly data were not weighted by the number of days in each month in computing the annual MTL. Records for years missing more than 3 months of data were discarded to minimize seasonal bias. The differences are not of significance here.

Computed RSL rates and the standard error of rates, in millimeters per year, spanning coastal Mississippi are presented in Table 1.6-1. Computed RSL rates from the Permanent Service for Mean Sea Level (PSMSL) web site, from NOAA's report (Ref. 5), and from Mobile District, USACE are shown for comparison. Large discrepancies in the rates are mostly attributable to the period of record analyzed; in general, rates computed from longer records are considered superior. Smaller discrepancies are due to differences in methods used to compute the rates.

Neither NOAA nor PSMSL had estimated rates for the Mississippi tide gages at Gulfport, Biloxi, nor Pascagoula. This is probably because these gages have historically been owned and operated by the Corps of Engineers, though the Corps turned the Biloxi gage over to NOAA (USACE continues to collect data from that gage) in September of 1999.

[2] As reported in U.S. House of Representatives Document No. 682, 80th Congress, 2nd Session, 1948. 'Annual Mean Stage' is therein defined as the average of all hourly tidal readings in a calendar year (8,760 readings might be obtained in one year) and is analogous to 'annual mean sea level.'

The Gulfport and Biloxi gages are in Harrison County. The Pascagoula gage is in Jackson County. Long-term gage data is not available for any locations in Hancock County, but data from short-record gages at Waveland Yacht Club (Y.C.) and Waveland were analyzed and are presented in Table 1.6-1 but application of those results is not recommended due to the short periods of record at these sites.

Table 1.6-1.
Relative Sea Level Rise Rates in the Vicinity of Coastal Mississippi

			USACE based on			
			Monthly Average Data		**Annual Average Data**	
			MTL	**MSL**	**MTL**	**MSL**
	PSMSL (2006)	**NOAA (2001)**	**mm/yr. + - std. error**			
Grand Isle, LA	9.52 +/- 0.37	9.85 +/- 0.35				
Record	1974-2003 (52 yrs.)	1947-1999				
Bay Waveland Y.C., MS	16.31 +/- 7.83	NA	5.40 +/- 0.17	5.21 +/- 0.16	4.65 +/- 2.04	4.44 +/- 2.06
Record	1987-1992 (5 yrs.)		1979-1992		1979-1992	
Waveland, MS	N/A	8.05 +/- 9.28*	9.33 +/- 0.42	10.58 +/- 0.41		
		1997-? 6 yrs.	1997-2005			
Gulfport, MS	N/A	N/A	1.70 +/- 0.04	N/A	1.96 +/- 0.7	N/A
Record			1964-1999		1964-2002	
Biloxi, MS	N/A	N/A	4.73 +/- 0.04	N/A	2.26 +/- 0.26	N/A
Record			1960-'98		1928-'76, '79-98	
Pascagoula, MS	N/A	N/A	2.9 +/- 0.04	N/A	3.72 +/- 0.30	N/A
Record			1960-'97		1940-'97	
Dauphin Island, AL	3.31 +/- 0.62	2.93 +/- 0.59	3.07 +/- 0.04	3.08 +/- 0.04	2.96 +/- 0.52	3.01 +/- 0.55
Record	1967-2003	1966-'97	1967-'68, '72-'74, '76-'80, '82-'97, '02-04		1967-'68, '72-'74, '76-'80, '82-'97, '02-04	
Pensacola. FL	2.12 +/- 0.17	2.14 +/- 0.15				
Record	1924-2003 (78 yrs.)	1923-'99				

*NOAA (2004) TR NOS/NGS 50.

Table 1.6-2 shows what were adopted, based on length of record, as the RSL rates for the vicinity of USACE gages in Mississippi. The computed rate for Biloxi, taken in conjunction with the 0.3 feet observed rise from 1900-1947 (1.94 mm/yr), suggests a 20th century relative sea level rise there of between 7.8 to 9.3 inches. These values may be compared to those computed by NOAA (Ref. 5) for all Gulf of Mexico tide stations with records exceeding 25 years in length shown in Table 1.6-3. The RSL rates computed for the Mississippi stations are lower than the average of all Gulf station values and consistent with those for coastal Florida and the southwestern Texas coast.

Table 1.6-2.
Relative Sea Level Rise as Indicated by USACE MS Coast Gages

Location	**Rise in mm/yr**	**Std. Error of Rise**
Gulfport, MS	1.70	0.04
Record	1964-1999	
Biloxi, MS	2.26	0.26
Record	1928-'76, '79-98	
Pascagoula, MS	3.72	0.30
Record	1940-'97	

Table 1.6-3.
Relative Sea Level Rise Rates at Various Gulf Coast Gages

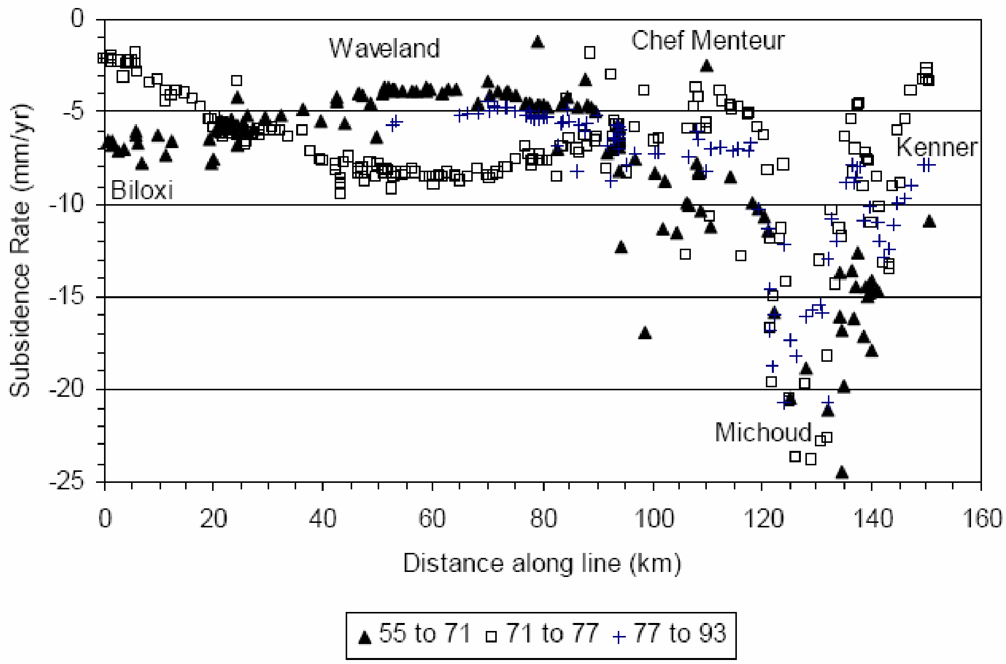
Station Name	First Year	Record Length	MSL Trend (mm/yr)	Std. Error (mm/yr)
Key West	1913	87	2.27	0.09
Naples	1965	35	2.08	0.43
Fort Meyers	1965	35	2.29	0.45
St. Petersburg	1947	53	2.4	0.18
Clearwater Beach	1973	27	2.76	0.65
Cedar Key	1914	86	1.87	0.11
Apalachicola	1967	33	1.53	0.58
Panama City	1973	27	0.3	0.64
Pensacola	1923	77	2.14	0.15
Dauphin Island, AL	1966	32	2.93	0.59
Grand Isle, LA	1947	53	9.85	0.35
Eugene Island, LA	1939	36	9.74	0.63
Sabine Pass, LA	1958	42	6.54	0.72
Galveston Pier 21, TX	1908	92	6.5	0.16
Galveston Pleasure Pier, TX	1957	43	7.39	0.53
Freeport, TX	1954	46	5.87	0.74
Rockport, TX	1948	52	4.6	0.41
Port Mansfield, TX	1963	35	2.05	0.75
Padre Island, TX	1958	37	3.44	0.56
Port Isabel, TX	1944	56	3.38	0.27
Average		49.2	4.00	
Weighted Average			4.06	
Median		42.5	2.85	

From Ref. 5.

1.6.1.1 Mississippi Coast Subsidence

NOAA’s National Geodetic Survey published (Ref. 2) estimated relative vertical displacement rates for areas including the Mississippi Gulf Coast in 2004. The rates were estimated by adjusting historic first-order leveling runs according to the estimated historic elevation of a Louisiana tide gage benchmarks in Louisiana. The elevation of said benchmark at the time of the historic leveling surveys was estimated by assuming it was subjected to an averaged eustatic sea level rise of 1.25 mm/year. This value is comparable to the 1.20 mm/year given in NRC’s document for 20th century eustatic sea level rise. The leveling data were then adjusted to estimate their true elevations at the time of the surveys; the elevation of a given point in one year was then compared to the elevation of the same point during a following survey many years later, which gave an estimate of the rate of subsidence. Results for surveys generally following the east-west alignment of the CSX railway line across Mississippi are shown in Figure 1.

Figure 1.6-1 gives estimated subsidence rates for the periods 1955 to 1971, 1971 to 1977, and 1977 to 1993. The figure suggests that Mississippi coast subsidence varies from about -2 mm/year to -9 mm/year and that the average subsidence is on the order of -6 mm/year (negative values imply that the ground is subsiding). It is interesting that series of the Mississippi portion of the comparison appear to be reflections of each other, though the reason for this is not apparent. The average subsidence rate, approximately 6 mm/year, of the railway line near the coast exceeds the RSL rates (1.7 to 3.7 mm/yr) determined from tide gage data.



From Ref. 2 Appendix 4.

Figure 1.6-1. Biloxi, MS to New Orleans, LA Subsidence Rates for Periods Indicated in Years

If equal confidence is given to both the RSL rate and the subsidence data, one must conclude that eustatic sea level in the Gulf of Mexico is not rising, but falling; this is doubtful. Therefore, it appears that either the gage data is erroneous, the subsidence estimates are flawed, or both. While it is most likely that neither the subsidence estimates nor the RSL estimates are infallible, the RSL rates are generally consistent with those observed elsewhere along the Gulf Coast, excepting those areas in Louisiana which are known to subside at abnormally large rates. This suggests that, while subsidence is probably occurring in Mississippi, tide gage data suggest that it may be occurring over much of the Mississippi coast at a rate that is consistent with Gulf Coast locations not associated with Mississippi River Delta formations. Also, since the question as to why subsidence estimates, taken in conjunction with tide gage data, suggest that sea level is dropping, as opposed to rising, remains unresolved, there is at present no clear rationale nor means to employ these subsidence estimates for purposes of estimating future RSL.

1.6.2 Projected Sea Level Rise

Table 1.6-4 shows extrapolated RSL for the period 2005-2100 based only on the rates derived from historic USACE station records (Table 1.6-2). The total relative rise predicted for the 95 year period is consistent with that suggested by Biloxi gage records over the 20th century.

Table 1.6-4.
Relative Sea Level Rise Assuming Observed Rates Persist, 2005-2100

Gulfport		Pascagoula		Biloxi	
meters	**feet**	**meters**	**feet**	**meters**	**feet**
0.16	0.53	0.35	1.16	0.21	0.70

1.6.2.1 National Research Council (NRC) Methods

Corps of Engineers guidance recommends Curve 3 of the NRC report (Ref. 3), or more definitive information, be used as the eustatic component of RSL for future high scenario estimates.

The NRC produced three curves, Curves 1, 2, and 3, which might be thought of as low, medium, and high rate of rise estimates due to climate change and are reproduced here as Figure 1.6-2. These curves were developed based on studies published between 1983 and 1986 and assume in global eustatic sea level of 0.5 meters, 1 meter, and 1.5 meters, respectively between 1986 and 2100. The curves are a function of time squared and thus suggest that the rate of sea level rise will increase into the future, though as of 2001, no such increase had been detected (Ref. f). The suggestion that sea level rise rates will increase in the future is common to all reports reviewed. These curves yield high, medium, and low eustatic sea level increases of 0.47 m., 0.95 m., and 1.44 m. (1.54 ft., 3.13 ft., 4.72 feet) respectively for the period 2005 to 2100. Relative sea level rise for a given location at the

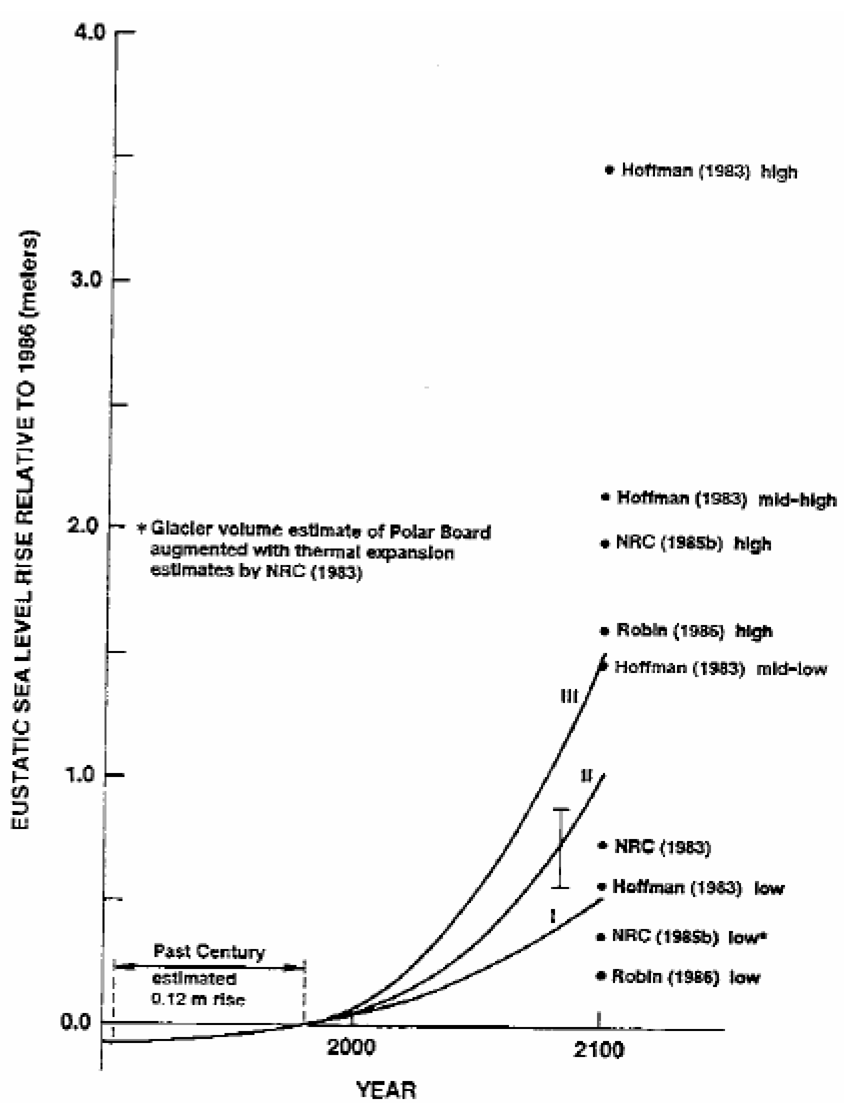


Figure 1.6-2. Eustatic Sea Level Rise Scenarios (Figure 2-2 from NRC, 1987)

year 2100 would be arrived at by adding these values to the locally predicted subsidence, where the local subsidence would be the observed (or estimated rate where observations were not available) subsidence rate multiplied by the time span (in this case, 2100-2005 = 95 years). The document implies that local subsidence rates might be estimated by subtracting 1.2 mm/year (the assumed rate of global eustatic sea level rise) from RSL rates computed from tide gage data.

Relative sea level rise estimates for the period 2005 to 2100 at the locations of coastal Mississippi USACE tide stations using NRC methods are shown in Table 1.6-5. The values in the table have been computed converting the gage RSL rates to subsidence rates by subtracting 1.2 mm/year as suggested by the NRC. The total rise given in this table for Curve 3 is five to eight times those predicted by extrapolation of rates computed from historic gage data (Table 1.6-4).

Table 1.6-5.
Relative Sea Level Rise Estimates by NRC (1987) Methods, 2005-2100

	Gulfport		Pascagoula		Biloxi	
Basis	**meters**	**feet**	**meters**	**feet**	**meters**	**feet**
Curve 1	0.51	1.69	0.71	2.32	0.57	1.86
Curve 2	1.00	3.28	1.19	3.91	1.05	3.46
Curve 3	1.49	4.88	1.68	5.51	1.54	5.05

1.6.2.2 Environmental Protection Agency (EPA) Methods

The EPA (Ref. 6) estimated future eustatic sea level rise and also attempted to identify the probability distribution of this rise occurring. The EPA report is the only report reviewed which has attempted to assign probabilities to the sea level rise phenomenon. The one percent, mean, and 99 percent non-exceedance eustatic sea level rise estimates for the time interval 1990 to 2100 are -0.01 m. (-0.03 ft.), 0.34 m. (1.11 ft.), and 1.04 m. (3.41 ft.). Estimates are also provided by EPA for the years 2050 and 2200. As with the other eustatic sea level rise forecasts discussed in this document, the estimates account for only those changes in sea level which might be attributed to climate change.

The EPA report recommends a simple procedure for estimating regional sea level rise based on their eustatic sea level rise estimates. The procedure is to add a normalized projection to the current (observed) relative rate of sea level rise as given by the following equation:

Local (t) = normalized (t) + (t -1990) x trend Eqn. 1.6-1

Where: Local (t) is the projected local rise in sea level in some future year t.

Normalized (t) is the normalized eustatic rise given by Ref. 6 Table 9-1.

Trend is the observed trend at a representative gage location.

The ‘normalized’ eustatic rise value was developed by EPA in order to avoid double-counting the effects of the historic contribution of climate change, which are inherent in the observed trend value; double-counting would occur were future projections made using the predicted (as opposed to the normalized) eustatic sea level rise values in this equation. This concern over double counting does not come into effect if the predicted eustatic sea level rise contribution were to be combined with a known local subsidence rate.

EPA methods were used to develop sea level rise estimates for the period 2005 to 2100 for the vicinities of the USACE tide gages at Biloxi, Gulfport, and Pascagoula. The 50% and 99% non-exceedance eustatic normalized sea-level rise predictions were used in conjunction with the historically observed rates for this purpose. Results are shown in Table 1.6-6. These values compare favorably to values give by NRC Curves 1 and 2 but are, as a rule, much lower than those given by Curve 3 (see Table 1.6-5). The 50% values are approximately 0.7 to 0.8 feet higher than those predicted by historical rates alone (see Table 1.6-4).

Table 1.6-6.
Relative Sea Level Rise Estimates by EPA (1995) Methods, 2005-2100

Location	m	feet
Gulfport 50%	0.39	1.3
99%	0.99	3.2
Biloxi 50%	0.44	1.4
99%	0.98	3.2
Pascagoula 50%	0.60	2.0
99%	1.18	3.9

1.6.2.3 Intergovernmental Panel for Climate Change (IPCC) Methods

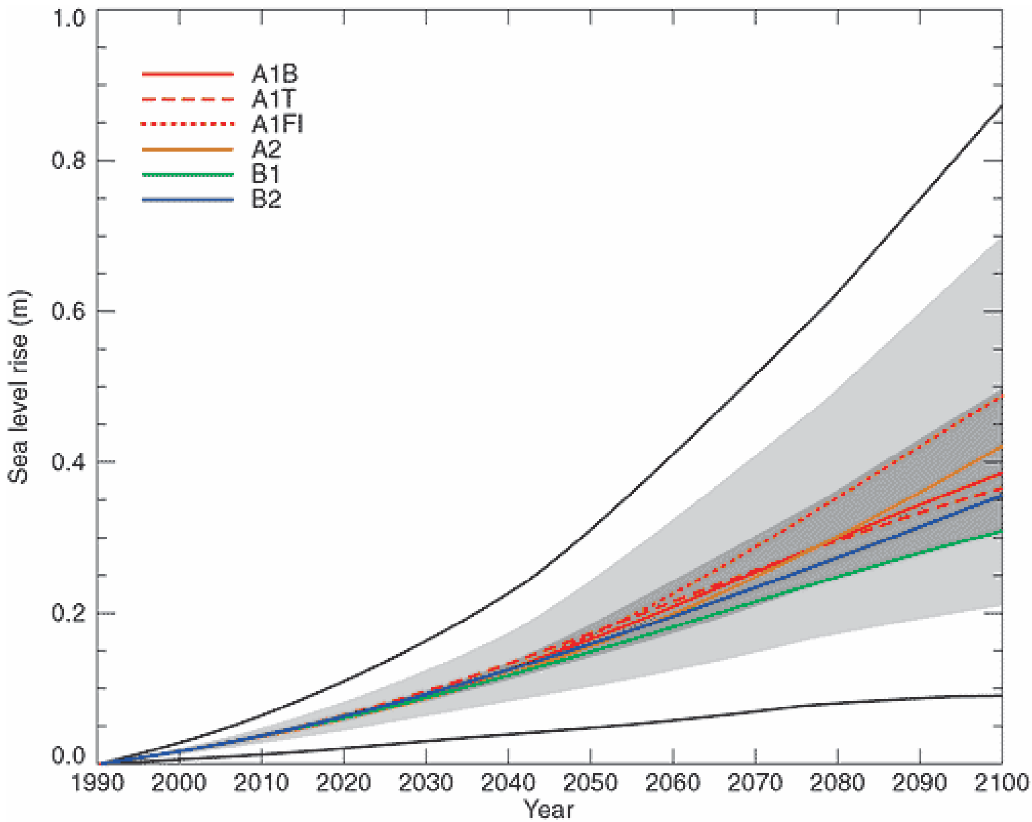
The Climate Change 2001 document (Ref. 7) by the IPCC is the most current and comprehensive publication available for this effort regarding the science of climate change and its implications for sea level rise. An updated IPCC climate change report is due in May of 2007 but due to the MsCIP schedule it arrives too late to be of use for estimating future sea level rise, though a summary of the findings of that report, released in early 2007, suggest that the global eustatic sea level rise central value estimate has not changed significantly.

The full suite of IPCC sea level rise projections is shown in Figure 1.6-3. The projections result from over 35 climate change scenarios, run in a number of different global circulation models. The projections represent the contribution of climate change to global average sea level rise. The IPCC predicts eustatic sea level rise of 0.09 to 0.88 m between 1990 and 2100 with a central value of 0.48 meters. The central value averaged over this time period is 4.36 mm/year, which is two to four times the average rate suggested by 20th century Mississippi Coast tide gage data.

In Figure 1.6-3:

- The black outer lines describe the range of all model estimates, including sensitivity to land ice withdrawal, sedimentation, and other assumptions.
- The lightly-shaded region shows the range from 35 scenarios tested in all circulation models.
- The darkly-shaded region shows the range of the average of those scenarios.
- The colored lines represent the computed average from each demonstration model, which are a subset of the 35 scenarios.

The values shown in Figure 1.6-3 are lower than those suggested in the earlier NRC study, and similar to those in the EPA publication. Since publication of the NRC document, estimates of the magnitude of future global warming have been cut in half, and this resulted in a reduction of the range of estimates of future sea level rise. The IPCC document suggests this reduction is primarily due to improvements in technology, improvements in the understanding of pollutant behavior (particularly aerosols), and revised pollutant discharge forecasts.



From Ref. 7 Figure 11.12.

Figure 1.6-3. IPCC Global Eustatic Sea Level Rise Estimates

The IPCC does not prescribe how these global eustatic sea level rise estimates might be adapted to estimate future local relative sea level rise. Like the EPA before it, the IPCC acknowledges double-counting as a valid issue, but does not provide normalized sea level rise estimates for use with their eustatic sea level rise estimates, nor do they provide explicit instructions for adapting their predicted sea level rise values to specific locations. The IPCC report does, however, suggest that the approach advocated in EPA's report might be used.

Comparison of EPA's normalized and non-normalized eustatic sea level rise estimates (Tables 9-1 and 7-4, respectively, Ref. 6) reveal that EPA estimates the 20th century climatological contribution to sea level rise at 0.82 mm/year. This contribution is essentially constant throughout the range of EPA's probable sea level rise estimates. This value is reasonably consistent with IPCC's central value (c.v.) estimate of said contribution at 0.7 mm/year (Table 11.10, Ref. 7). Since EPA has applied a constant normalizing rate adjustment, it may be argued that the IPCC's estimate of 0.7 mm/year might be used in the same manner to normalize IPCC's estimate and facilitate use with RSL rates obtained from gage data.

Therefore, future estimates of local sea level rise over the interval 2005 to 2100 might be obtained using IPCC values as follows:

Local rise = [(IPCC 2100 – IPCC 2005) – n*(2100-2005)] + (2100-2005) * trend Eqn. 1.6-2

Where: IPCC is the projected eustatic rise in sea level for the given year, from Figure 3 herein.

Trend is the observed trend at a representative gage location.

n is the normalizing factor = 0.7 mm/yr.

The normalization function is shown in brackets on the right-hand side of Equation 1.6-2.

Results of this method applied to the vicinity of the USACE gages are shown in Table 1.6-7.

Table 1.6-7.
Relative Sea Level Rise Estimates using IPCC Predictions, 2005-2100

Location	m	feet
Gulfport c.v.	0.54	1.8
high	0.96	3.2
Biloxi c.v.	0.60	2.0
high	1.02	3.3
Pascagoula c.v.	0.74	2.4
high	1.16	3.8

1.6.3 Relative Sea Level Rise Summary

Corps of Engineers Planning Guidance Notebook (Ref. 4) states that potential relative sea level rise should be taken into consideration for coastal or estuarine projects at the feasibility level of study and recommends. At a minimum, project performance would be evaluated based on extrapolation of the observed historic rate and should also consider a higher rate, based on NRC Curve 3 or more definitive data, than that historically observed.

'High,' 'medium,' and 'low' eustatic sea level rise estimates as given in the NRC in 1987 and more recent authoritative reports by the EPA (1995) and IPCC (2001) reports are summarized below in Table 1.6-8. The values in the table are eustatic values only and do not reflect local nor historic trends at the Mississippi Coast. While the three methodologies differ slightly, they commonly adjust, according to each agency's prediction of climate change effects, extrapolated historic local relative sea level rise. In other words, the only difference in the predicted RSL for each scenario at each location is that portion of rise attributed to possible effects of climate change. The table shows that the climate-driven component of sea level rise estimates has dropped substantially since publication of the NRC report, primarily due to advances in global fluid dynamics modeling technology and revised pollutant discharge estimates.

Table 1.6-8.
Comparison of Eustatic Sea Level Rise Predictions, 1990-2100

	Low, in m. (ft)	Medium, in m. (ft)	High, in m. (ft)
NRC[1] (1987)	0.47 (1.53)	0.95 (3.13)	1.44 (4.72)
EPA[2] (1995)	-0.01 (-0.03)	0.34 (1.11)	1.04 (3.41)
IPCC[3] (2001)	0.09 (0.29)	0.48 (1.57)	0.88 (2.89)

Notes: 1. NRC (1987) curves 1, 2, and 3 respectively. 1%, 50%, and 99% non-exceedance probabilities, respectively. From Ref. 6 Table 7-3. 2. Low and high values represent the extreme range with uncertainty and the medium value is the 'central estimate.'

The observed historic relative rates of relative sea level rise along the Mississippi Gulf Coast were determined from long-term USACE tide gage data and are summarized in Table 1.6-2. The rates are typical of RSL rates determined from other long-term Gulf of Mexico tide and lower than rates observed in Louisiana and eastern Texas.

Extrapolation of historically observed Mississippi coast RSL rates results in a relative sea level rise of 0.5 to 1.2 feet for the period 2005 to 2100. Extrapolation of observed rates is inconsistent with the climate change community view, held since at least the early 1980's, that sea level rise will accelerate in the 21st century. The USACE Mississippi coast gage data have not been interrogated to detect RSL rise acceleration for present purposes.

Subsidence rates and magnitudes have been estimated for the MS Coast but the estimated rates, weighed in consideration of RSL rates derived from tide gage data, do not seem to support the generally accepted view that sea level is rising. The reason this is so cannot at present be resolved and therefore the subsidence estimates were not used in favor of RSL rates determined from tide gage records to estimate future sea level rise.

Future relative sea level rise estimates were developed using NRC, EPA, and IPCC projections. The IPCC estimates are the most current available. The findings are summarized in Table 1.6-9 and rounded to the nearest 0.1 feet.

IPCC's 'high' values compare to within 0.1 feet of those computed using EPA's 99% non-exceedance values, while IPCC's central value (c.v.) estimates are slightly higher than those yielded using EPA's 50% (mean) normative sea level rise values. The 'high' and c.v. are similar to values yielded using NRC's Curve 1 ('low') and Curve 2 ('expected'; see Table 1.6-5).

The IPCC 2001 predictions were the most current and definitive available. The IPCC 'high' values were selected for evaluating project performance as the 'higher than observed rate' versus those predicted using EPA and NRC methods because the IPCC values are more recent and more widely (globally) used. In a subtle departure from USACE guidance, relative sea level rise values based on IPCC 'expected' (also referred to as 'medium' and 'central value') eustatic sea level rise predictions were adopted for present study purposes in lieu of rise computed using extrapolated historic rates because most experts believe that the rate of sea level rise will increase in this century and extrapolated historic rise assumes past relative sea level rise rates will persist.

Table 1.6-9.
Comparison of Computed Relative Sea Level Rise Estimates, 2005-2100

	Gulfport		Pascagoula		Biloxi	
Basis	**High[1] m. (ft)**	**Expected[2] m. (ft.)**	**High[1] m. (ft)**	**Expected[2] m. (ft.)**	**High[1] m. (ft)**	**Expected[2] m. (ft.)**
Extrapolated	-	0.16 (0.5)	-	0.35 (1.2)	-	0.21 (0.7)
NRC (1987)	1.49 (4.9)	1.00 (3.3)	1.68 (5.51)	1.19 (3.9)	1.51 (5.0)	1.05 (3.5)
EPA (1995)	0.99 (3.2)	0.39 (1.3)	1.18 (3.9)	0.60 (2.0)	0.98 (3.2)	0.44 (1.4)
IPCC (2001)	**0.96 (3.2)**	**0.54 (1.8)**	**1.16 (3.8)**	**0.74 (2.4)**	**1.02 (3.3)**	**0.60 (2.0)**
	Values in **bold** are adopted.					

Notes: 1. NRC Curve 3; EPA 99% non-exceedence; IPCC upper-bound. 2. NRC Curve 2; EPA 50% non-exceedeence; IPCC 'central value'.

It was mentioned earlier that the project lifetime evaluation period had been revised from 2005 – 2100 to the 100-year period 2012 through 2111 after the time had passed for which RSL revisions might have been able to have been incorporated into related engineering efforts. RSL estimates were generated for the revised time frame using the IPCC predictions and compared to the adopted results from the 2005-2100 time frame as in Table 1.6-10. The project lifetime shift results in about 0.2 feet (2.4 inches) difference which is believed to be insignificant for present purposes.

Table 1.6-10.
Comparison of Adopted RSL 2005-2100 Versus Computed 2012-2111 RSL

	Gulfport		Pascagoula		Biloxi	
Time Frame	**High[1] m. (ft)**	**Expected[2] m. (ft.)**	**High[1] m. (ft)**	**Expected[2] m. (ft.)**	**High[1] m. (ft)**	**Expected[2] m. (ft.)**
2005-2100	**0.96 (3.2)**	**0.54 (1.8)**	**1.16 (3.8)**	**0.74 (2.4)**	**1.02 (3.3)**	**0.60 (2.0)**
2012-2112	1.01 (3.3)	0.60 (2.0)	1.21 (4.0)	0.81 (2.6)	1.05 (3.5)	.66 (2.2)
	Values in **bold** are adopted.					

Notes: 1. IPCC upper-bound. 2. IPCC 'central value'.

1.6.4 Relative Sea Level Rise Application

Plan formulation has identified three RSL scenarios to be evaluated over the project lifetime: (1) existing sea level; (2) 'expected' sea level rise, and (3) 'high' sea level rise. Existing sea level was selected primarily for exploratory comparative economic analysis of damage attributable to sea level rise in and of itself, which can be inferred by comparing storm damages due to storm surge at existing sea level against storm damage due to storm surge at some higher sea level. Note that there is no accompanying expectation or recommendation that any storm damage reduction system or element would be formulated or proposed based on a future sea level as it exists today. Expected relative sea level rise is interpreted to be that prediction based on IPCC's 'central value' eustatic sea level rise, and 'high' sea level rise was adopted based on the upper bound of IPCC's scenario testing results. Results are consistent with the level of detail appropriate for present needs but should be viewed as a 'first cut' at identifying historic and predicted relative sea level rise in the vicinity of coastal Mississippi.

The effects of sea level rise are many. From a practical standpoint, it is impossible to thoroughly explore all ramifications of sea level rise. Sea level rise implications will be tested in economic terms using the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) software, and the Engineer Research and Development Center's BeachFX program; these efforts are discussed in Chapter 2 of this Engineering Appendix, and related plan formulation considerations are discussed in the main body of this report. Flood damage evaluations will also be performed in HEC-FDA over a 50-yr planning period to test the sensitivity of economic damages to the assumed project lifetime; computed relative sea level rise values using IPCC (2001) eustatic sea level rise predictions for this purpose are shown in Table 1.6-11. Coastal levee construction cost and levee protection implications are also discussed in Chapter 3 of this report.

Table 1.6-11.
Computed 50-year Relative Sea Level Rise Estimates, 2005-2055

	Gulfport		Pascagoula		Biloxi	
	High m. (ft)	**Expected m. (ft.)**	**High m. (ft)**	**Expected m. (ft.)**	**High m. (ft)**	**Expected m. (ft.)**
Extrapolated	-	0.09 (0.3)	-	0.19 (0.6)	-	0.11 (0.4)
IPCC (2001)	0.40 (1.3) [1]	0.26 (0.9) [2]	0.60 (2.0) [1]	0.46 (1.5) [2]	0.46 (1.5) [1]	0.32 (1.0) [2]

Notes: 1. IPCC upper-bound. 2. IPCC 'central value'.

Future design and evaluation efforts will require that these relative sea level rise predictions be updated, as (a) the IPCC published updated climate change effects documents in May of 2007 and (b) there are opportunities to improve local relative sea level rise estimation and prediction methods versus the status quo methodologies presented herein.

1.6.5 References

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PART 2. LONG-TERM ENGINEERING SOLUTIONS

2.1 Long-term Engineering Solutions

With the task of developing a hurricane damage reduction plan for the coast of Mississippi, several issues had to be considered. First, it had to be technically feasible. Could a storm damage reduction system be designed that would be constructible and at the same time not destroy what it was supposed to help protect? It had to be reliable so when needed, it would do the job it was designed for. It also needed to be cost effective. This system also had to integrate into other storm reduction concepts such as non-structural solutions such as buy-out programs and re-establishing some areas as environmental components of the plan. The development along the coast had some areas that were not contiguous to highly developed areas like found in Harrison County where the entire coastline is densely developed. These outlying areas may require individual means for any storm damage reduction. Almost any project along a coastline has environmental concerns and this is true in Mississippi. In Jackson County, the Pascagoula River system separates the city of Pascagoula from most of the coast to the west. This river system in one of the last major free-flowing rivers in the southeast and is home to endangered fish. In the western portion of the state, extensive marshes create other concerns along with the Pearl River that separates Mississippi from Louisiana. Other technical issues also made working in this river problematic. Another issue that was voiced early in this project was the population did not want a high structural seawall along the coast. The concern of losing the view of the water and beaches was repeated consistently in public meetings.

Review of the coastline in Mississippi using aerial photographs, topographic maps, LIDAR surveys, and storm inundation data revealed that natural topography could play a major role in forming storm barriers. Other features such as the offshore barrier islands, extensive beaches in many areas, and existing beach-front roadways were also realized as having a role in formulating a storm defense system. An existing railway track crosses the entire state near the coast and in the typical fashion of railways, these tracks follow high ground. This same general alignment was judged to be favorable for any type of inland barrier.

Review of the inundation maps from Katrina also revealed the extensive low-lying areas associated with two bays that extend inland from the coast. It was apparent that any storm protection systems would have to consider these as breaks in the line. Closing off rivers and bays with surge gates are used in Europe to protect inland areas and these type structures could be considered for Mississippi.

During planning sessions with the project delivery team, a structural “Lines of Defense” (LOD) concept was drafted that started with the offshore barrier islands and progressed inland to what could be considered the worst possible scenario with a extremely large hurricane, even worse than Katrina. Research identified numerous methods that have been developed to provide protection from flooding. Along with the traditional methods of levee or structural seawall construction, many other types of protection were reviewed. These included inflatable barriers, concrete sidewalks or roadways that could be hydraulically rotated upwards to form a seawall, sliding panel gates, offshore breakwaters, and many types of surge barriers to close off the bays. The lines would also provide increasing levels of protection as you transgressed inland. It was understood that some lines would not provide protection from large storms. It was also evident that several areas of the coast could not be included in continuous line of defense and would be either placed in a ring levee system or designated to a non-structural solution.

In the early stages of the study, it was understood that the results of proposed storm surge modeling would not be available to the designers. These studies would be used to develop new stage-frequency curves to predict a wide range of storm surge for the entire cost of Mississippi based on a large suite of storms. This modeling effort would also provide a prediction of the largest hurricane surge event that is considered possible along the coast of Mississippi. This storm, labeled the Maximum Possible Intensity (MPI) event would be used to define a line, based on ground surface elevation that a storm surge would not exceed.

From the planning session came five conceptual lines of defense. The general concept for this plan was made during project delivery team meetings that included engineers, environmentalists, planners, and geologists. Information from along the coastline was gathered that included large scale aerial photography, topographic maps, navigation maps, and a large collection of pre and post-Katrina photographs. As this discussion progressed, a color illustration, shown in Figure 2.1-1, was drawn that evolved into the five lines of defense that is the foundation to the structural aspect of this study. A refined version by a graphic artist, Figure 2.1-2, was completed based this initial sketch.

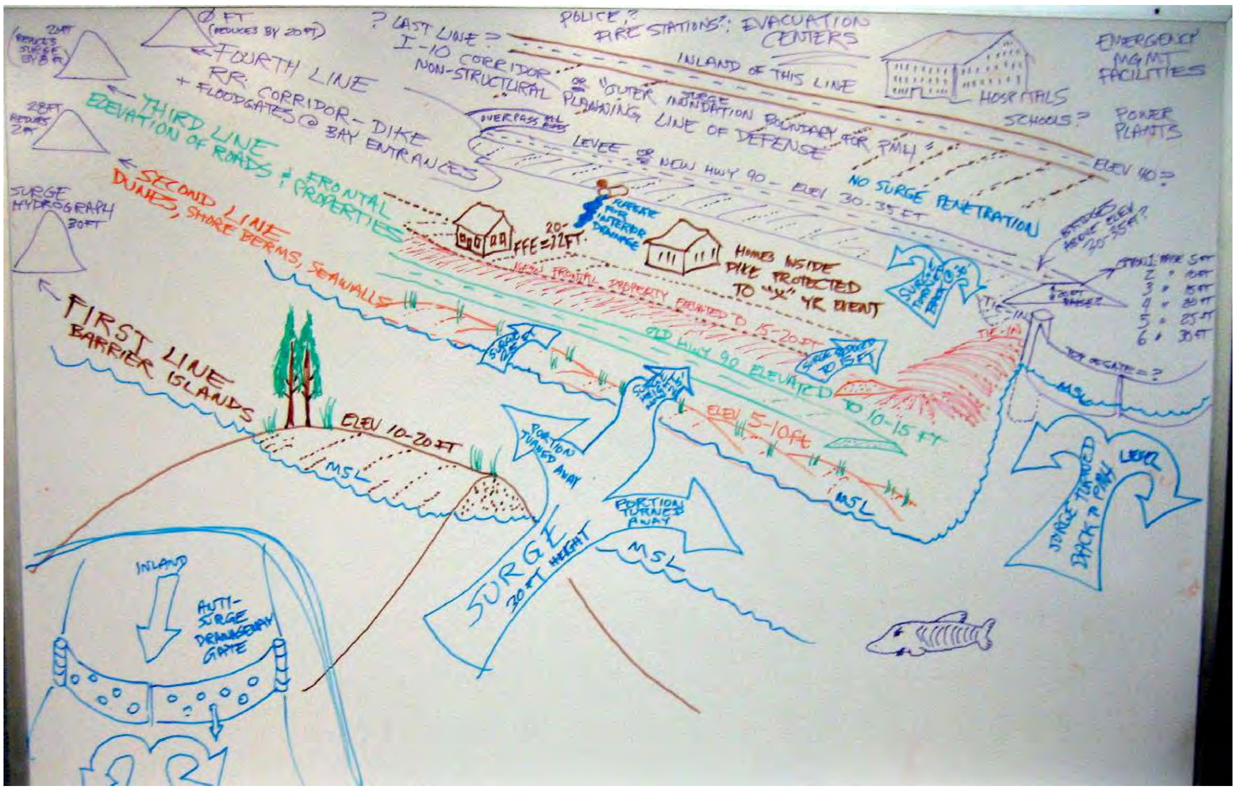


Figure 2.1-1. Graphic developed during initial planning sessions that visualized a "Lines of Defense" approach

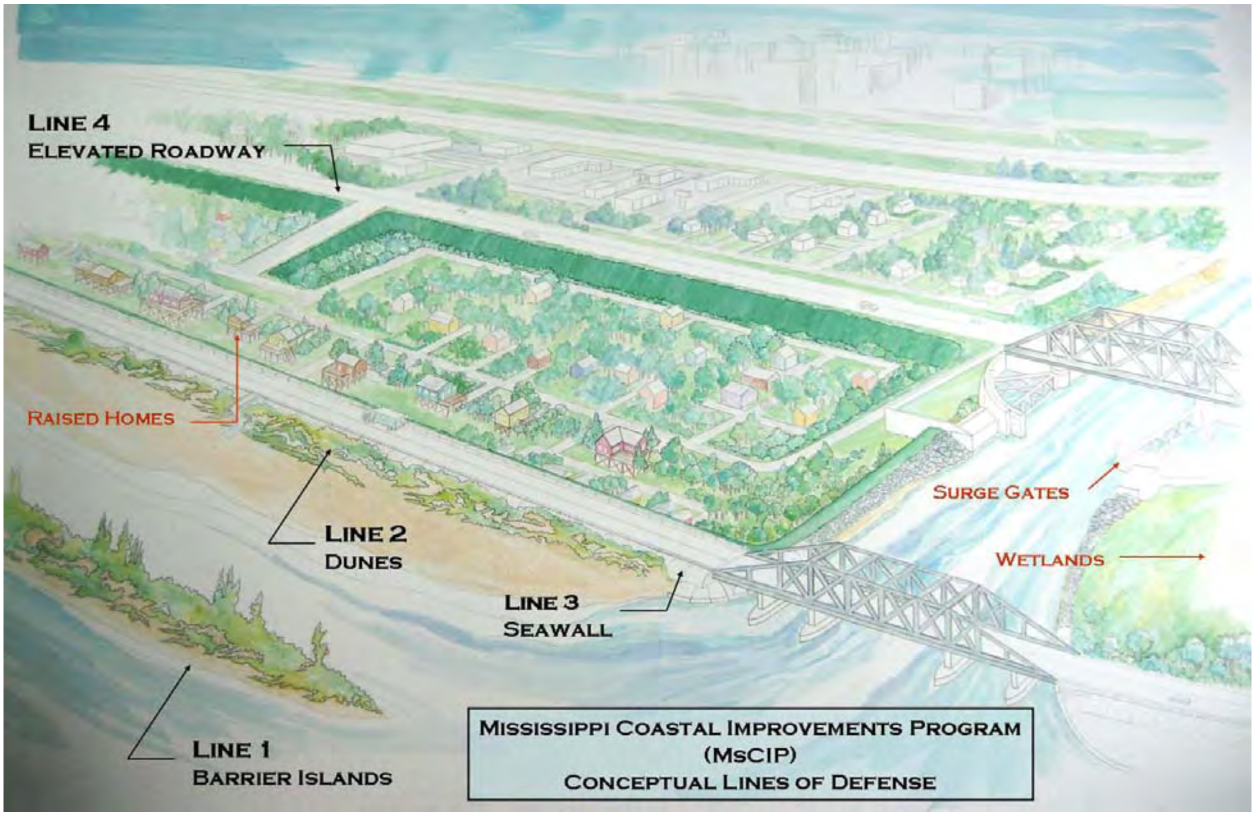


Figure 2.1-2. Artist's conceptual drawing based on the initial vision for Lines of Defense (Dawkins, 2006)

The first apparent feature to be discussed was the offshore barrier islands that had been included in the Mississippi Governor's recovery plan. Designated as LOD 1, the barrier islands have been eroded by numerous storms. In 1969, Hurricane Camille caused extensive erosion on the islands and created a large breach in Ship Island. After Katrina, it was widely expressed that if the islands had been in a pre-Camille condition, the storm surge would have been much less along the mainland coast. It was decided to model that scenario to help predict what effects the islands play in storm reduction.

The beaches (manmade in the 1950s) that extend along much of the coast were also considered as a feature that could be modified to provide some level of protection by the inclusion of dunes on the beaches. Other projects were underway to improve the some of the beaches and proposed projects would construct small dunes on most of the beaches. Improving on these by studying dunes at crest elevations of 10.0 (NAVD88) and 15.0 (NAVD88) was designated as LOD-2. These would not provide protection from large storms, but would be beneficial for smaller storms and would provide recreational and environmental benefits.

Another existing condition along the coast is roadways that coincide with the beaches. It was envisioned that raising these roadways would have minimal environmental impact and provide the first hardened barrier to surge damage. These roadways, while not continuous along the coast, were designated as LOD-3. Elevations of 12.0 (NAVD88), 18.0 (NAVD88) and 24.0 (NAVD88) were initially selected for study. It was also recognized that LOD-3 would require that barrier be placed at the mouths of the bays to be effective.

Some areas of the coast were not associated with these beaches and existing roadways or for environmental and/or technical reasons could only be viewed as stand alone projects such as ring levees. These areas included six communities in Jackson County and one in Hancock County. For discussion purposes, these were also included in LOD-3 and would be studied at the same proposed elevations.

Further inland, the existing railroad grade had provided a levee-like barrier to storm surge from Katrina in some areas. Using the same high-ground alignment, an inland barrier was envisioned that could be constructed to such an elevation as to protect from a large storm surge, even larger than Katrina. Like LOD-3, this system would require that the bays be closed off from surge to be effective. As LOD-4, this barrier was to be studied at elevations of 24.0 (NAVD88), 32.0 (NAVD88) and 40.0 (NAVD88). Many alignments were considered before one that was recommended due to technical and environmental reasons. This system would not cross the Pearl River on the western side of the state nor the Pascagoula River in Jackson County.

For the highest level of protection from the largest storm surge event, the limits of surge predicted from the MPI event was transposed to maps and while a non-structural measure, it was designated as LOD-5. It would be an area north of any potential surge damage that would be recommended for location of critical infrastructure such as hospitals and emergency facilities.

Figure 2.1-3 represents a section extending from the barrier islands to the MPI line.

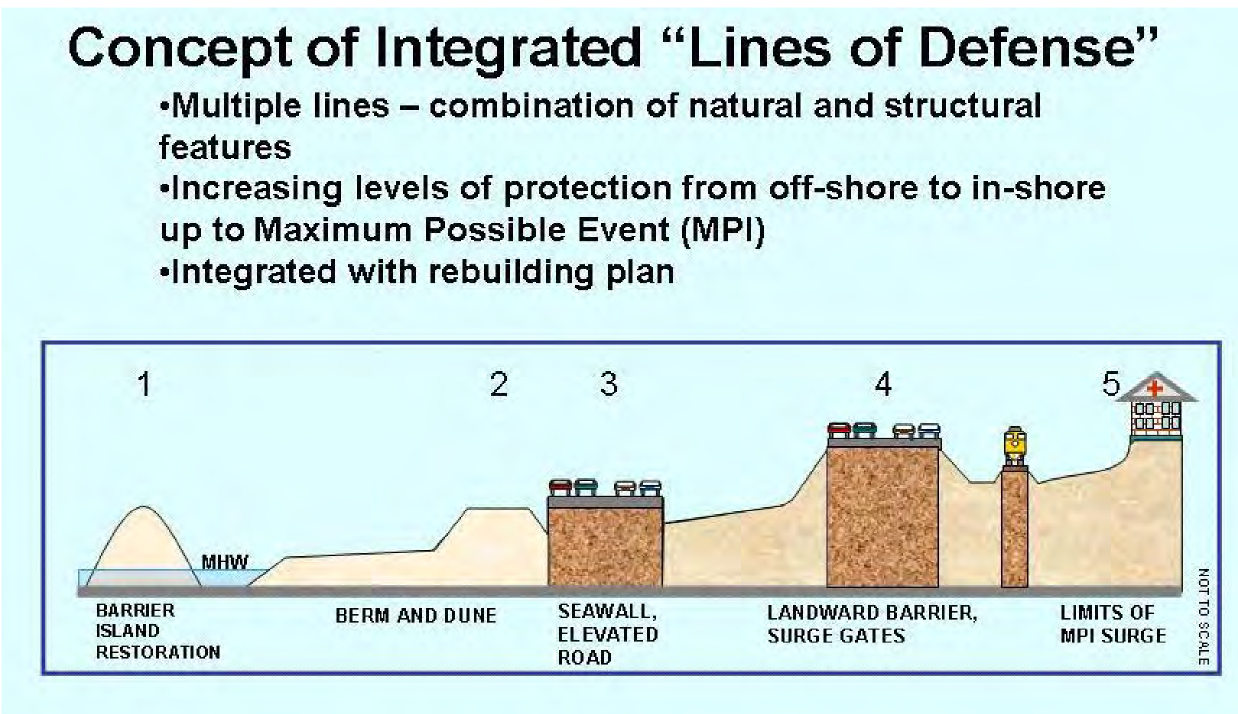


Figure 2.1-3. Conceptual section that includes five lines of defense extending from the barrier islands inland to the upper limits of the maximum possible intensity (MPI) hurricane

The proposed alignments for the LODs in each of the three coastal counties are shown in Figures 2.1-4, 2.1-5 and 2.1-6.

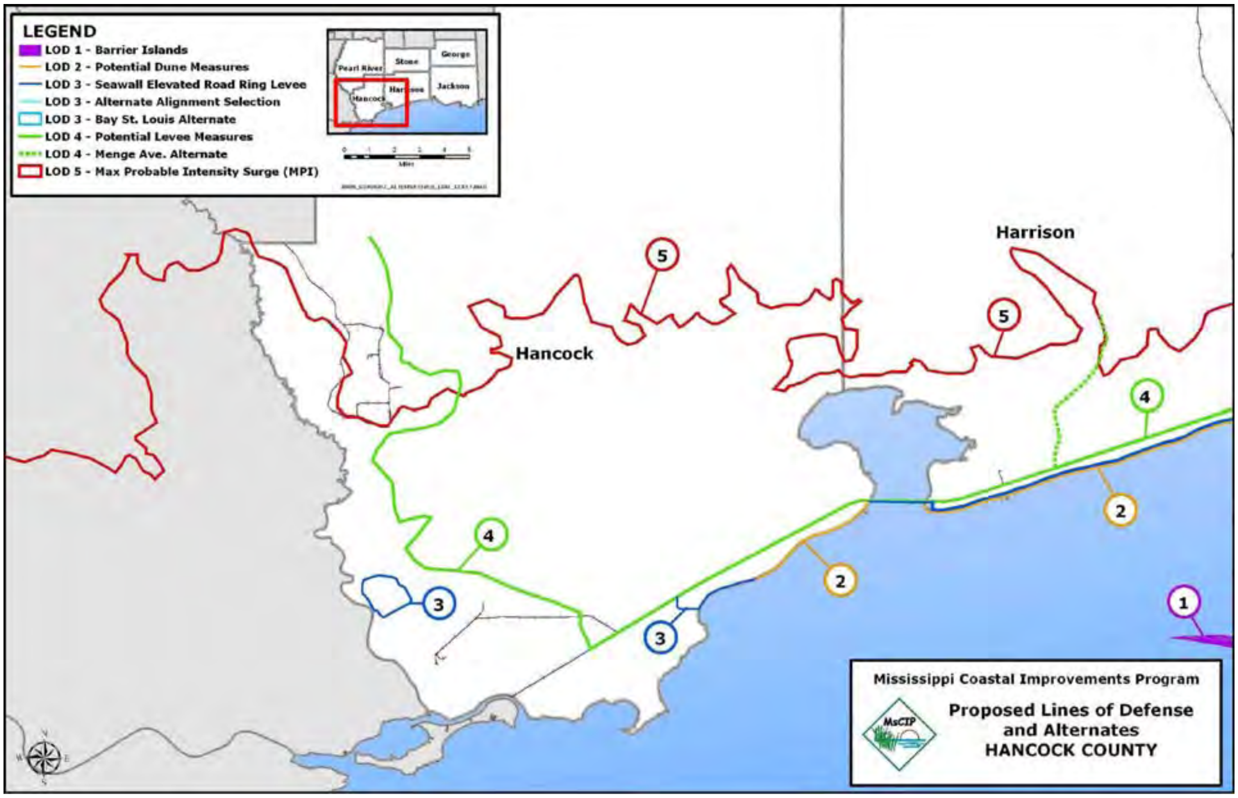


Figure 2.1-4. Line of Defense Alignments in Hancock County

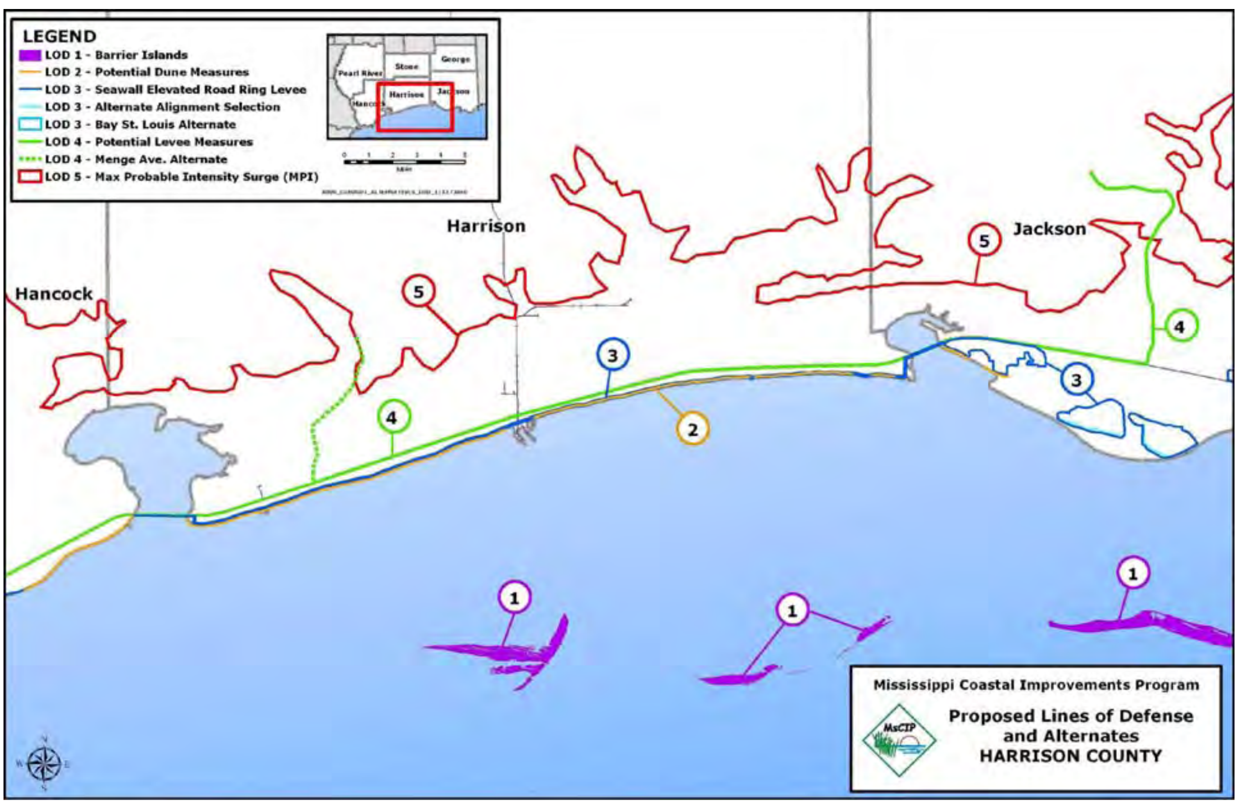


Figure 2.1-5. Line of Defense Alignments in Harrison County

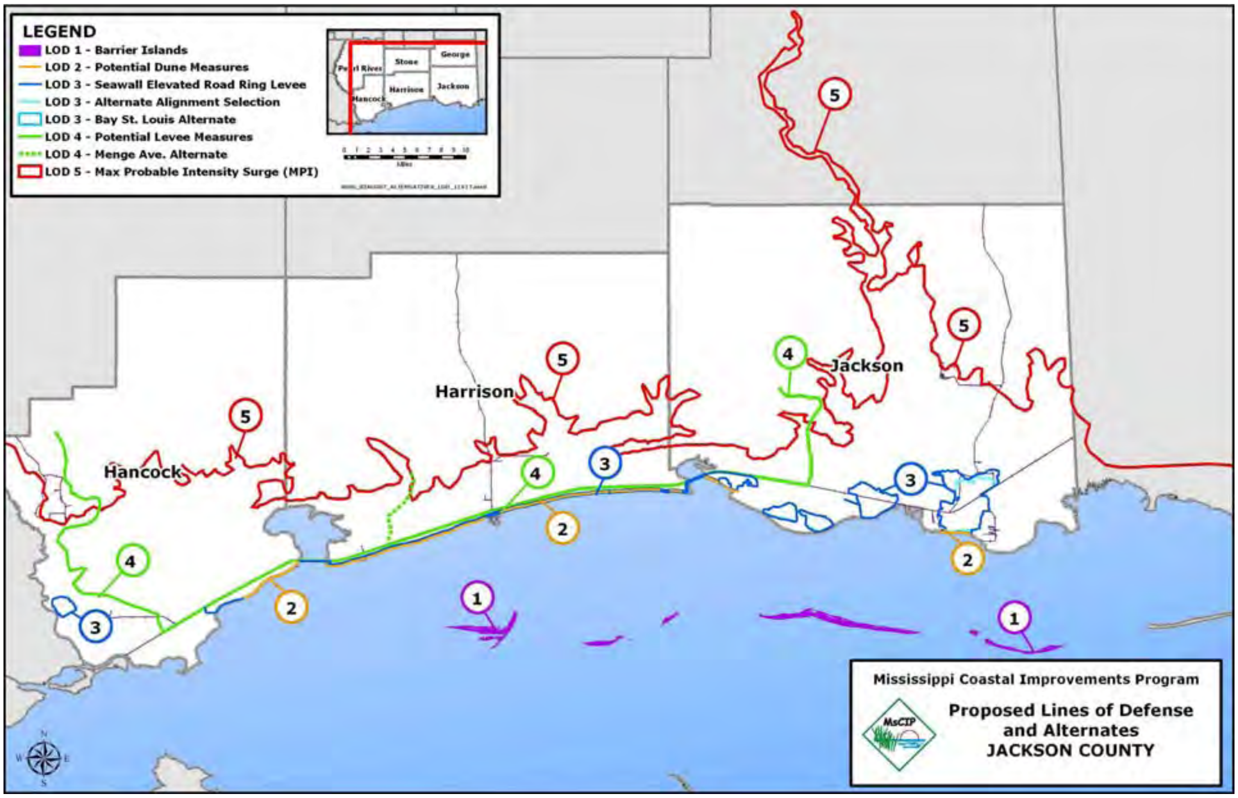


Figure 2.1-6. Line of Defense Alignments in Jackson County

The following discussions provide more detailed descriptions of the evolutions of each of the Lines of Defense from the initial concepts. Since this study generally did not provide feasibility level of design, there is still components that must be completed during “engineering and design” activities as shown in the cost estimates. This will include the completion of geotechnical investigations on the options that are carried forward. Part of the geotechnical work will be the verification of the different borrow areas including both onshore and off-shore sources.

2.1.1 First Line of Defense – Barrier islands

The coastline of mainland Mississippi is bordered on the south by the Mississippi Sound, a shallow body of water that separates the coast from four barrier islands that lie 10 to 15 miles to the south. These barrier islands are located along a littoral drift zone that moves sand westward creating three elongated islands and then to the westward most Cat Island where littoral currents are not as well defined. As shown in Figure 2.1-7, the islands are near several navigation channels. From east to west, the islands are Petit Bois, Horn, Ship, and Cat. Ship Island has been breached by prior hurricanes and now is actually two small islands, West Ship Island and East Ship Island, with a shallow sand bar between the two. Figure 2.1-8 shows the effect of recent hurricanes on Ship Island.

Since Hurricane Camille in 1969, the breach in Ship Island has existed with varying amounts of natural rebuilding between later storms as documented by the Mississippi Department of Environmental Quality, (Schmid and Yassin). The western ends of both Petit Bois and Ship Islands have migrated to the edge of navigation channels and the continuing littoral drift of the sand into the channels is causing an artificial termination of the migration. A new island has emerged on the west side of the channel from Petit Bois Island, created from the dredged sand coming from island that is disposed of on the west side of the channel.

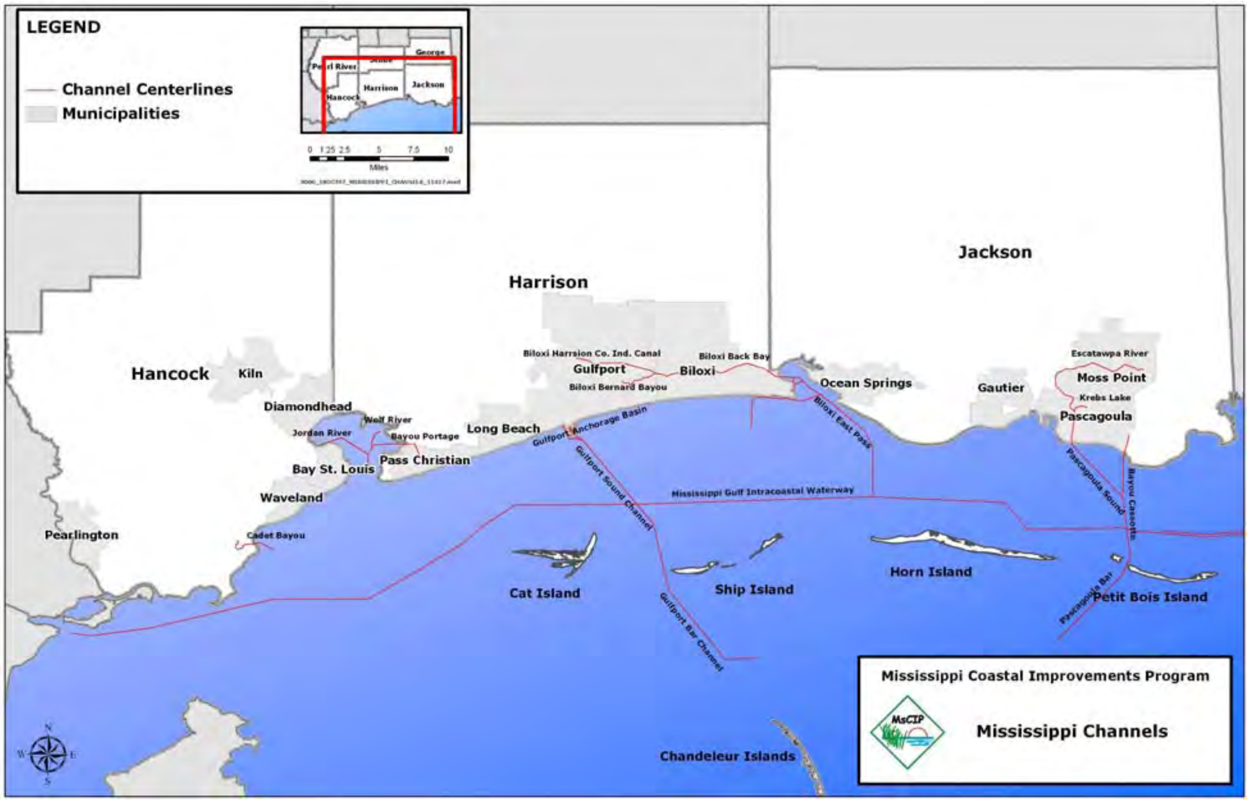
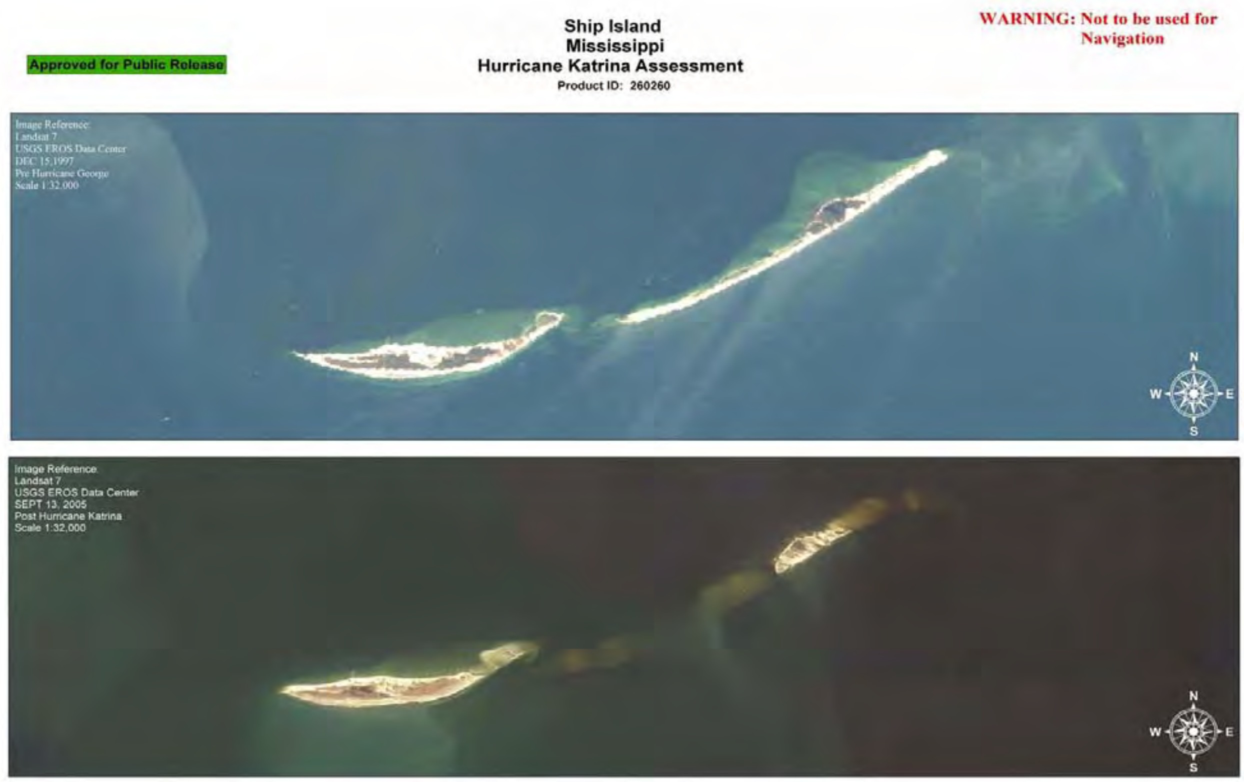


Figure 2.1-7. The Mississippi Barrier Islands shown in relationship to the numerous navigation channels near the islands

All of Petit Bois, Horn, and Ship Islands and part of Cat Island are within the boundaries of the Gulf Islands National Seashore under the jurisdiction of the National Park Service. In most cases, the boundary extends one mile from the shore of the island. The National Seashore boundaries are shown in Figure 2.1-9. Petit Bois and Horn Islands have also been designated as Wilderness Areas by the U.S. Department of the Interior and have a higher degree of protection than the other islands.

Other locations outside of park boundaries were studied for potential sites to construct breakwater type barriers that might serve the same purpose as a barrier island. Numerous constraints were identified with this concept. The depth of the water, other than being very close to the mainland shoreline, would have required a vast amount of material such as jetty stone in creating these breakwaters. And, as identified as a concern to local residents, locating these type structures close to the mainline shoreline would not be aesthetically acceptable.

Soon after Hurricane Katrina, it was reported that many in Mississippi felt that if the islands had been in the condition that existed prior to Hurricane Camille, there would have been less damage along the coast from Hurricane Katrina. This idea was also included in the Mississippi Governor's which called for restoring the islands to a pre-Camille footprint. This concept was included in the hurricane protection study as LOD-1.



(Source – US Navy)

Figure 2.1-8. The aerial photograph on top shows the islands in 1997 prior to Hurricane George in 1998. The bottom photograph shows the same view of the eroded condition of East and West Ship Island after Hurricane Katrina. Prior to a breach during Hurricane Camille, Ship Island was a single island, although the island has been breached prior to Camille.

Figure 2.1-9. Aerial view of the Gulf Islands National Seashore showing the park boundaries that extend approximately one mile offshore in most areas (National Park Service)

To determine the effects of the islands in reducing the surge damage to the mainland, a number of storms were selected to model against the chain of islands in a pre-Camille and a post-Katrina configuration. The post-Katrina condition can be considered a baseline condition for the modeling and the pre-Camille condition would be an improved condition. The pre-Camille footprint of the islands was obtained from historical records. It should be noted that some of the islands have migrated and any reconstruction would be to increase their footprint at their present location and not move them back to historical locations. Restoration of Ship Island in a pre-Camille configuration includes closing the post-Katrina, 3-mile long breach to a 2000-foot width and with dunes, along with some rebuilding of the other islands to a larger land area. Modeling efforts have concluded that over a wide range of storms, there would be some protection provided to the eastern coast of Mississippi along the Jackson County shoreline if the islands are in the pre-Camille condition. This area is the most protected from the restored islands and this protection may result in only up to a 10% reduction in storm surge. The result of the modeling is shown in Figure 2.1-10. The effect of this protection diminishes rapidly to the west from Jackson County.

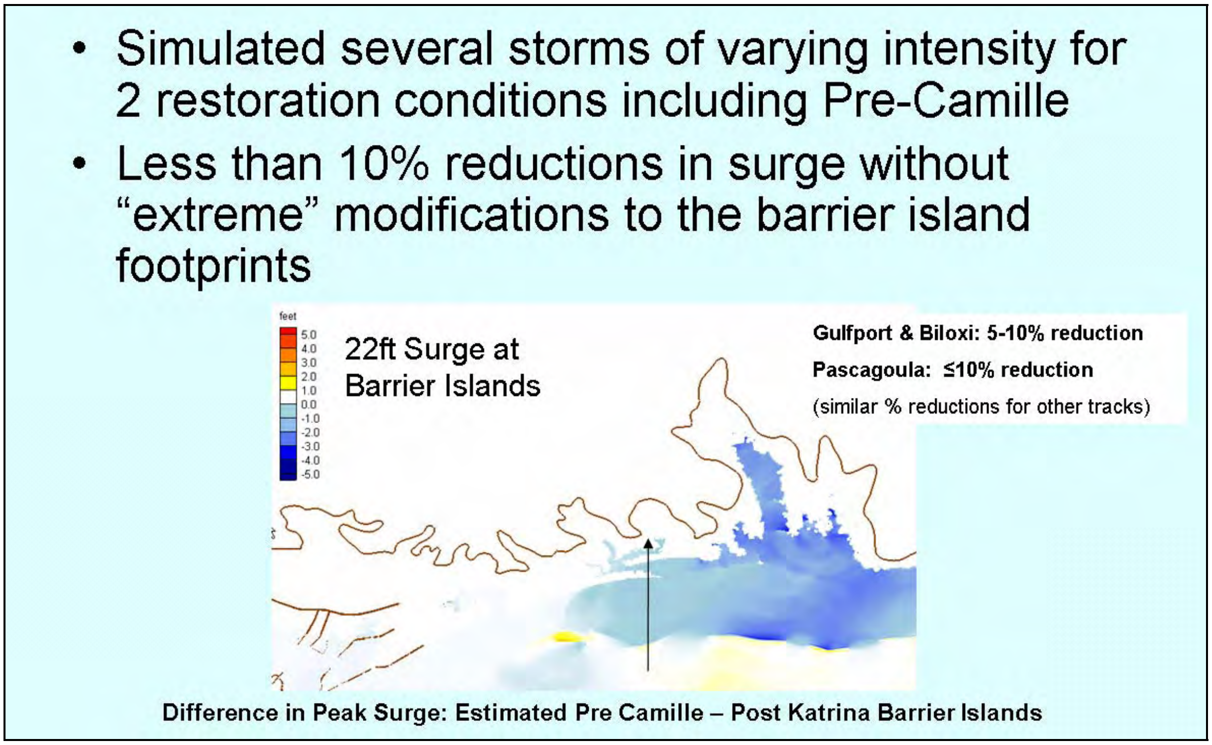


Figure 2.1-10. Sensitivity analysis of barrier island modification to differences in changes in surge heights along mainland

Another positive affect that the islands have is to provide a natural off-shore breakwater for the large sea waves that are generated from hurricanes. The presence of the islands and the relatively shallow water of the Mississippi Sound between the islands and the mainland prevent the sea waves from maintaining their considerable size as they move towards the mainland. Sea waves, often reported at heights of 40 feet and higher in large storms, would break as they approach the chain of islands. The open water between the islands and the mainland, generally ten miles or more, would have enough fetch for waves to regenerate, but at a much lower height due to the shallower water. The generally accepted relationship between water depth and wave height is that the wave can sustain itself at a height that is one half the depth of the water.

An environmental impact of the islands continuing to diminish in size is to allow salinity increases in the Mississippi Sound. Under current conditions, the islands provide a boundary condition between the sea water salinity of the open Gulf of Mexico and the brackish water found in the Sound. Loss of the islands would allow the salinity in the Sound to increase and result in a change of the ecological habitats that exist now. This would impact shellfish and other forms of marine life. This occurred at the Chandeleur Islands near the Mississippi barrier islands when almost the entire island structure was eroded away by Hurricane Katrina (see Figure 2.1-11). Like Cat Island on the Mississippi barrier islands, Chandeleur Islands are a remnant of a delta lobe from the Mississippi River where wave action created a beach that remained as an island after sea level rise and erosion removed the land mass between the island and the mainland.

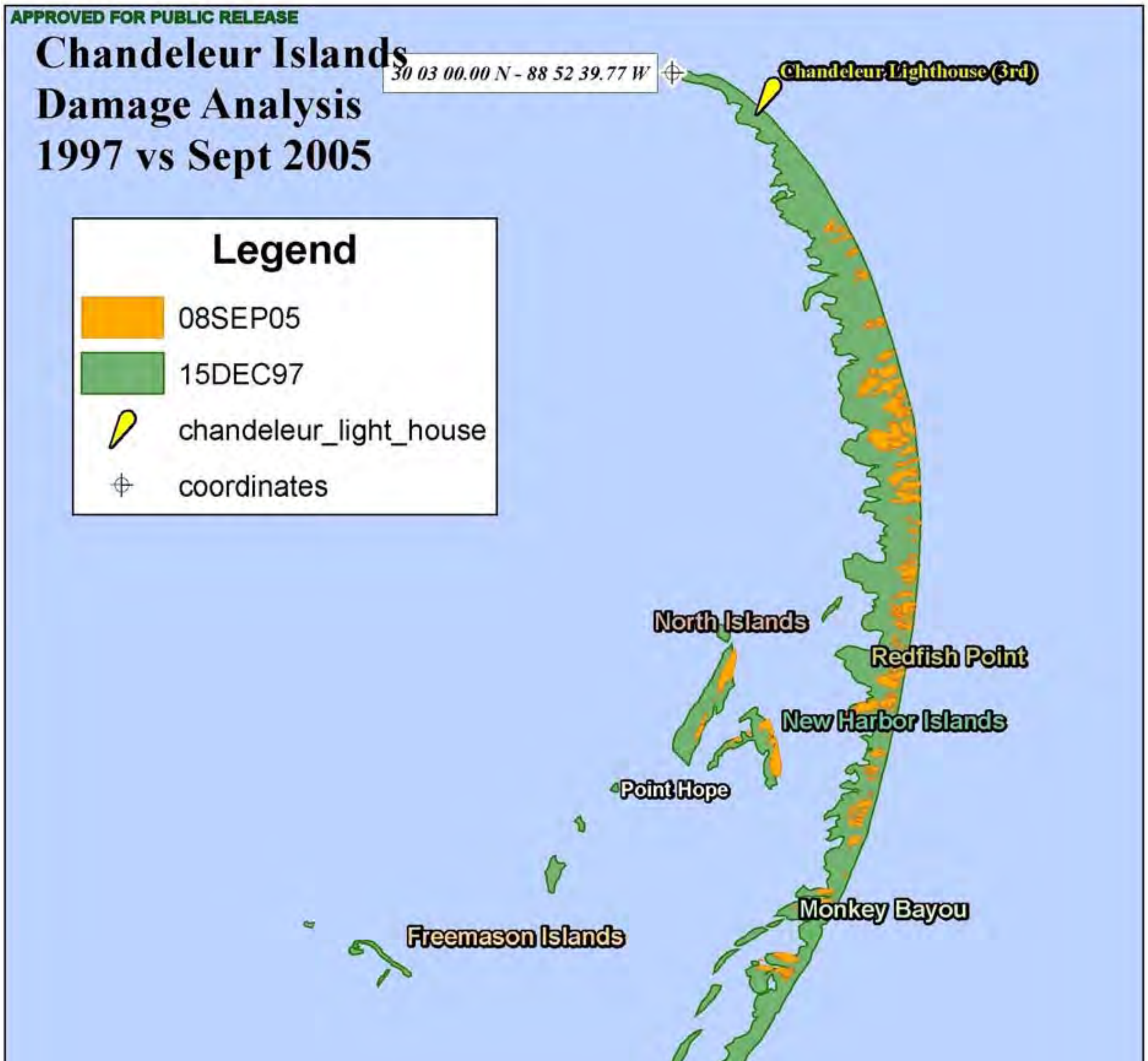


Figure 2.1-11. Loss of land mass from storm erosion at the Chandeleur Islands, 1997 to 2005. (US Navy)

With the consideration that these islands are within the National Park Service and that Petit Bois and Horn Islands are designated Wilderness Areas, any improvements to these islands may be politically difficult. One other consideration to help restore the islands is to supplement the sand in the littoral system. This could be accomplished by adding sand in specific locations based on sediment transport modeling. This would allow the littoral currents to move the sand onto the islands where the natural process of island building could take place. This would not directly affect the present-day islands and would help mitigate any effects of dredging the ship channels that pass through the chain of islands where sand may have been lost from the system.

Another plan could involve environmental restoration of the islands through reshaping dunes on the beaches with planted vegetation, planting of marshes and maritime forests, and planting sea grasses in the near-shore areas of the islands.

2.1.2 Second Line of Defense – Dunes Along Existing Beaches

Essentially all the beaches along coastal Mississippi are man-made. Harrison County has the most beach-front with 26-miles extending from Biloxi Bay to St. Louis Bay. Hancock County has several miles of beach and Jackson County only a short length. In total, the beaches extend along less than half of the Mississippi coastline. Most of the dunes that previously existed along these beaches were destroyed by Katrina and much of the beach was damaged. Reconstruction of the dunes, where beaches exist, will provide reduction of damaging wave action from smaller storms. A project to restore the beaches in Harrison County has been funded and is underway. Other projects to construct dunes to a height of 5-feet in Harrison County and to 2-feet in Hancock and Jackson County has been proposed as an interim projects and has already been designed and are awaiting funding.

Figure 2.1.2-1. View of Harrison County beach looking towards existing seawall at US Highway 90

The beaches, as situated immediately seaward of roads and developed areas, provide a location where elevated dunes could be constructed to provide some protection from smaller hurricanes. Original concepts were to look at crest elevations of 10.0 (NAVD88) and 15.0 (NAVD88) as options for the all dunes. Further discussions focused on the top elevation of the dunes needing to be below the elevation of the adjoining roadway. This was to help mitigate the migration of the sand onto the roadway as eolian (wind blown) deposits. It was decided to correlate the top of the dune to an elevation that would be 1-foot lower than the adjacent road that would be included in LOD-3. As described in the following section, LOD-3 elevated roadway elevations of 11.0 (NAVD88) were selected for Jackson and Hancock Counties and 16.0 (NAVD88) for Harrison County. These decisions for LOD-3 then dictated dune crest elevations of 10.0 (NAVD88) for Jackson and Hancock Counties and 15.0 (NAVD88) for Harrison County.

Dunes are consistent with public preference for a more natural appearing defense than a hard structure. Construction of dunes will include adding vegetation and sand fencing to help stabilize the dunes. The dunes would be a sacrificial barrier, but could also be important by providing additional protection for the toe of the existing roadway, especially in an elevated seawall or roadway configuration as LOD-3. Placement of the dunes directly against a raised seawall or roadway would also serve aesthetically to mask the appearance of a structural barrier.

While the measure described above joins LOD-2 with the adjoining roadway, consideration could be given to having a stand-alone LOD-2 dune system that is on the existing beach, but separated from the road. The quantity of sand for an option such as this would increase since the northern slope of the dune would go down to a grade elevation of about 5.0 (NAVD88) and not abut against the roadway. By doing so, the top elevation of the dune could vary and be above the roadway as necessary. This may increase the need for maintaining the sand in the designated dune alignment since it would be expected that the sand dune would tend to migrate under the prevailing wind direction. This option was not fully designed as many unanswered questions remain that may have to be simulated with models. This includes the width of the dune crest and the width of the beach berm that might be required in front of the dune. This option would also block any view of the water from the existing roadway in most areas, replacing the view with a dune scene including plantings of sea oats or other beach type vegetation.

2.1.3 Third Line of Defense – Elevated Roadways/Seawalls and Ring Levees

As previously mentioned, all of the beaches described as LOD-2 have a roadway landward of the beach. The roads vary from local or county roads to US Highway 90, a major, four-lane, highway that extends across the entire Harrison County coast. The existing roadways vary in elevation from four to five feet in Jackson and Hancock County and up to about 15 feet above sea level in Harrison County. All of these roads are evacuation routes and all have been damaged in past hurricanes. In a damaged or destroyed condition, these roads make re-entry to the area difficult after a hurricane has passed. Raising and using these roadways as barriers or having an associated seawall defines a portion of the 3rd line of defense, LOD-3. This line will be the first hard engineered structure that will not be affected by erosion from a storm such as a dune system.

Figure 2.1.3-1. Photo of existing beach-front roadway and sea wall in Hancock County, June 2006. Equipment in the background is moving sand from the area just off-shore back onto the beach after being eroded by Hurricane Katrina.

Initial strategy was to study three elevations for the structure, elevations 12.0 (NAVD88), 18.0 (NAVD88) and 24.0 (NAVD88). It was understood that due to limited heights, it would only provide protection from more frequent, smaller storms, but would be overtopped by some large storms. This coastal barrier will coincide with the beaches where they exist. Raising the beach-front road did present some engineering challenges due to the numerous intersections with other streets and roads. With several feet of elevation, the intersecting roads would require ramps that would be extremely long to have a reasonable grade. Each of these ramps would also create areas where rainfall would collect and have to be removed during a storm. It also soon became apparent that public opinion was against any structure that would block the view of the beaches and water from the adjoining properties immediately north of the roads. This was voiced in public meetings and also from agencies that were involved in the study. To maintain some level of support for this defense, it was decided to raise the roadways an average of six feet. This allowed reasonable road intersection construction and allowed the aesthetic view of the water to be maintained and would not be perceived as a high seawall along the coast.

Figure 2.1.3-2. Photo of existing beach and seawall/US Highway 90 in Harrison County, 13 June 2007.

Review of the typical roadway elevations allowed raising the roadways in Jackson and Hancock County to Elevation 11.0 (NAVD88) and Highway 90 in Harrison County to Elevation 16.0 (NAVD88). It was decided to study these elevations without other options as the main part of LOD-3 with the understanding that these structures would not provide protection from large storms. As describe above, the LOD-2 dunes could also be constructed against the elevated roadway to help protect the toe of the structural wall associated with the road.

This line of defense would be connected to Line 4, described below, at the mouth of Biloxi Bay and St. Louis Bay. It would also extend northward to higher ground or to Line 4 in Jackson County and Hancock County. The bays are an inlet for storm surge that will be controlled by surge gates that are a part of Line 4. It was also recognized that if LOD-3 was constructed without LOD-4, surge gates across the bays would have to be included as part of LOD-3.

As the first hard structural defense, Line 3 will exclude some areas that may be considered potential areas of retreat or have other non-structural solutions. This may be due to low population density, ecological sensitivity, areas that contain numerous waterway crossings or areas that could not function with a structural barrier in place. In Jackson County, Line 3 will encompass the southern portion of Ocean Springs, but due to extended marshes and streams, it will extend northeastward from near the eastern end of East Beach Road to higher ground. Areas east of this location contain numerous marshes, streams, and scattered development. Ring levees will be evaluated for housing developments in some areas. Further east in Jackson County are the cities of Gautier, Pascagoula and Moss Point. The presence of numerous streams and inlets will make a continuous barrier very difficult and these areas are also envisioned to have individual ring levees.

At the western end of LOD-3, the barrier will extend down North Beach Boulevard for several miles to near Bayou Caddy and then turn north to tie in with higher ground. By following this path, the existing roadway will provide an alignment and it will encompass much of the developed waterfront from Bay St. Louis to Waveland, MS. Further west, the town of Pearlington will be evaluated for construction of a ring levee.

As with the main portion of LOD-3, the ring levees were initially considered with the same three elevations of 12.0 (NAVD88), 18.0 (NAVD88) and 24.0 (NAVD88). Closer study revealed that in many cases, the elevation 12.0 (NAVD88) was too low based on existing ground surfaces and the elevation 24.0 (NAVD88) may not be high enough to be certified by FEMA for a 100-year storm event. The elevations to be studied for the ring levees then was changed to 20.0 (NAVD88) and 30.0 (NAVD88) with the assumption that the 100-year event would fall between these elevations and that the elevation 30.0 (NAVD88) design would be sufficiently high for even a 500-year event. A 100-year minimum event is necessary for levee certification by FEMA. Having a conceptual design with cost estimates for these two elevations would allow for a cost curve to help predict the costs for certain storm events once the modeling studies were complete and stage frequency curves developed.

Initial alignments were set for the levees that tried to enclose most of the development. These alignments were used to estimate quantities of materials required for construction. After these alignments had been analyzed, the results of the surge modeling indicated that large reductions in the quantities of material could be realized by moving the alignments to higher ground in some areas to exclude some properties that facing or near the edges of marsh or water. Placing the levee behind the structures on these properties would not provide any type of protection, but would greatly decrease the cost of construction and at the same time preserve the aesthetic valve that brought the residents there. An example of this is shown in Figure 2.1.3-3. It was also noted that some of these properties were within potential non-structural zones that were identified as potential flood-proofing areas, either by raising or buyouts.

Another consideration is the presence of multiple large drainages or tidal inlets. Enclosing these drainages within the levee system will require pumping stations to remove rainfall during storm events. Depending on the area to be drained during a storm, these pumping stations can be very large items, both in space required to construct them as well as initial cost and future maintenance. These pumping facilities require a design that can withstand large storm events as they must stay operational during and after the hurricane passes. Depending on topography, some areas such as the interior of the potential Pascagoula – Moss Point levee (see Figure 2.1.3-4) would have numerous drainages that would require water removal.

Using these stations in Mississippi has both advantages and disadvantages over their popular use in Louisiana. In Louisiana, the pumps are required to keep large areas dry because parts of the city are below sea level and the Mississippi River. This condition does not allow many areas to drain by gravity flow. In Mississippi, operation of the pumps are only required when a hurricane has caused a storm surge to push against the levee and gravity flow structures are closed. In most cases, this drainage will be through culverts with flap gates that will not require any type of mechanically assisted closure. This can also present a problem in many cases since the pumps will be in drainages that will be dry unless there is rainfall occurring. Without a supply of water, exercising the pumps as part of a maintenance program may be a problem. During this initial phase of design, most drainages had pumping stations assigned to remove rainfall. Additional studies should allow for the siting and design of storm water retention areas in many of the drainages that will negate the need for pumps, but will require the acquisition of some property.

Figure 2.1.3-3. Bell Fountaine Ring levee alignments. The alignment inside the outer line is being considered for cost savings due to being located on a higher base elevation. This alternate alignment would place any structures between the lines into a non-structural solution.

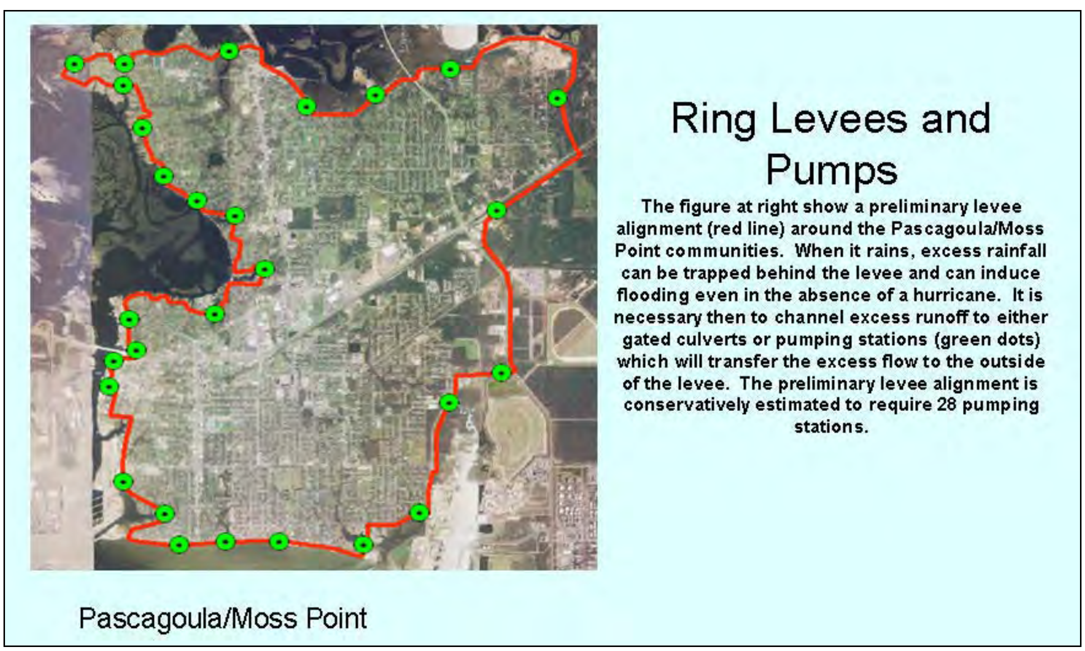


Figure 2.1.3-4. Required Pumping Stations for the Pascagoula-Moss Point Ring Levee

Modeling for storms that could impact the Mississippi Coast will define the predicted return frequency for these storms. The LOD-3 structures that might be used in Mississippi will not provide protection from large storms and this level of protection will vary based on the location and type of structure. While many options were reviewed for the type of structure to be used along the roadways, a simple elevated roadway associated with an extension of the existing seawall was chosen for reliability reasons. A structure that did not mainly rely on powered systems or with multiple moving systems was deemed more suitable for the purposes of this line of defense. As previously described, numerous conceptual designs were considered including inflatable barriers, concrete sidewalks or roadways that could be hydraulically rotated upwards to form a seawall, sliding panel gates within a seawall, and structural concrete seawalls. The ring levees were all designed as earthen structures. It should be understood that all of these LOD-3 structures would provide much less protection than would be required for a Camille or Katrina-like storm. LOD-3 storm damage reduction levels are limited and will be determined based on public and local government acceptance and the amount of risk that Mississippi is willing to accept.

As previously mentioned, this line is dependent on having the ability of closure across the two bays to prevent the storm surge from running inside the mouths of the bays. While the plan calls for surge gates to be associated with Line 4, surge gates would also have to be incorporated with Line 3 if Line 4 was not selected as an alternative. The top elevation of surge gates used solely for Line 3 would be of an elevation that would be compatible with the rest of that barrier. To develop a cost curve for the barriers, cost estimates for elevations of 20.0 (NAVD88), 30.0 (NAVD88) and 40.0 (NAVD88) have been completed and will be used in conjunction with both LOD-3 and LOD-4. More detailed discussion of the surge gates is found below under the LOD-4 section.

Interior drainage behind these barriers must be considered. Any large rainfall event would require that the water trapped behind the barrier have a means to drain or even be mechanically pumped. The amount of storage that a given watershed could provide behind a barrier during surge conditions will vary. The means to block surge but allow drainage as the surge passes may include conduits with flap valves or gated culverts up to surge gates across large bodies of water. The areas where pumping is required are numerous, but necessary to prevent residual damages associated with this blockage of normal drainage.

The pumping stations, where required, must survive any storm damage and continue to operate until the storm event has passed. Elevated pump housing and power systems would be constructed to a height commensurate with the risk associated with that line of defense. In some instances, housings may need to be hardened to ensure protection from wind related damage.

2.1.4 Forth Line of Defense – Inland Barrier

To preserve the shoreline environment as much as possible, a 4th line of defense for very large storms is envisioned that would be inland from the coast. This line of defense would be the highest line and could contain a larger storm surge up to that associated with a “Maximum Possible Intensity” (MPI) hurricane. LOD-4 was to be modeled as an infinitely high barrier with the screening storms defining a surge elevation against the barrier. The top elevation could then be defined based on selected protection from a selected screening storm. Storms that will be modeled against this line will vary up to the MPI.

As the requirements of the MsCIP project studies were developed it became apparent early on that several massive gate structures would be required to protect the large inlets from tidal surges during larger storm events. In order to protect much of the developed areas around Biloxi and St. Louis Bays, LOD-4 would have to include a structural surge barrier that would also cross the mouth of these bays. These surge barriers would prevent storm surge from moving in through the inlets of the bays. The structural barriers across the bays could be similar to designs used in Europe for storm surge protection.

Initially it was thought that some adaptation of our customary tainter or vertical lift gate assemblies might serve this purpose, but as the water levels to be resisted and the required length of the structures were developed it became apparent that much more massive construction than we had heretofore experienced would be required. This was further complicated by the need to minimize the visual impact, obstruction to small vessel traffic, and normal tidal flow.

The search for a method of construction that would be efficient and effective while optimizing freedom of tide flow and minimizing visual and physical obstruction under normal conditions led us to the Netherlands, Italy, Russia, and the River Thames in the United Kingdom, where several very massive and large scale projects of this type have been constructed or are presently in the planning stages. While many types of barriers were reviewed, the rising sector design used on the Thames River in London, England was selected

The Thames River Barrier was constructed during the 1980’s to protect portions of historic London and the surrounding area from tidal flooding. At this site there is a naturally wide variation in the “spring tides” resulting in frequent very high tides, the maximum observed to date being +3.2 meters (i.e. 3.2 meters above the normal tide influenced water level). Also at this site storm surges of as much as +3.66 meters have been experienced. In the event that a storm surge equivalent to the maximum experienced to date and a very high spring tide were to occur at the same time, the water level could conceivably reach as much as +6.86 meters at this site. Based on this possibility, the top of the gates at the Thames River barrier was set at +6.9 meters. This elevation is sufficient to fully contain the 100-year flood event which would yield a water elevation of approximately +5.5 meters. The design flood event was estimated as being the 2000-year flood.

The Barrier constructed includes a series of reinforced concrete piers and sills, supporting massive steel gates. Each main pier is 11 meters wide and extends to a point slightly above the top of the gates, with the operating machinery and machinery housings mounted atop each pier. Protective and decorative machinery housings were constructed consisting of large curved coverings made of wood and clad with stainless steel. The lowest pier foundations were sunk some 17 meters into the chalk beneath the river bottom.

The barrier includes four main navigation openings measuring 61 meters (approximately 200 feet) in width and two 31.5 meter (approximately 103-foot) openings for passage of smaller vessels. Each of these openings is fitted with a rising sector gate. To allow for free water flow for practically the full width of the river, four more 31.5 meter openings were included each having a falling radial gate, similar to the tainter type gates common to our inland waterway control structures, as a barrier against flood waters.

The rising sector gates are hollow stainless steel structures with the downriver side curved. Each gate is mounted at either end to large steel disks giving the entire gate structure the appearance of a cut-away cylinder. The gates are supported on trunnion shafts which rotate in bearings mounted in the piers. They are operated by means of reversible hydraulic rams and operating arms mounted on the top of the piers. Under normal conditions the gates lie flat in curved concrete sill recesses in the river bed. Each can be operated upward and stopped at four positions, partially closed (1/8 turn of the disk upward), fully closed (1/4 turn of the disk upward), underspill position (3/8 turn of the disk upward), and maintenance position (1/2 turn of the disk upward). To facilitate operation of the gate the interior of each gate chamber is evacuated of water resulting in a partially buoyant structure.

The facilities are operated from a Control Tower located on one bank of the river with a backup control room on the opposite bank. Two service tunnels pass through the foundation of the barrier beneath the river to connect between the two control rooms and to provide power and other utility service access to each pier. In case of extreme emergency each gate can be operated from the individual pier engine rooms. Operating power is provided by three 1.5 MW on-site power generating units, with backup connection to the local electrical grid.

Since its commissioning the Thames River Barrier has been operated 4 to 5 times per year, for a total of 276 times as of 29 April 2002. Each closing cycle takes approximately 15 minutes, though the operation time is greatly extended because of the coordination required with operation of the port facilities.

The Thames River Barrier was constructed between 1972 and 1982 and was formally opened in 1984. The total project construction cost was approximately $760 million. The annual operating and maintenance cost for the Barrier and appurtenant facilities is approximately $13 million.

In considering the rising sector gate design for application to the MsCIP barrier structures several points of advantage were identified. Under normal conditions the gates rest out of view at river bottom level. This is appealing in that it would offer a minimum of obstruction to view, to tidal ebb and flow, and to navigation through the structure. The piers, while substantial, are placed wide enough apart that they should be no more obtrusive than the existing bridge structures. The speed of operation would minimize the time the gates would be required to be in place before and after a storm event, and the fact that the gates can be rotated to a full up position for maintenance completely in the dry without installation of unwatering devices or dismantling of the structure is a great maintenance advantage. The maintenance aspect is further enhanced by the fact that the gate surface material is all stainless steel.

Readily observable disadvantages or questionable considerations include the very high construction cost, the relatively small design head required at the Thames River installation as compared to those for the MsCIP sites, the considerably weaker foundation materials existing at the Mississippi Gulf Coast sites, and the relative lengths of the barrier structures required for the MsCIP project sites compared to the Thames River site.

This type of structure would allow the least restriction to natural tidal flow and with gates flush with the natural bottom, provide the least environmental concern.

The general alignment of line 4 is envisioned along the path of a railway that crosses the coast of Mississippi. In Harrison County, this pathway is through heavily populated and commercial zones. To the east in Jackson County, a decision was made not to cross the Pascagoula River and associated marshes. To do so would have both technical and environmental concerns. Crossing this major river system would create environmental problems as well as interior flooding. Constructing barriers or levees across the marshes will change the surface water flow, restrict tidal exchange and could alter existing salinity conditions leading to major ecosystem changes. Blocking the rivers with surge gates, even for short periods could cause extensive flooding due to water backing up behind the gates during storms as rain falls inland. This could cause more flooding than the storm surge. The Pascagoula River system is also habitat to the endangered Gulf Sturgeon and any approved construction or modifications in the river would be unlikely.

For these reasons, the first major watershed divide west of the Pascagoula River was selected to turn the barrier north and extend it to a location beyond the extent of the storm surge associated with a MPI event. Similarly to the west in Hancock County, LOD-4 follows the railway to a watershed divide that is located east of the Pearl River where it follows the divide north to the MPI line. Both of these northward extensions will cross the path of Interstate 10 and may dictate some modifications to the highway depending on the selected top elevation of the line.

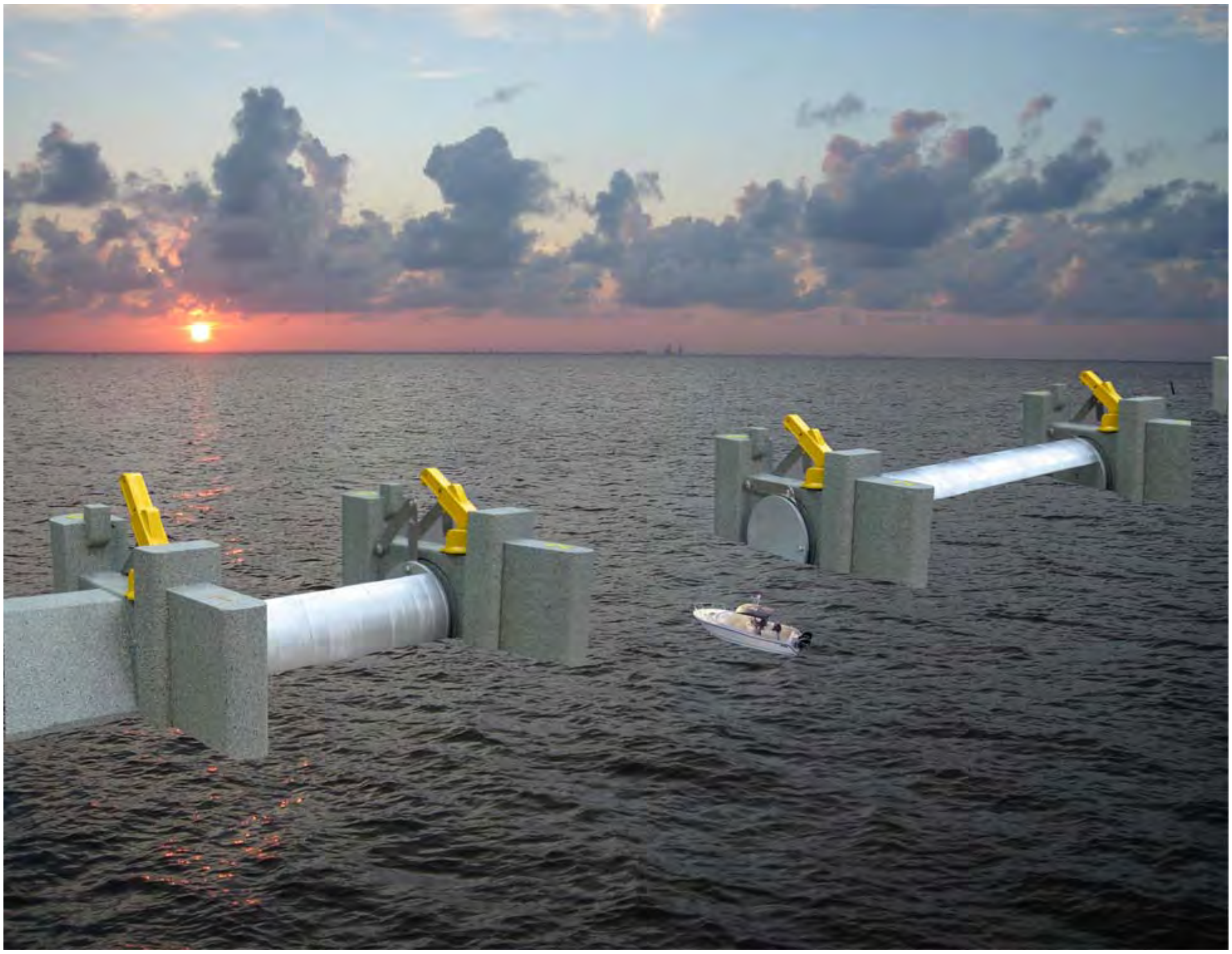


Figure 2.1.4-1. Conceptual graphic of rising sector gate used to close the mouths of the bays in Mississippi during a storm surge. This would be of similar design to what was used on the Thames River in London. The gate to the left of the boat is in the raised position, the gate in front of the boat is in the down or open position and the gate to the right of the boat is in the up or maintenance position.

LOD-4 could also be designed to have roadways, even major highways on top if desired. This line would be the highest defense, but would not protect structures seaward from the larger storms that might overtop Line 3. All facilities seaward of Line 4 would be prone to flooding in a large storm, so flood-proofing would be necessary in this zone. As described prior, this barrier would extend from high ground east of the Pearl River to high ground west of the Pascagoula River for a distance of approximately 57 miles. It would not cross either of these river systems.

Like Line 3, interior drainage behind this barrier must also be considered. The watersheds may be large and large rainfall events would require substantial structures designed to allow the water to drain or be pumped over the structure in a storm.

2.1.5 Fifth Line of Defense – Beyond the Surge Limits

Computer simulations have predicted how far inland storm surge will extend if the worse-case hurricane or maximum possible intensity (MPI) event hits the Mississippi coast. This line represents a line of safety where homes, facilities or transportation routes north of this line should not be affected by any storm surge. This would be an area where hospitals, schools, emergency response and management facilities might be located. Present predictions based on modeling sets this line near elevation 40 feet.

2.2 Hydrodynamic and Coastal Process Modeling

Part 2 documents the hydrodynamic and coastal processes modeling required to evaluate the lines of defense. The coastal processes modeling analysis employed the engineering-economic model Beach-*fx* (Gravens et al. 2007) is discussed first. Beach-*fx* relies on a shore response database to evaluate the beach and dune line of defense (line of defense two). The beach and dune analysis is primarily concerned with levels of protection below a 50-year return period and therefore the shore response database was developed with an existing surge database commonly applied for beach studies. The statistical methodology for computing the frequency relationships necessary for the evaluation of the no project condition and lines three and four is then introduced. The numerical models and methodology for providing the data to the statistical analysis is detailed including wind and atmospheric pressure modeling, offshore wave modeling, nearshore wave modeling, and storm surge modeling. The resulting frequency relationships are presented and discussed. The part concludes with documentation of various sensitivity analyses, including sensitivity to barrier island configuration (an evaluation of line of defense one), and wetlands.

2.2.1 Introduction

Part 2 documents the hydrodynamic and coastal processes modeling required to evaluate the lines of defense. The coastal processes modeling analysis employed the engineering-economic model Beach-*fx* (Gravens et al. 2007) is discussed first. Beach-*fx* relies on a shore response database to evaluate the beach and dune line of defense (line of defense two). The beach and dune analysis is primarily concerned with levels of protection below a 50-year return period and therefore the shore response database was developed with an existing surge database commonly applied for beach studies. The statistical methodology for computing the stage-frequency curves necessary for the evaluation of the no project condition and lines three and four is then introduced. The numerical models and methodology for providing the data to the statistical analysis is detailed including wind and atmospheric pressure modeling, offshore wave modeling, nearshore wave modeling, and storm surge modeling. The resulting stage-frequency curves are presented and discussed. The part concludes with documentation of various sensitivity analyses, including sensitivity to model inputs, barrier island configuration (an evaluation of line of defense one), and wetlands.

2.3 Shore Response Database

2.3.1 Purpose

The coastal processes modeling analysis employed the engineering-economic model Beach-*fx* as the primary analysis tool. The purpose of this analysis is to evaluate the physical performance of the beach and dune system for anticipated future without-project and alternative with project conditions and to estimate the economic costs and benefits of each. This section of the report documents development of the coastal processes input data and physical performance results of the Beach-*fx* analysis, the economic results of the Beach-*fx* analysis are documented elsewhere in this report. Central to the application of Beach-*fx* is development of the Shore Response Database (SRD). The SRD is a relational database that stores results of beach profile change simulations of a historically based suite of plausible storms impacting a pre-defined range of anticipated beach profile configurations, as defined by ranges of berm width, dune width, and dune height. The SRD contains the primary coastal morphology change data that is one of the basic elements of Beach-*fx,* a comprehensive analytical framework for evaluating the physical performance and economic benefits and costs of shore-protection projects, particularly, beach nourishment along sandy shores. The SRD is site- and study-specific; that is, it is developed uniquely for each shore protection project study area. Results stored in the SRD for each storm/profile combination are changes in berm width, dune width, dune height and upland width, and cross-shore profiles of erosion, maximum wave height, and total water elevation. The morphology changes (berm width, dune width, dune height and upland width changes) are used to update the simplified pre-storm beach profile to obtain the post-storm profile. Hence, through Monte Carlo simulations of the project lifecycle the morphological evolution of the study area is estimated along with project costs and infrastructure damage estimates. Simulation of multiple project lifecycles allows for the quantification of average expected project evolution, project costs, and infrastructure damages together with statistical distributions of these quantities. The damage driving parameters (cross-shore profiles of erosion, maximum wave height, and total water elevation) are used to estimate damages within reaches associated with that representative profile. The SRD is thus a pre-generated set of beach profile responses to storms and a range of profile configurations that are expected to exist under different scenarios of storm events and management actions (beach nourishment). The SRD, once generated, serves as a look-up table by the Monte Carlo simulation model. The Monte Carlo simulation has available to it the same set of storms used in populating the SRD.

2.3.2 Computational Models

The models applied to evaluate beach profile response to storms and project induced shoreline change are model SBEACH (Larson and Kraus 1989) and GENESIS (Hanson and Kraus 1989). SBEACH is a numerical model for simulating storm-induced beach change that has been applied at numerous projects. SBEACH takes as input the storm time series (wave heights, wave periods, and total water elevations) and the initial profile definition, as well as other descriptors of the beach (e.g., grain size) and model parameters, and produces as output, the estimated beach profile at the end of the storm, as well as cross-shore profiles of erosion, maximum wave height, and total water elevation including wave setup. This information is extracted from the SBEACH output by post-processing routines and stored in the SRD. The storm time series input is derived from a pre-computed surge response database developed by the Dredging Research Program (DRP) and the Wave Information Studies (WIS) database.

Estimates of the project-induced shoreline change rate are obtained through application of a one-line shoreline change model such as GENESIS (Hanson and Kraus 1989). The GENESIS model has been applied to numerous engineering projects and has demonstrated favorable capability to predict long-term shoreline change. GENESIS was designed to simulate long-term shoreline change produced by temporal and spatial differences in the longshore sand transport at coastal engineering projects. The beach profile is assumed to remain in a state of quasi-equilibrium over the long-term. The accretion or erosion of the beach is realized as a seaward or landward translation of the entire profile so that only one point of the profile, taken as the shoreline, is required to model the evolution of a sandy coast. Project-induced shoreline change rates are computed for each of the planned beach nourishment cycles which accounts for the improved performance of beach nourishment projects that comes with project maturation. That is, theory and beach nourishment experience has shown that dispersion losses at a beach nourishment project tend to decrease with the number of project renourishments. This information is stored in the database by reach and nourishment cycle. Project-induced shoreline changes capture the “spreading out” of a nourishment project on a long straight shoreline. In this phase of the analysis, potential improvements along the Mississippi Sound shoreline are assumed to be continuous along the Harrison and Hancock County shorelines. The project in Harrison County is assumed to extend from Biloxi Bay in the east to Saint Louis Bay in the west. In Hancock County the project is assumed to extend from Saint Louis Bay to Bayou Caddy.

Consequently, project-induced shoreline changes were assumed negligible because as the project is continuous across the study domain.

2.3.3 Surge Database

The DRP tropical storm database consists of surge data hydrographs recorded at 486 discrete locations corresponding to selected WIS and nearshore stations along the east and Gulf of Mexico Coasts and Puerto Rico. The database was constructed by numerically simulating historically based hurricanes that have impacted the east and Gulf coasts. The source of data for the simulations was the National Oceanic and Atmospheric Administration's National Hurricane Center's HURDAT (HURicane DATabase).

For this study, a storm suite containing 71 historical storms from the year 1886 to 2001 with at least a foot of storm surge along the Mississippi coast were identified. Storm surge hydrographs were extracted from the DRP database for these 71 historical events at stations 509 and 510. The storm surge hydrographs were subsequently combined with statistically representative astronomical tides corresponding to high, mean, and low tidal ranges and the peak storm surge was aligned with four tidal phases (high tide, mean tide falling, mean tide rising and low tide) to expand the historical storm suite by a factor of 12 resulting in a plausible storm suite of 852 unique storm events. Time series of wave heights and periods were obtained from WIS station 144 for those storms coinciding with the WIS database. For storms not included in the WIS database wave heights and periods were estimated based on methods outlined in the Shore Protection Manual.

2.3.4 Methodology

The methodology for generating the SRD involves a series of steps. First, representative beach profiles are generated based on available measured beach profiles. Then the expected range of upper beach profile configurations (dune height, dune width, and berm width) are surmised and combined with the representative submerged beach profile. SBEACH simulations of beach profile response are then performed for each unique beach profile and plausible storm combination. Finally, a data extraction routine extracts upland width, dune height, dune width and berm width changes as well as cross-shore profiles of erosion, total water elevation and maximum wave height from the SBEACH output files and writes these data to the SRD. The SRD, once generated, serves as a look-up table by the Beach-*fx* Monte Carlo simulation model. The Monte Carlo simulation has available to it the same set of storms used in populating the SRD. As a given storm from the simulated sequence takes place, the current profile (defined by representative profile, dune width, dune height and berm width) is used to look up the results that are associated with that storm in the SRD for the profile that is closest to the pre-storm profile as tracked in the simulation. The SRD results define the post-storm profile to track volume changes and to determine within-storm erosion, and wave heights and water elevations associated with the storm along the cross-shore profile. Within Beach-*fx*, storm-based morphology change includes a representation of scarping of the seaward dune face. Dune scarping takes place when the berm retreat is calculated to invade the seaward toe of the dune.

In this study, SRD databases were separately generated for Harrison and Hancock counties because the representative profiles, design conditions and storm surge hydrographs differ between the two counties. Unique SRD databases were generated for existing and future without-project conditions and for the with-project conditions. In Harrison County the existing and future without-project condition SRD is comprised of beach profile responses for a total of 10,224 beach profile – storm combinations (12 beach profiles by 852 storms). The Harrison County with-project SRD is comprised of beach profile responses for a total of 127,800 beach profile – storm combinations (150 beach profiles by 852 storms). In Hancock County the existing and future without-project condition SRD is comprised of beach profile responses for a total of 6,816 beach profile – storm combinations

(8 beach profiles by 852 storms). The Hancock County with-project SRD is comprised of beach profile responses for a total of 58,788 beach profile – storm combinations (69 beach profiles by 852 storms).

The SRD also includes an applied shoreline change rate, project-induced shoreline change rate, and post-storm berm width recovery. The user-specified applied shoreline change rate is a reach level calibration parameter and is specified in feet per year, for each reach. The applied shoreline change rate is set so that the combination of the applied shoreline change rate and storm-induced change returns on average over multiple lifecycle simulations the historical shoreline change rate for the reach. The target historical shoreline change rate is determined based on a separate analysis of the available historical beach profile and or shoreline position data. In this study, the applied shoreline change rate for Harrison County was assigned a value of -0.244 ft/year to cause Beach-fx to return the estimated historical shoreline change rate of -3.00 ft/year, based on available historical shoreline position data. In Hancock County the applied shoreline change rate was assigned a value of -2.116 ft/year to cause Beach-fx to return the estimated historical shoreline change rate of -4.85 ft/year.

Post-storm recovery of eroded berm width after passage of a major storm is recognized by the coastal engineering community although the present state of coastal engineering practice has not yet developed a predictive capability for estimating this process. Consequently, the post-storm recovery is represented in an ad hoc procedure in which the user specifies the percentage of the estimated berm width loss during the storm that is recovered over a user specified recovery interval. In this study the post-storm recovery factor was assigned a value of 80 percent and a recovery interval of 21 days. That is, 80 percent of the berm width loss caused by a storm in the simulation is restored over the 21 days following the storm event. If a second storm event occurs prior to full berm width recovery (within 21 days) berm width recovery for the first storm is suspended. The post-storm recovery factor in combination with the applied shoreline change rate serve as calibration factors in Beach-*fx*. The basis for the selected recovery factor was made based on engineering judgment and the expectation that, in the absence of storm activity, the Mississippi mainland shoreline should be either stable or modestly erosional, selection of an 80 percent recovery factor resulted in an applied erosion rate of -0.244 ft/year to achieve the long-term historical shoreline change rate of -3.00 ft/year, which satisfies the assumption of a mostly stable shoreline in the absence of storm activity.

2.3.4.1 Treatment of Future Sea Level Rise

The analysis described above was repeated three times for three different potential sea level rise scenarios corresponding to the existing rate of sea level rise, a potential future moderate rate of sea level rise and a potential future high rate of sea level rise. Incorporating potential future sea level rise in the coastal processes analysis involved adding an increment of water elevation to the total water elevation input to SBEACH. For the potential moderate future rate of sea level rise input water levels were increased 2.4 ft and for the potential high future rate of sea level rise input water levels were increased 3.8 ft. The potential future sea level rise scenarios significantly change the predicted morphology evolution, the required nourishment fill volumes, and the predicted damages. For example, in Harrison County the average long-term shoreline change rate of -3.00 ft/year for the existing rate of sea level rise increases to -5.83 ft/year for the moderate rate of future sea level rise and to -5.18 ft/year for the high rate of future sea level rise. The reason for the decrease in the average annual rate of shoreline change between the moderate rate of sea level rise and the high rate of sea level rise is due to more frequent complete inundation of the beach berm for the high rate of sea level rise which results in less berm erosion and consequently less shoreline change. In Hancock county the average long-term shoreline change rate of -4.85 ft/year for the existing rate of sea level change increases to -5.18 ft/year for the moderate rate of future sea level rise and to -5.98 ft/year for the high rate of future sea level rise.

As mentioned, the future potential sea level rise scenarios result in much more frequent inundation of the beach system. The potential moderate rate of future sea level rise increases the peak total water elevation of 48 percent of the historical storms by more than a factor of 2. Similarly, the potential high rate of future sea level rise increases the peak total water elevation of 78 percent of the historical storms by more than a factor of 2, the peak total water elevation of 20 percent of the historical storms is increased by more than a factor of 3.

2.3.5 Results

The results of the coastal processes analysis are presented in the context of the with- and without-project simulations. All Beach-*fx* simulations involved consideration of 300 potential future lifecycles of 105 year duration. The sequence and number of storm events in each lifecycle was randomly selected from the plausible storm suite of 852 historically-based storm events. The sequence and number of storms encountered in each lifecycle simulation is unique. However, the series of lifecycles used in the evaluation of the with- and without-project alternatives are identical. On average, 65 storms were encountered in each of the 105-year lifecycle simulations whereas the maximum and minimum number of storms per lifecycle is 90 and 45 storms, respectively. The standard deviation in the number of storms per lifecycle is 8. To illustrate the stochastic character of the Beach-*fx* simulations Hurricane Camille, the most intense event in the suite of historically-based plausible storms, is encountered a total of 267 times in the 300 105-year lifecycle simulations. In 96 lifecycles Camille is encountered just once, in 54 lifecycles Camille is encountered twice, in 14 lifecycles Camille is encountered three times, in four lifecycles Camille is encountered four times and in one lifecycle Camille is encountered five different times. In 33 of the 300 simulated lifecycles Hurricane Camille is never encountered.

2.3.5.1 Future Without-Project Simulations

The future without-project simulations assumed continuation of existing shore protection activities and two alternative scenarios of the future shore protection activities were examined. The first alternative examined continued maintenance of the existing berm project in Harrison and Hancock counties. The second future without project alternative involved not only maintenance of the existing berm project but also construction and maintenance of the “interim dune” feature, which involves a 2.9 yd^3/ft dune feature positioned 50 ft seaward of the Hwy. 90 seawall with a 10 ft (NAVD 88) dune elevation and a 10 ft dune crest width in Harrison County. In Hancock County the interim dune feature is comprised of approximately 1.6 yd^3/ft of sand with a 7 ft (NAVD 88) dune elevation and a 10 ft dune crest width. The berm project in Harrison County involves a 230 ft wide berm extending seaward from the Hwy. 90 seawall to the Sound. The berm elevation varies from an elevation of approximately 7.2 ft (NAVD 88) at the seawall to 3.5 ft at the slope break to the Sound. The Hancock County berm project involves a 150 ft wide berm extending from the seawall to the Sound. The berm elevation varies from approximately 5.0 (NAVD88) ft at the seawall to 3.5 ft at the slope break to the Sound.

Maintenance of the future without project alternatives in Harrison County occurs on a 12 year interval at which time the without project alternative template is restored by hydraulic placement of fill material obtained from offshore sand sources. In Hancock County maintenance of the future without project alternatives occurs on an annual basis by truck haul placement. These differences in the frequency of beach maintenance in Harrison and Hancock Counties significantly influence the volume requirements of maintaining the beaches in the two counties. The magnitude of the influence of the maintenance cycle on with and without project volume requirements is discussed further in section 2.3.4.

Table 2.3-1 summarizes the results of the Harrison County without-project Beach-*fx* simulations. The data in Table 2.3-1 indicate that existing beach maintenance practices will require approximately 130 yd^3/ft of beach over a 100 year project life assuming the existing rate of sea level rise persists into the future. If however, future rate of sea level rise increases the simulations indicate that the potential moderate rate of future sea level rise will result in about a 90 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about a 115 percent increase in project volume requirements.

Table 2.3-1.
Harrison County Without-Project Summary

Alternative Name[1]	Number of Nourishments				Nourishment Volume (yd^3/ft)			
	mean	SD	max	min	mean	SD	max	min
Berm_ESLR	6	1	8	4	142.3	22.0	214.7	85.6
Interim Dune & Berm ESLR	7	1	8	5	124.9	22.1	208.6	72.5
Berm_MSLR	8	1	8	5	278.1	36.6	385.3	179.6
Interim Dune & Berm MSLR	8	0	8	7	229.2	26.1	310.4	169.4
Berm_HSLR	8	0	8	6	324.4	39.2	437.7	211.3
Interim Dune & Berm HSLR	8	0	8	7	248.9	28.5	338.9	192.1

[1] ESLR refers to "existing" sea level rise, MSLR refers to a "moderate" potential future sea level rise rate, and HSLR refers to a "high" potential future sea level rise rate.

Table 2.3-2 summarizes the results of the Hancock County without-project Beach-*fx* simulations. The data in Table 2.3-2 indicate that existing beach maintenance practices will require approximately 304 yd^3/ft of beach over a 100 year project life assuming the existing rate of sea level rise persists into the future. If however, future rate of sea level rise increases the simulations indicate that the potential moderate rate of future sea level rise will result in about a 51 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about a 69 percent increase in project volume requirements.

Table 2.3-2.
Hancock County Without-Project Summary

Alternative Name[1]	Number of Nourishments				Nourishment Volume (yd^3/ft)			
	mean	SD	max	min	mean	SD	max	min
Berm_ESLR	100	0	100	100	297.7	28.1	379.7	250
Interim Dune & Berm ESLR	100	0	100	100	310.3	31.6	396.9	250
Berm_MSLR	100	0	100	100	443.7	47.0	581.7	302.9
Interim Dune & Berm MSLR	100	0	100	100	473.1	53.5	607.0	285.7
Berm_HSLR	100	0	100	100	497.2	53.1	654.8	351.9
Interim Dune & Berm HSLR	100	0	100	100	531.5	32.1	619.7	452.1

[1] ESLR refers to "existing" sea level rise, MSLR refers to a "moderate" potential future sea level rise rate, and HSLR refers to a "high" potential future sea level rise rate.

2.3.5.2 Future With-Project Simulations

The future with-project simulations involved evaluation of four alternative design cross-sections in both Harrison and Hancock counties. The maintenance or renourishment of the design cross-sections are the same as those used in evaluation of the future without-project alternatives: renourishment every 12 years by hydraulic placement in Harrison County and annual reconstruction of the design cross-section, as required, by truck haul placement in Hancock County. The design cross-sections in Harrison County involved a 15 ft dune height, 35 ft dune crest width and a 160 ft berm width (Alternative 1); a 15 ft dune height, 25 ft dune crest width and a 170 ft berm width (Alternative 2); a 13 ft dune height, 45 ft dune crest width and a 160 ft berm width (Alternative 3); and a 13 ft dune height, 15 ft dune crest width and a 160 ft berm width (Alternative 4). Dune volumes for the four Harrison County design alternatives are 17.2 yd^3/ft, 13.9 yd^3/ft, 14.2 yd^3/ft, and 6.7 yd^3/ft for Alternatives 1, 2, 3, and 4, respectively. The design cross-sections in Hancock County involved a 10 ft dune height, 40 ft dune crest width and 80 ft berm width (Alternative 1); a 10 ft dune height, 20 ft dune crest width and 100 ft berm width (Alternative 2); an 8 ft dune height, 50 ft dune crest width and 80 berm width (Alternative 3); and an 8 ft dune height, 30 ft dune crest width and 100 ft berm width. Dune volumes for the four Hancock County design alternatives are 10.7 yd^3/ft, 6.6 yd^3/ft, 7.3 yd^3/ft, and 4.7 yd^3/ft, for Alternatives 1, 2, 3, and 4, respectively.

Table 2.3-3 summarizes the results of the Harrison County with-project Beach-*fx* simulations. The data in Table 2.3-3 indicate that, in general, nourishment is required at the end of every nourishment cycle (the maximum number nourishments is 9) for the moderate and high potential future sea level rise rate. However, for the existing rate of sea level rise on average 2 nourishment cycles can be skipped for Alternative 1 and one nourishment cycle can be skipped for Alternatives 2 and 4. Nourishment volume requirements over the 100-year project life are approximately 197 yd^3/ft of beach assuming the existing rate of sea level rise persists into the future. If however, the future rates of sea level rise increases the simulations indicate that the potential moderate rate of future sea level rise will result in about a 65 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about an 86 percent increase in project volume requirements.

Table 2.3-4 summarizes the results of the Hancock County without-project Beach-*fx* simulations. The data in Table 2.3-4 indicate that with-project nourishment volumes for the existing rate of sea level rise are approximately 369 yd^3/ft of beach over a 100-year project life. If however, future rate of sea level rise increases the simulations indicate that the potential moderate rate of future sea level rise will result in about a 75 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about a 102 percent increase in project volume requirements.

Table 2.3-3.
Harrison County With-Project Summary

Alternative Name[1]	Number of Nourishments				Nourishment Volume (yd^3/ft)			
	mean	SD	Max	min	mean	SD	max	min
Alternative 1 ESLR	7	1	9	4	202.6	37.5	334.7	116.6
Alternative 2 ESLR	8	1	9	4	199.9	37.3	360.2	99.4
Alternative 3 ESLR	8	1	9	4	203.7	38.0	351.9	122.8
Alternative 4 ESLR	8	1	9	4	180.7	35.5	321.3	82.8
Alternative 1 MSLR	9	1	9	7	366.7	48.5	506.6	240.8
Alternative 2 MSLR	9	0	9	7	360.9	47.9	488.6	243.1
Alternative 3 MSLR	9	1	9	7	351.5	46.7	483.7	235.0
Alternative 4 MSLR	9	0	9	7	296.9	40.1	396.2	203.8
Alternative 1 HSLR	9	0	9	7	421.4	49.1	531.2	312.0
Alternative 2 HSLR	9	0	9	7	411.2	49.4	539.9	278.1
Alternative 3 HSLR	9	0	9	7	418.0	44.7	540.8	294.9
Alternative 4 HSLR	9	0	9	7	335.5	36.8	437.1	247.8

[1] ESLR refers to "existing" sea level rise, MSLR refers to a "moderate" potential future sea level rise rate, and HSLR refers to a "high" potential future sea level rise rate.

Table 2.3-4.
Hancock County With-Project Summary

Alternative Name[1]	Number of Nourishments				Nourishment Volume (yd^3/ft)			
	mean	SD	max	min	mean	SD	max	Min
Alternative 1ESLR	100	0	100	100	384.1	68.3	829.8	283.6
Alternative 2 ESLR	100	0	100	100	352.9	61.8	758.5	272.0
Alternative 3 ESLR	100	0	100	100	380.6	65.8	748.8	294.7
Alternative 4 ESLR	100	0	100	100	358.1	75.7	1,117.7	279.3
Alternative 1MSLR	100	0	100	100	690.1	121.9	1,034.5	445.8
Alternative 2 MSLR	100	0	100	100	587.5	93.0	877.4	404.7
Alternative 3 MSLR	100	0	100	100	674.1	136.3	1,059.4	410.3
Alternative 4 MSLR	100	0	100	100	587.4	100.1	887.6	371.6
Alternative 1HSLR	100	0	100	100	835.8	107.6	1,252.4	624.3
Alternative 2 HSLR	100	0	100	100	682.1	77.3	883.9	490.9
Alternative 3 HSLR	100	0	100	100	704.0	80.5	1,012.8	549.6
Alternative 4 HSLR	100	0	100	100	599.9	63.7	853.2	449.3

[1] ESLR refers to “existing” sea level rise, MSLR refers to a “moderate” potential future sea level rise rate, and HSLR refers to a “high” potential future sea level rise rate.

2.3.6 Summary

The coastal processes analysis conducted as a part of this study has provided a number of useful insights with respect to morphology change, coastal evolution, and the primary drivers for storm-induced damages along the Mississippi Sound shoreline. First, the Mississippi Sound shoreline is primarily a stable, low energy coast that is dramatically impacted by tropical storm events. In the absence of tropical storm events the shoreline is expected to be only slightly erosive with shoreline change rates on the order of -1 ft/year. In general, moderate storm events produce more coastal erosion and volumetric beach change along the Mississippi Sound shoreline than do major hurricanes. This is because the large storm surge associated with the very intense storms completely inundates the beach system and protects it from the high energy dissipation associated with wave breaking, which results in less overall shoreline change and volumetric erosion of the beach. Damages to upland infrastructure are largely driven by inundation and direct wave attack as opposed to erosion, partly because most of the infrastructure is located landward of the sea wall that runs along Hwy 90 Harrison County and Beach Boulevard in Hancock County.

As a result of the difference in maintenance cycles in Harrison and Hancock counties the project volume requirements in Hancock County exceed those in Harrison County by approximately 225 percent for without project conditions under existing sea level rise conditions, for the potential future sea level rise scenarios the increase in volume requirement is about 180 percent. For with project conditions the volume requirements in Hancock County exceed those in Harrison County by approximately 190 percent. The reason the volume requirements are so much higher in Hancock County is because the beach is restored to design conditions every year if needed, whereas in Harrison County the beach is restored to design conditions once every 12 years. If the beach in Harrison County is damaged by a major storm in the year following reconstruction of the design template the beach remains vulnerable for the remainder of the 11 year nourishment cycle. Essentially, the present analysis indicates that the nourishment cycle in Harrison County should be shortened or augmented with a provision for emergency dune reconstruction after the occurrence of a major storm event.

2.4 Statistical Methodology

A team of Corps of Engineers, FEMA, NOAA, private sector and academic researchers have been working toward the definition of a new system for estimating hurricane inundation probabilities. The findings and recommendations of this group are documented in a White Paper on Estimating Hurricane Inundation Probabilities (Resio 2007). The approach recommended by the group was a modified Joint Probability Method (JPM) referred to as the JPM with Optimal Sampling (JPM-OS). The JPM-OS methodology was applied for this study and is summarized here. For a full description, see Resio (2007).

2.4.1 JPM-OS

The JPM was developed in the 1970's (Myers, 1975; Ho and Meyers, 1975) and subsequently extended by a number of investigators (Schwerdt *et al.*, 1979; Ho *et al.*, 1987) in an attempt to circumvent problems related to limited historical records. In this approach, information characterizing a small set of storm parameters was analyzed from a relatively broad geographic area. The underlying concept of the JPM-OS methodology is to provide a good estimate of the surges in as small a number of dimensions as possible, while retaining the effects of additional dimensions by including an ε term within the estimated Cumulative Distribution Function (CDF) for surges. The ε term is considered to include, at a minimum, tides, random variations in the Holland B parameter, track variations not captured in storm set, model errors (including errors in bathymetry, errors in model physics, etc.), and errors in wind fields due to neglect of variations not included in the PBL winds. It is evident that the overall distribution of ε can only be approximated from ancillary information on errors in comparisons to high water marks and comparisons of results from runs with the "best-estimate" wind fields and PBL wind fields. Tides are factored into the analysis assuming linear superposition, with some degree of error introduced. Based on the best available approximations to all of these terms, assuming that all the "error" contributions are independent, and a loose application of the Central Limit Theorem, it is assumed that the "error" term can be represented as a Gaussian distribution with a mean of zero (assuming that the model suite is calibrated to this condition) and a standard deviation equal to some percentage of the modeled surge.

The JPM-OS treats geographic variation by using the Chouinard et al. (1997) method for determining optimal spatial size for estimating hurricane statistics. In this method, the optimal size for spatial sampling is estimated in a manner that balances the opposing effects of spatial variability and uncertainties related to sample size. It can be shown that the optimal spatial sample (kernel) size is in the range of 160 km for frequency analyses, and that the optimal spatial size for intensities reaches a plateau above about 200 km and does not drop off substantially at higher spatial kernel sizes. For developing the JPM-OS for the Mississippi and Louisiana coasts, a basic data set of 22 hurricanes, which had central pressures less than 955 mb, were analyzed. The hurricane sample set covers the interval 1941 through 2005.

A "line-crossing" frequency analysis methodology was applied since the frequency of landfalling storms is inherently better posed in this context. Sensitivity studies showed that the results for spatial samples for spatial kernels above 250 km do not vary markedly and a sample size of ±3 degrees (333 km) along this line was selected. Results from this analysis were converted into an estimate of the frequency of hurricanes (which attain a minimum central pressure of 955 mb or less) making landfall within contiguous 1-degree increments along the reference line. For each 1-degree increment along the coast, pressure differentials at the time of landfall for all storms making landfall within the ±3-degree distance along the reference line were used to define a best-fit (conditional) Gumbel distribution, i.e. the distribution of hurricane intensity given that a hurricane (with central pressure less than 955 mb) does occur. Combining the storm frequency estimates with the Gumbel coefficients for the pressure differentials, estimates of the omni-directional probability of intensity along the Gulf coast at the time of landfall can be made.

Storm size is not independent of storm intensity. Recently, Shen (2006) has shown that the potential intensity achievable by a hurricane is very sensitive to the size of a hurricane eye. Figures 2.4-1a and 2.4-1b show the relationships between the pressure scale radius (R_p) (i.e storm size) and central pressure of all storms exceeding Category 2 within the Gulf of Mexico at their time of maximum strength (52 storms –shown in Figure 2.4-1a) and the 22-storm sample of landfalling storms (Figure 2.4-1b). The following equation gives an estimate of the conditional probability of storm size as a function of central pressure:

$$P(R_p \mid \Delta p) = \frac{1}{\sigma(\Delta p)\sqrt{2\pi}} e^{-\frac{x^2}{2}}$$

where E2.4-1

$$x = \left(\frac{R_p - \bar{R}_p(\Delta p)}{\sigma(\Delta p)} \right)$$

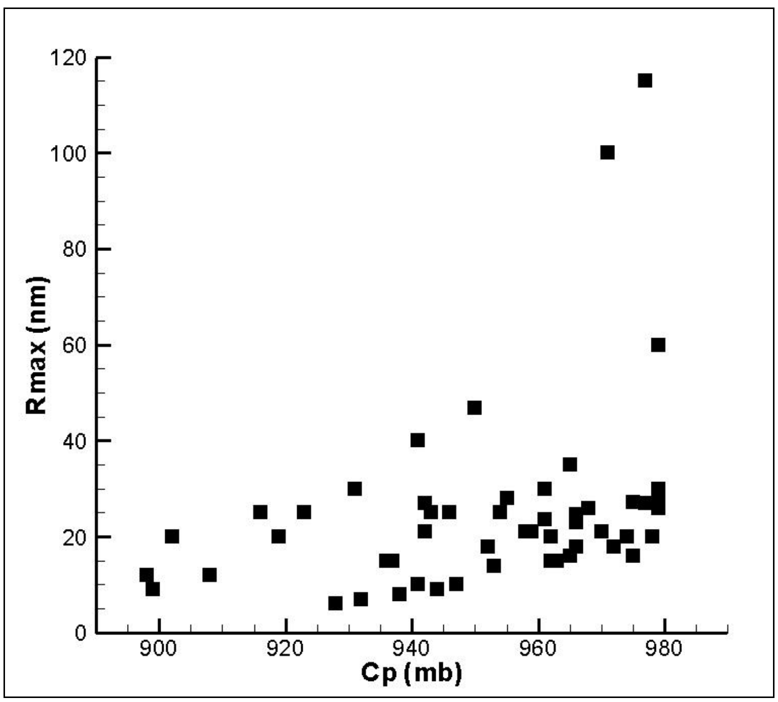


Figure 2.4-1a. Relationship between size scaling parameter (Rp) versus Central Pressure for 52 storm set in Gulf of Mexico (all storms > Cat 2). For reference, Hurricane Camille is characterized as Cp=909 mb and Rmax=11 nm. Hurricane Katrina is characterized as Cp=920 mb and Rmax=19 nm.

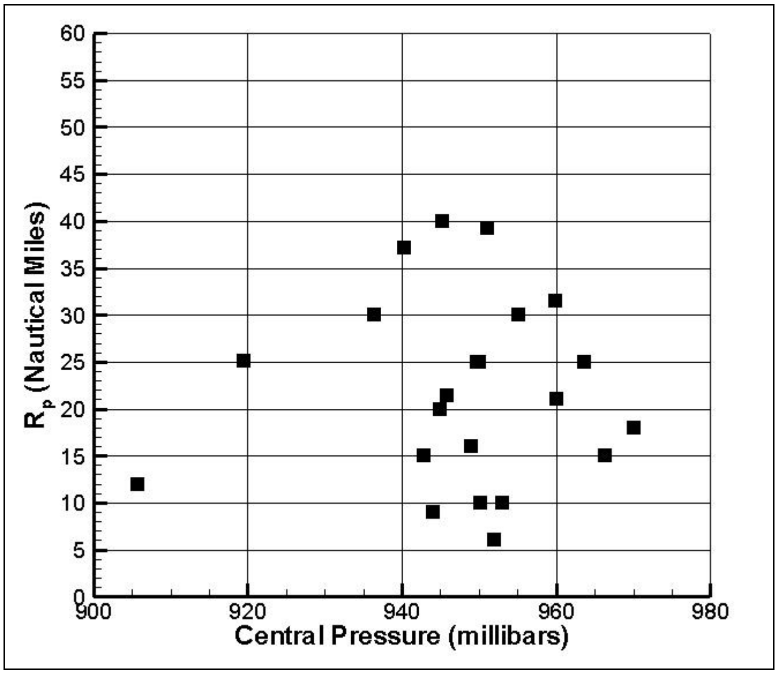


Figure 2.4-1b. Relationship between size scaling parameter (Rp) versus Central Pressure for 22 storm set in Gulf of Mexico (all storms with central pressure < 955)

Figure 2.4-2 gives the mean angle of storm heading as a function of distance along the reference line shown in Figure 2.4-3, along with the standard deviation of the heading angles around this mean value. The direction convention used here is that a heading of due north represents an angle of zero degrees. Storms heading more westerly than due north will have positive angles, while storms heading more easterly will have negative angles. These estimates were derived by the same spatial averaging procedure used in deriving the central pressures and frequencies. A circular normal distribution is used to represent the storm heading probability distribution as a function of location along the reference line.

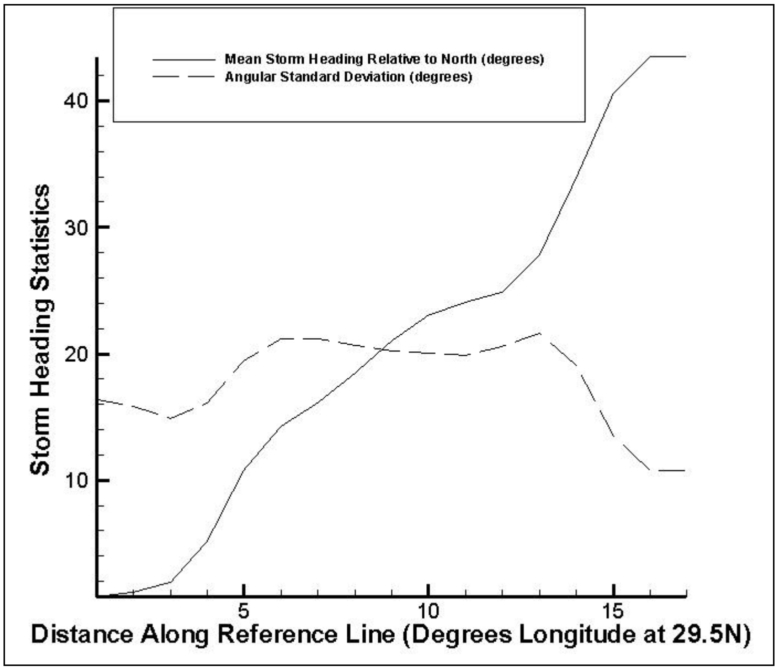


Figure 2.4-2. Plot of mean storm heading angle and standard deviation around this angle as a function of location along reference line. Distance along the x-axis can be taken as equivalent to 1-degree increments along the coast.

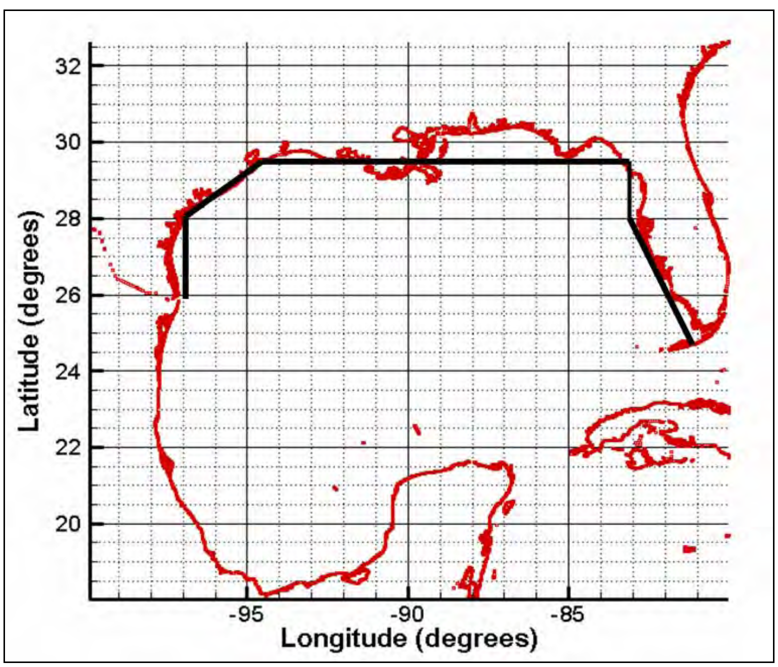


Figure 2.4-3. Location of line for analysis of hurricane landfalling characteristics

Figure 2.4-4 presents the estimated forward storm speed as a function of central pressure. This figure suggests that storm intensity and the forward speed of the storm are approximately independently distributed. Forward storm speed is plotted as a function of storm heading at landfall for the 14 storm subset that intersect with the 29.5-degree latitude portion of the reference line in Figure 2.4-3 and for the entire 22-storm sample of landfalling storms (shown in Figures 2.4-5a and 2.4-5b). These figures show that there is a tendency for higher forward speeds to be associated with lower storm heading angle (a correlation of 0.52 which is significant at the 0.05 level of significance with 21 degrees of freedom in a "Student's *t*" test). This is consistent with the expected behavior of re-curving storms that become swept up in stronger westerly circulations. The primary exception to the overall relationship is Hurricane Betsy, represented by the point in the upper right-hand corner of Figure 2.4-5b. This storm moved rapidly into the New Orleans area after crossing the lower portion of the Florida peninsula.

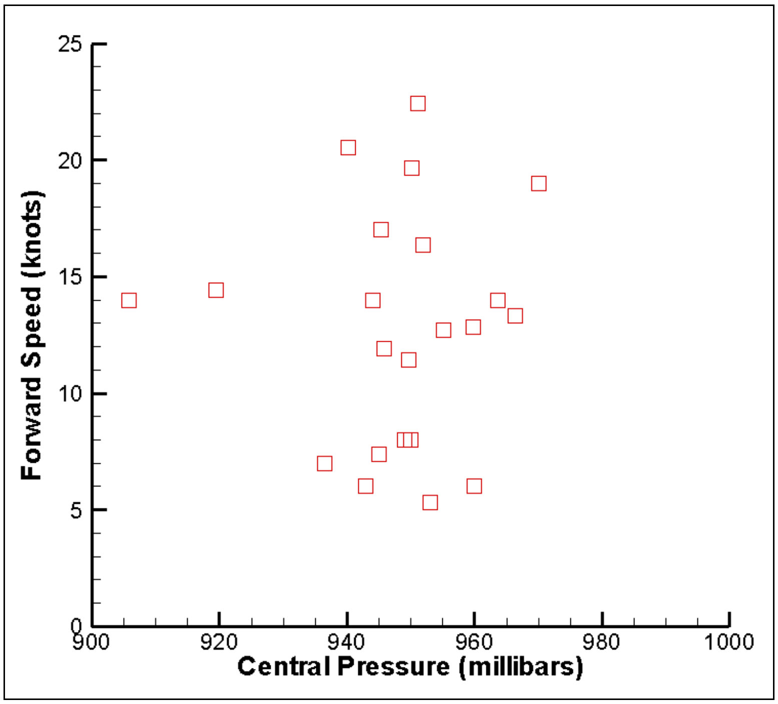


Figure 2.4-4. Plot of forward speed of storm at landfall versus central pressure at landfall

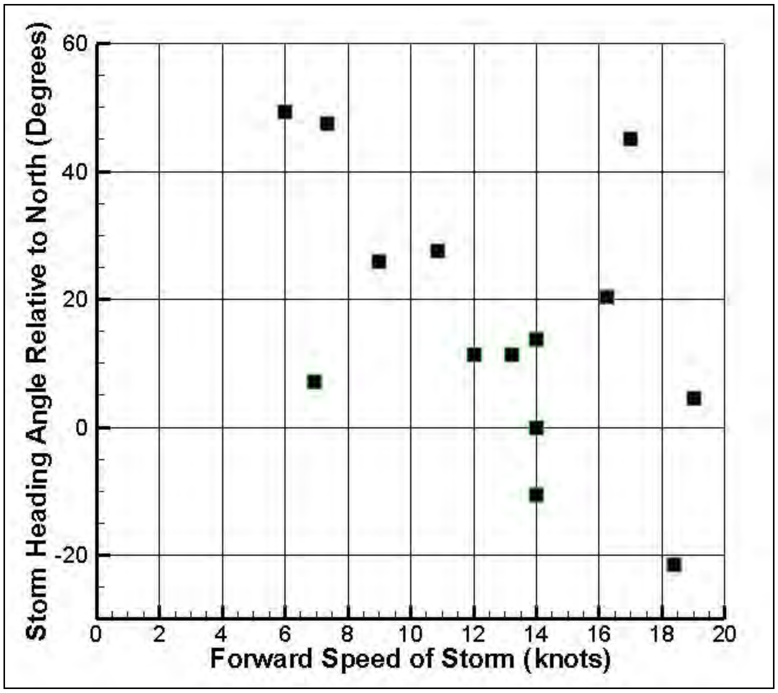


Figure 2.4-5a. Plot of storm heading and forward speed at time of landfall for only central Gulf landfalling storms

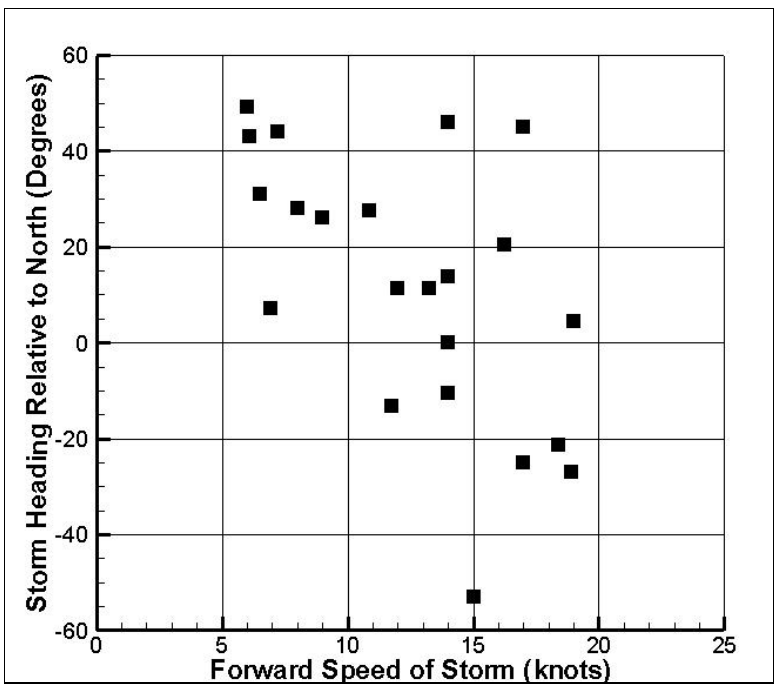


Figure 2.4-5b. Plot of storm heading and forward speed at time of landfall for the entire 22-storm sample

Consolidating this information, for any point in the five-dimensional parameter space (retaining appropriate interrelationships among parameters), the final estimates of joint probability densities can be written as

$$
\begin{aligned}
& p(c_p, R_p, v_f, \theta_l, x) = \Lambda_1 \cdot \Lambda_2 \cdot \Lambda_3 \cdot \Lambda_4 \cdot \Lambda_5 \\
& \Lambda_1 = p(c_p \mid x) = \frac{\partial F[a_0(x), a_1(x)]}{\partial c_p} = \frac{\partial}{\partial x}\left\{ \exp\left\{ -\exp-\left[\frac{c_p - a_0(x)}{a_1(x)} \right] \right\} \right\} \text{ (Gumbel Distribution)} \\
& \Lambda_2 = p(R_p \mid c_p) = \frac{1}{\sigma(\Delta P)\sqrt{2\pi}} e^{-\frac{(\bar{R}_p(\Delta P) - R_p)^2}{2\sigma^2(\Delta P)}} \\
& \Lambda_3 = p(v_f \mid \theta_l) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(\bar{v}_f(\theta_l) - v_f)^2}{2\sigma^2}} \\
& \Lambda_4 = p(\theta_l \mid x) = \frac{1}{\sigma(x)\sqrt{2\pi}} e^{-\frac{(\bar{\theta}_l(x) - \theta_l)^2}{2\sigma^2(x)}} \\
& \Lambda_5 = \Phi(x)
\end{aligned}
\qquad \text{E2.4-2}
$$

where the overbars denote average values of the dependent variable for a specified value of an independent variable in a regression equation, $a_0(x)$ and $a_1(x)$ are the Gumbel coefficients for the assumed Gumbel form of the central pressures, and $\Phi(x)$ is the frequency of storms per year per specified distance along the coast (taken as one degree in examples presented here).

2.4.1.1 Estimation of the ε term

Although there may be some degree of nonlinearity in the superposition of tides and storm surges, numerical experiments have shown that for the most part linear superposition provides a reasonable estimate of the (linearly) combined effects of tides and surges. Thus, the tidal component of the ε term, represents the percentage of time occupied by a given tidal stage and can be directly derived from available tidal information along the coast.

Careful analyses appropriate for formulating Holland B parameters for ocean response modeling have shown that this parameter falls primarily in the range of 1.1–1.6 offshore and 0.9–1.2 at the coast. For Gulf of Mexico hurricanes, a mean value of 1.27 in offshore areas is assumed with a standard deviation of 0.15, while at the coast the corresponding mean and standard deviation is 1.0 and 0.10, respectively. Via numerical experiments, the maximum storm surge generated by a hurricane has been found to vary approximately linearly with variations in the Holland B parameter, at least for changes of the Holland B parameter in the range of 10–20%.

Off-coast track variations affect surges at the coast primarily through the effects of these track variations on wave fields, rather than by their effects on direct wind-driven surges. Wave fields tend to integrate wind field inputs over tens of hours; consequently, off-coast track variations tend to shift the wave fields somewhat while maintaining the general form and magnitude of the wave height contours. Near-coast radiation stresses are approximately proportional to gradients in wave energy fluxes, which, in turn, can be related to the square of the wave height gradient. In shallow water, where contributions of radiation stresses to surges are most important, wave heights tend to be depth limited. It is only in the incremental region, where larger waves make additional contributions due to increased energy losses offshore, that larger wave conditions affect the total wave set-up at the coast. Numerical sensitivity studies suggest that once incident waves become much larger than about 10 meters, most of the additional energy loss is in depths that do not contribute very much to wave set up. For this reason plus the fact that in general the wave set-up term tends to be only about 15-30% of the total surge, we expect the effect of storm track variations on wave set-up at the coast to be fairly small (due to the fact that surge response is on a much faster scale than wave generation, where we noted that the "straight-track" approximation was not very good). It is assumed that the deviations around the mean surge will be approximately Gaussian. A standard deviation of 20% of the calculated wave-set up contributions to the total surge (determined by subtracting the direct wind-only surge from the total surge due to winds and waves combined) will be used within this distribution.

Model errors combined in calibration/verification runs of ADCIRC have shown that this combination of model and forcing in the Louisiana-Mississippi coastal area provides relatively unbiased results with a standard deviation in the range of 1.75–2.50 ft. Details on model validation are given in IPET (2007a). Relative errors associated with the use of PBL winds increase the value of the standard deviation to 2.00 to 3.50 ft. See IPET (2007b) for details. This is not too surprising, since the accuracy of HWM's (the primary measurements to which the model results are compared) are quite variable.

Combining all of these terms, under the assumption that they are each independently distributed, gives

$$p(\varepsilon) = \iint\iint \delta(\varepsilon_1 + \varepsilon_2 + \varepsilon_3 + \varepsilon_4 - \varepsilon)\, p(\varepsilon_1)\, p(\varepsilon_2)\, p(\varepsilon_3)\, p(\varepsilon_4) d\varepsilon_1 d\varepsilon_2 d\varepsilon_3 d\varepsilon_4 \qquad \text{E2.4-3}$$

where

ε_1 is the deviation between a storm at a random tide phase and a zero tide level;

ε_2 is the deviation created by variation of the Holland B parameter;

ε_3 is the deviation created by variations is tracks approaching the coast; and

ε_4 is the deviation created primarily by errors in models and grids.

Three of the terms ε_1, ε_3 and ε_4 are treated here as though they are approximately independent of the magnitude of the surge, while the remaining term, ε_2 has been found to depend essentially linearly with the magnitude of the surge. For a monochromatic tide, the tidal elevation distribution, ε_1, is known to be bimodal distributed around its zero value; however, in nature, the effect of combining several tidal components with varying phases is to force the distribution toward a unimodal distribution. The probabilities of terms ε_3 and ε_4 are assumed to be normally distributed; thus, the probability distribution of the sum of these two terms will also be a normal distribution with the variance given by the sums of the individual variances of the two terms.

Figure 2.4-6 gives a numerical example of the combination of all four terms assuming a storm surge of 15 ft, as might be associated with a particular deterministic model execution based on a set of track and PBL parameters. As can be seen in this figure, the overall magnitude of these effects can add or subtract substantially to the total water depth. In this case, the distribution appears similar to a Gaussian distribution, since it is dominated by the term with the largest variance (deviations due to the omission of the Holland B parameter); however, the other terms have been included within the integral for $p(\varepsilon)$. Table 2.4-1 shows an example of the effect of adding this term on expected surge levels for selected return periods. In this example, a Poisson frequency of 1/16 was used in combination with a Gumbel distribution, with parameters $a_0 = 9.855$ and $a_1 = 3.63$. For this example, the effect of adding the ε-term is less than ½ ft for return periods up to 175 years and only exceeds1 ft at return periods greater than 400 years. However, for risk-based calculations which often include very large return periods (1000-10000 years), this term can become as large as 2-3 ft, even for the case where the effects of all neglected factors are assumed to be distributed around a mean deviation of zero. The effect could of course be larger if the deviations were biased.

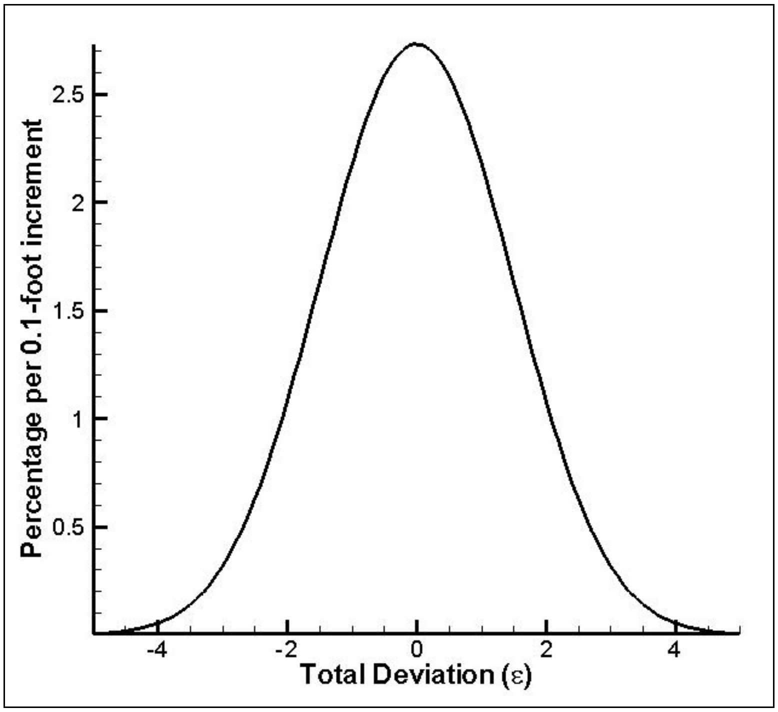


Figure 2.4-6. Percentage of deviations per 0.1-foot class as a function of deviation in feet

Table 2.4-1.
Example of Expected Surge Values as a Function of Return Period With and Without ε-Term

Return Period (years)	Without ε-Term (ft)	With ε-Term (ft)
50	11.98	12.06
100	14.82	15.21
200	17.67	18.35
300	19.33	20.18
400	20.52	21.49
500	21.43	22.50

From Table 2.4-1 and the above discussion, we see that the effect of the ε -term becomes much more pronounced at large return periods. Thus, older applications of the JPM that neglected this term were probably reasonably accurate at the 100-year return period, but were likely to have been progressively biased low at higher return periods. The important points to stress here are twofold. First, any neglect or suppression of natural variability in a procedure to estimate extremes will lead to some degree of underestimation of the estimated extremes; therefore, it is important to recognize and attempt to quantify all significant factors affecting surge heights at the coast. Second, to avoid making the number of dimensions in the JPM unmanageable, the estimated effects of the neglected factors contributing to extreme surges should be addressed statistically, such as done here via the addition of the ε-term to the JPM integral.

2.4.1.2 Sampling of Storm Parameters for the JPM-OS

In the conventional JPM, each simulation was typically treated as representative of its entire discrete probability range (i.e. all of the probability for each multi-dimensional box centered on its mean position). In these applications, the computational burden was considerably less than what is considered appropriate for surge simulations. Even in the original JPM, however, a scaling relationship between the pressure differential of a storm and computed surge levels was used to reduce the number of computer runs. This relationship, based on theoretical considerations and confirmed numerically in several studies, shows that surges are linearly proportional to the pressure differential of a storm at all areas close to the area of maximum storm impact. This information can be used effectively to interpolate between two different numerical results within the JPM integral. Such an interpolation provides added resolution along the pressure differential axis in this integral, which is very important due to the highly nonlinear characteristics of the probability of pressure differentials [$p(\Delta P)$].

In addition to the scaling relationship between surge levels and pressure differentials, the JPM-OS attempts to sample the parameter space in a fashion that can be used to estimate surges (develop the response surface) in an optimal manner. This method has been developed via hundreds of simulations on relatively straight coasts, as well as on coasts with other simple geometries, and is in the process of being extended to more complex coasts. It attempts to alleviate the need for very closely spaced parameter values in numerical simulations (essentially track spacing and number of storm sizes, forward speeds, and track angles considered); thereby potentially greatly reducing the total number of computer runs required for JPM execution. The storm suite for this study is discussed in section 2.4.2.

2.4.1.3 Specification of Variations in Pre-landfalling Hurricanes

Whereas the original JPM considered storm size, intensity, and wind field distribution to be constant in storms approaching the coast, the new JPM uses information from recent storms to estimate the rate of change of these parameters for pre-landfall conditions. In general these trends show that storms tend to fill by about 10-15 millibars, become slightly (15-30%) larger and have less peaked wind speed distributions (Holland B parameter decreasing from about 1.27 to around 1.0) over the last 90 nautical miles of coastal water before landfall. Since all of our probabilities have been developed based on landfalling characteristics, the offshore characteristics must be estimated from a generalized transform

$$p(\Delta P, R_p, v_f, \theta_l, x)_{offshore} = p(\Delta P, R_p, v_f, \theta_l, x)_{landfall} J^{-1} \qquad \text{E2.4-4}$$

where *J* is the Jacobian for the transform from nearshore to offshore conditions. However, since 1) storm heading during approach to the coast is relatively constant, 2) the forward speeds are assumed to be constant during approach to land and 3) the points of intersection (x) are identical for each offshore and landfall case, the transform can be viewed in only two dimensions, ΔP and R_p.

2.4.2 Storm Suite

Figures 2.4-7a to 2.4-7d show the synthesized primary tracks used in the study. The central tracks essentially mimic the behavior of intense landfalling historical storms in the record, while preserving the geographic constraints related to land-sea boundaries. These storms preserve the historical pattern of the tracks better than simply shifting the same storm tracks east or west along the coast, since they capture the observed variations in mean storm angles along the coast.

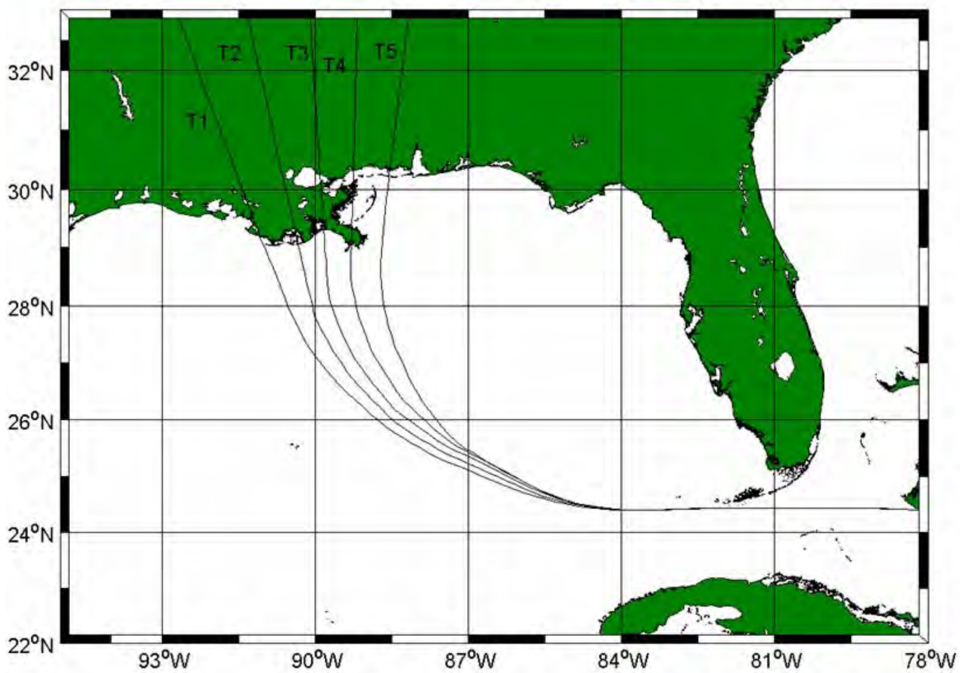


Figure 2.4-7a. Synthesized Tracks Used in the Study

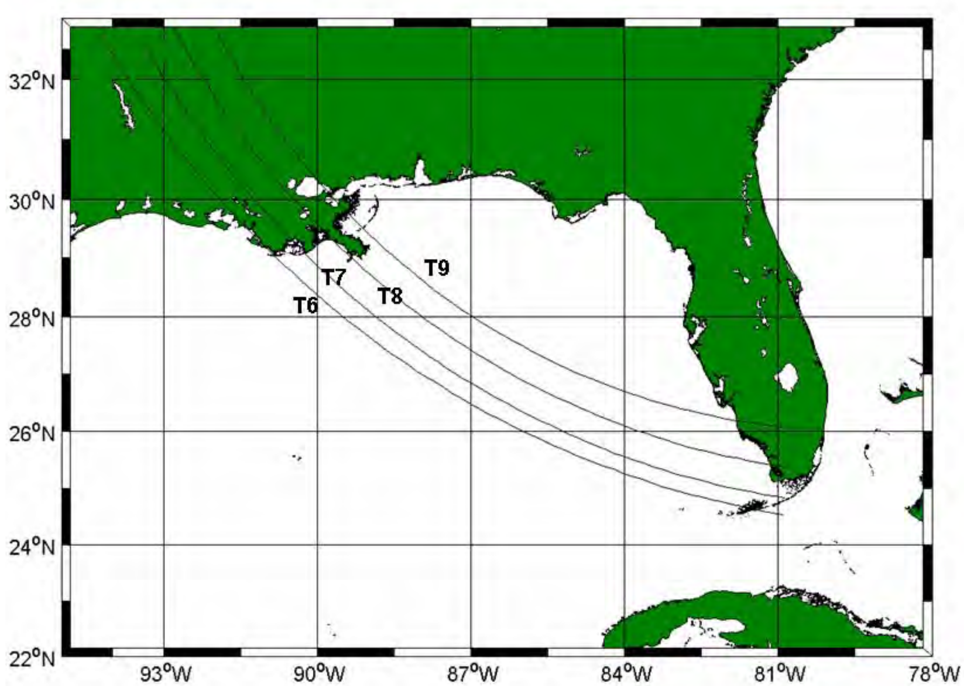


Figure 2.4-7b. Synthesized Tracks Used in the Study

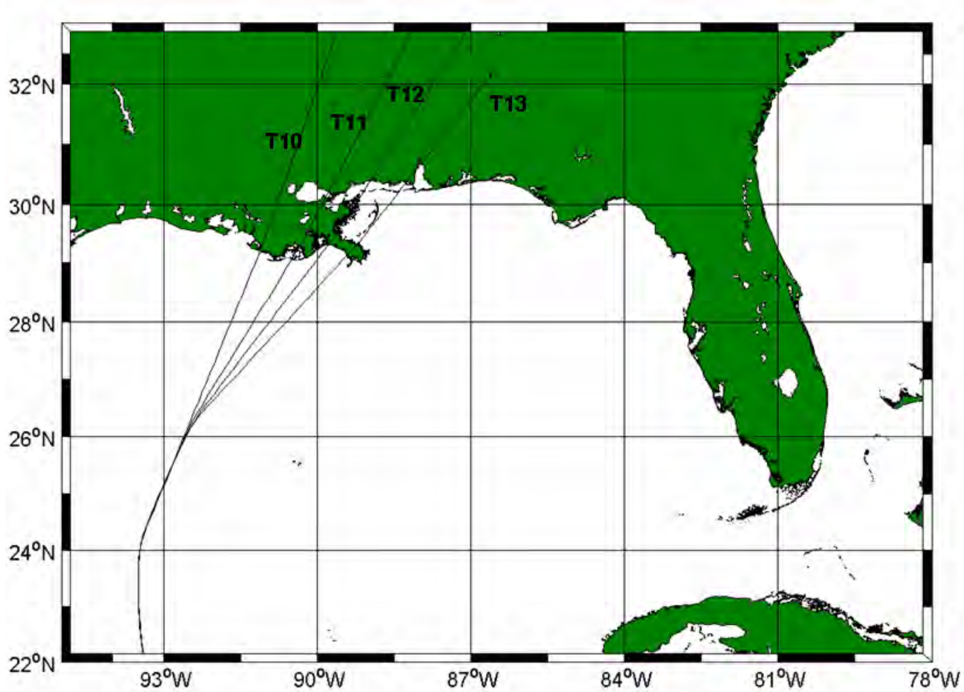


Figure 2.4-7c. Synthesized Tracks Used in the Study

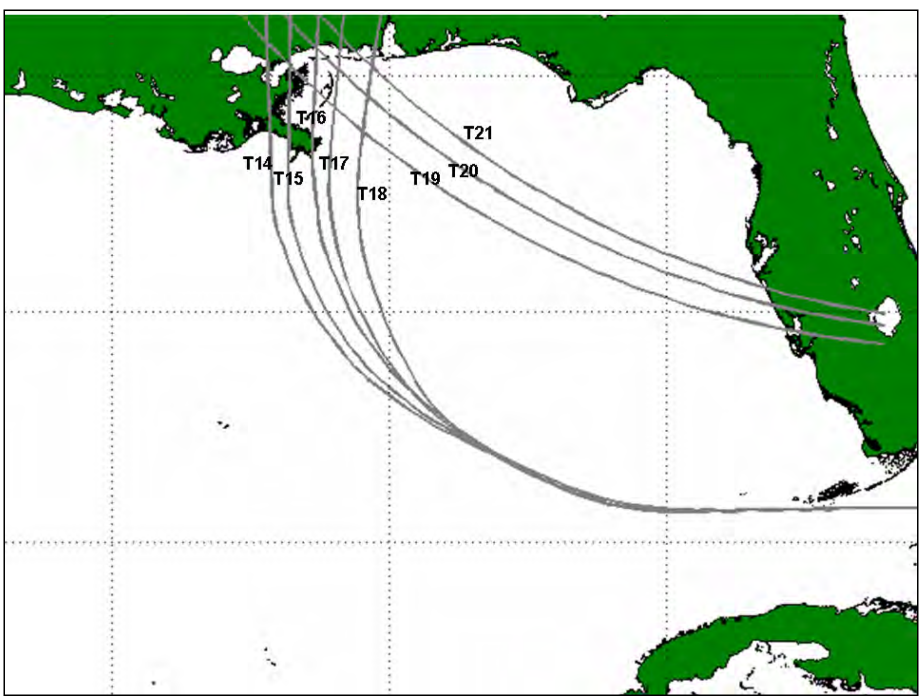


Figure 2.4-7d. Synthesized Tracks Used in the Study

Along each of the tracks modeled, the central pressure is allowed to vary during a simulated intensification interval until its intensity reaches a plateau. This plateau is maintained until the storm comes within 90 nautical miles of the coast at that time, the pressures decay according to the (linear interpolation) relationship

$$C_p(s) = \lambda_0 C_p(s_0) - (1-\lambda_0)\Delta P_{decay} \quad \text{E2.4-5}$$

where

C_p is the central pressure at s

s is the distance along the storm track, with s_0 located 90 nm from landfall

λ_0 is an interpolation multiplier (=1 at 90 nm from landfall and =0 at landfall)

ΔP_{decay} is the total change in central pressure over 90 nm approach to landfall

The pressure decay term is somewhat dependent on storm size, so the following relationship was used to represent this term

$$\Delta P'_{decay} = R_p - 6 \quad \text{(with } R_p \text{ given in nautical miles)}$$
$$\text{constrained by } \Delta P_{decay} = Max(\Delta P'_{decay}, 18); Min(\Delta P'_{decay}, 5) \quad \text{E2.4-6}$$

Once a storm is one hour past landfall the pressure decay factor according to Vickery is applied

$$C_p = P_\infty - \delta P \quad \text{E2.4-7}$$

where

$$\delta P = \delta P_0 e^{-a\Delta t}$$

where

δP is the local pressure differential

δP_0 is the pressure differential one hour after landfall

a is an empirical constant

Δt is time after landfall minus 1 hour

Rmax and the Holland B parameter are allowed to vary linearly over the same distance as Cp for all storms except the smallest storm class used in this application. For that class (Rmax = 6 nm), the storm is assumed to retain its intensity, its size, and its Holland B parameter all the way to landfall. Table 2.4-2 summarizes the central pressure / size scaling radius combinations used to define the storm suite.

Table 2.4-2.
Central Pressure/Size Scaling Radius Combinations

Central Pressure (mb)	Rmax (nm)					
900	6.0	12.5	14.9	17.7	18.4	21.8
930	8.0	17.7	25.8			
960	11.0	17.7	18.2	21.0	24.6	35.6

Defining three angles covers the important range for estimating the response surface of the surges. With the secondary variables (tidal phases, Holland B variations, wind field variations around the PBL central estimate, etc.) added to the integral, this provides a reasonable estimate of the surge CDF. The tracks approaching the Mississippi/Louisiana coast from the southeast are similar to the tracks of the 1947 Hurricane, Betsy, and Andrew. During the 1941-2005 interval, no tracks approached from the southwest; however, other storms such as the 1893 storm did approach eastern Louisiana from this direction. The 1893 track is fairly similar to one of the hypothetical tracks out of the southwest. A track from this direction represents the fact that these storms have to become caught up in the more westerly flow (winds blowing toward the east). For a storm to maintain its strength it cannot move too far west or too close to land; consequently, the track of a major storm is constrained somewhat to come from the region from which all the hypothetical (+45 degree) tracks emerge in order for these storms to strike the Mississippi coast.

The effect of storm heading angle on surges at the coast appears to be twofold. First, the overall along-coast pattern is broadened; since the storm moves along the coast at the same time that it moves toward landfall. Second, there is a relatively slow variation in the maximum surges produced by a storm as a function of the angle of the storm track with the coast. Sensitivity studies have shown that the maximum surge is relatively weakly dependent on the angle of storm intersection with the coast. In general, the hurricane approaching slightly (15-30 degrees) from west of perpendicular to a straight east-west coast produces a somewhat higher surge (5% or so) than hurricanes moving perpendicularly to the coast. On the other hand, hurricanes approaching the straight east-west coast from a more easterly direction will tend to produce lower surges than hurricanes moving perpendicular to such a coast. This appears to be a fairly broad pattern that can be represented via interpolation.

The effect of forward storm speed is addressed by considering three different forward velocities Vf=(11,6,17) knots, where 11 is around the mean and the 6-kt and 17-kt speeds span almost the entire range of Vf values at landfall for storms with Cp's less than 950. Increased forward storm speed contributes to higher wind speeds in the hurricane PBL model. Consequently, one effect of increasing forward storm velocity is to increase the surge at the coast by a factor, which is similar to increasing the wind speeds within the hurricane, i.e.

$$\eta_1 = \eta_2 \left(\frac{v_{\max} + 0.5 v_{f_1}}{v_{\max} + 0.5 v_{f_2}} \right)^2 \qquad \text{E2.4-8}$$

where

η_1 is the surge at the coast in storm 1, with forward speed = v_1

η_2 is the surge at the coast in storm 2, with forward speed = v_2

$v_{\max}$ is the maximum wind speed of a stationary storm

v_{f_i} is the forward storm velocity of the ith storm

A second effect of storm speed is to change the duration that a flood wave has to propagate inland. Thus, a slowly moving storm may produce more extensive inland flooding than a faster moving storm. By covering essentially the entire range of forward storm speeds observed in major storms within the Gulf (see Figures 2.4-5a and 2.4-5b), the range of the effects of storm speed on surges can be quantified. Table 2.4-3 identifies the various parameters for the entire 197-storm suite. Tracks denoted with a and b are secondary tracks that fall between the primary tracks plotted in Figure 2.4-7.

Table 2.4-3.
Storm Suite

Run Number	Central Pressure (mb)	Rmax (nm)	Track (see Figure 1-7)	Forward Speed (knots)
Run001	960	11	1	11
Run002	960	21	1	11
Run003	960	35.6	1	11
Run004	930	8	1	11
Run005	930	17.7	1	11
Run006	930	25.8	1	11
Run007	900	6	1	11
Run008	900	14.9	1	11
Run009	900	21.8	1	11
Run010	960	11	2	11
Run011	960	21	2	11
Run012	960	35.6	2	11
Run013	930	8	2	11
Run014	930	17.7	2	11
Run015	930	25.8	2	11
Run016	900	6	2	11
Run017	900	14.9	2	11
Run018	900	21.8	2	11
Run019	960	11	3	11
Run020	960	21	3	11
Run021	960	35.6	3	11
Run022	930	8	3	11
Run023	930	17.7	3	11
Run024	930	25.8	3	11
Run025	900	6	3	11
Run026	900	14.9	3	11
Run027	900	21.8	3	11
Run028	960	11	4	11
Run029	960	21	4	11
Run030	960	35.6	4	11
Run031	930	8	4	11
Run032	930	17.7	4	11
Run033	930	25.8	4	11
Run034	900	6	4	11
Run035	900	14.9	4	11
Run036	900	21.8	4	11
Run037	960	11	5	11
Run038	960	21	5	11
Run039	960	35.6	5	11
Run040	930	8	5	11
Run041	930	17.7	5	11
Run042	930	25.8	5	11
Run043	900	6	5	11
Run044	900	14.9	5	11

Table 2.4-3.
Storm Suite (continued)

Run Number	Central Pressure (mb)	Rmax (nm)	Track (see Figure 1-7)	Forward Speed (knots)
Run045	900	21.8	5	11
Run046	960	18.2	6	11
Run047	960	24.6	6	11
Run048	900	12.5	6	11
Run049	900	18.4	6	11
Run050	960	18.2	7	11
Run051	960	24.6	7	11
Run052	900	12.5	7	11
Run053	900	18.4	7	11
Run054	960	18.2	8	11
Run055	960	24.6	8	11
Run056	900	12.5	8	11
Run057	900	18.4	8	11
Run058	960	18.2	9	11
Run059	960	24.6	9	11
Run060	900	12.5	9	11
Run061	900	18.4	9	11
Run066	960	18.2	10	11
Run067	960	24.6	10	11
Run068	900	12.5	10	11
Run069	900	18.4	10	11
Run070	960	18.2	11	11
Run071	960	24.6	11	11
Run072	900	12.5	11	11
Run073	900	18.4	11	11
Run074	960	18.2	12	11
Run075	960	24.6	12	11
Run076	900	12.5	12	11
Run077	900	18.4	12	11
Run078	960	18.2	13	11
Run079	960	24.6	13	11
Run080	900	12.5	13	11
Run081	900	18.4	13	11
Run082	960	17.7	1	6
Run083	900	17.7	1	6
Run084	960	17.7	2	6
Run085	900	17.7	2	6
Run086	960	17.7	3	6
Run087	900	17.7	3	6
Run088	960	17.7	4	6
Run089	900	17.7	4	6
Run090	960	17.7	5	6
Run091	900	17.7	5	6

Table 2.4-3.
Storm Suite (continued)

Run Number	Central Pressure (mb)	Rmax (nm)	Track (see Figure 1-7)	Forward Speed (knots)
Run092	930	17.7	6	6
Run093	930	17.7	7	6
Run094	930	17.7	8	6
Run095	930	17.7	9	6
Run097	930	17.7	10	6
Run098	930	17.7	11	6
Run099	930	17.7	12	6
Run100	930	17.7	13	6
Run101	930	17.7	1	17
Run102	930	17.7	2	17
RuN103	930	17.7	3	17
Run104	930	17.7	4	17
Run105	930	17.7	5	17
Run106	930	17.7	6	17
Run107	930	17.7	7	17
Run108	930	17.7	8	17
Run109	930	17.7	9	17
Run111	930	17.7	10	17
Run112	930	17.7	11	17
Run113	930	17.7	12	17
Run114	930	17.7	13	17
Run115	960	17.7	1b	11
Run116	900	17.7	1b	11
Run117	960	17.7	2b	11
Run118	900	17.7	2b	11
Run119	960	17.7	3b	11
Run120	900	17.7	3b	11
Run121	960	17.7	4b	11
Run122	900	17.7	4b	11
Run123	960	17.7	6b	11
Run124	960	17.7	7b	11
Run125	960	17.7	8b	11
Run126	900	17.7	6b	11
Run127	900	17.7	7b	11
Run128	900	17.7	8b	11
Run131	960	17.7	10b	11
Run132	900	17.7	10b	11
Run133	960	17.7	11b	11
Run134	900	17.7	11b	11
Run135	960	17.7	12b	11
Run136	900	17.7	12b	11
Run137	960	17.7	1	6
Run138	900	17.7	1	6
Run139	960	17.7	2	6

Table 2.4-3.
Storm Suite (continued)

Run Number	Central Pressure (mb)	Rmax (nm)	Track (see Figure 1-7)	Forward Speed (knots)
Run140	900	17.7	2	6
Run141	960	17.7	3	6
Run142	900	17.7	3	6
Run143	960	17.7	4	6
Run144	900	17.7	4	6
Run145	930	17.7	6b	6
Run146	930	17.7	7b	6
Run147	930	17.7	8b	6
Run149	930	17.7	11b	6
Run150	930	17.7	12b	6
Run151	930	17.7	13b	6
Run152	930	17.7	1	17
Run153	930	17.7	2	17
Run154	930	17.7	3	17
Run155	930	17.7	4	17
Run156	930	17.7	6	17
Run157	930	17.7	7	17
Run158	930	17.7	8	17
Run160	930	17.7	10b	17
Run161	930	17.7	11b	17
Run162	930	17.7	12b	17
Run801	960	11	18	11
Run802	960	21	18	11
Run803	960	35.6	18	11
Run804	930	8	18	11
Run805	930	17.7	18	11
Run806	930	25.8	18	11
Run807	900	6	18	11
Run808	900	14.9	18	11
Run809	900	21.8	18	11
Run810	960	11	14	11
Run811	960	21	14	11
Run812	930	8	14	11
Run813	930	17.7	14	11
Run814	900	6	14	11
Run815	900	14.9	14	11
Run816	960	11	15	11
Run817	960	21	15	11
Run818	930	8	15	11
Run819	930	17.7	15	11
Run820	900	6	15	11
Run821	900	14.9	15	11
Run822	960	11	16	11
Run823	960	21	16	11

Table 2.4-3.
Storm Suite (continued)

Run Number	Central Pressure (mb)	Rmax (nm)	Track (see Figure 1-7)	Forward Speed (knots)
Run824	930	8	16	11
Run825	930	17.7	16	11
Run826	900	6	16	11
Run827	900	14.9	16	11
Run828	960	11	17	11
Run829	960	21	17	11
Run830	930	8	17	11
Run831	930	17.7	17	11
Run832	900	6	17	11
Run833	900	14.9	17	11
Run846	960	18.2	19	11
Run847	960	24.6	19	11
Run848	900	12.5	19	11
Run849	900	18.4	19	11
Run850	960	18.2	20	11
Run851	960	24.6	20	11
Run852	900	12.5	20	11
Run853	900	18.4	20	11
Run854	960	18.2	21	11
Run855	960	24.6	21	11
Run856	900	12.5	21	11
Run857	900	18.4	21	11

2.4.3 Measure Evaluation

To evaluate the lines of defense 3 and 4, a subset of the 197-storm suite was simulated with the structures in place. Storms 019 to 045 were selected for this purpose. The deviation of the surge and wave response from the no project condition with the lines of defense in place was computed for each storm in the measure evaluation suite. It is assumed that the rank order of the storms does not change from the no project condition such that for a given station location

$$\eta'(T)= \eta(T)+\zeta(T) \qquad \text{E2.4-9}$$

where η' is the surge/waves with the line of defense in place, T is return period, η is surge/waves for the no project condition, and ζ is the deviation from the no project condition. The deviation as a function of return period ($\zeta(T)$) is computed from the subset of 27 storms. Figure 2.4-8 is an example plot showing the deviation between line 3 and the no project condition for a save station in St. Louis Bay. With a shape preserving interpolation, the deviation at each return period is computed and applied to adjust the stage frequency relationships for the proposed lines of defense.

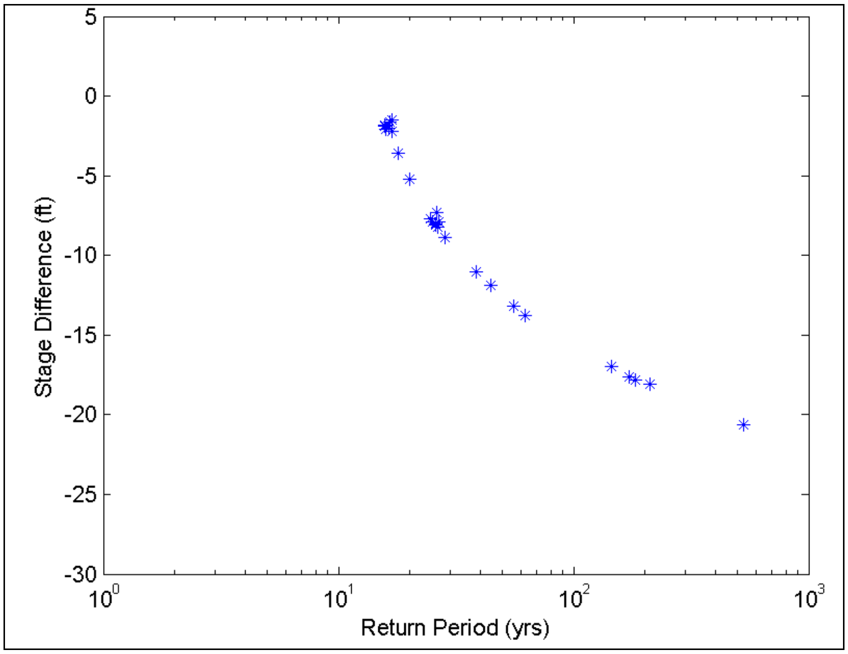


Figure 2.4-8. Plot of difference in storm surge between line of defense 3 and the no project condition as a function of return period at a save location in St. Louis Bay.

2.4.4 References

Interagency Performance Evaluation Task Force, 2007a, "Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Volume IV – The Storm," U.S. Army Corps of Engineers, Washington, D.C., https://ipet.wes.army.mil/

Interagency Performance Evaluation Task Force, 2007b, "Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Volume VIII – Engineering and Operational Risk and Reliability Analysis, Appendix 8" U.S. Army Corps of Engineers, Washington, D.C., https://ipet.wes.army.mil/

2.5 Wind and Atmospheric Pressure Modeling

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 197 storms in the JPM-OS suite. The wind fields specified with this methodology drive the storm surge simulations and the offshore and nearshore wave simulations.

2.5.1 Computational Model

The wind and pressure fields are generated with an Oceanweather Inc (OWI) highly refined meso-scale moving vortex formulation developed originally by Chow (1971) and modified by Cardone et al. (1992). The model is based on the equation of horizontal motion, vertically averaged through the depth of the planetary boundary layer. The numerical modeling grid is represented by a series of nests defined on a rectangular system; the highest resolution residing in the center of the vortex (about 2-km) decreasing in resolution by a factor of two to the outer extremities. It is assumed a tropical system changes structure relatively slowly (over a period of one or more hours). Hence, the spatial and temporal evolution of this system can be represented by a series of *snapshots* representing distinct phases in the storm's process. One added feature of this model is to conserve the integrity in storm's structure so that the horizontal velocity components can be linearly interpolated without loss in energy.

This model computes the surface wind and pressure field in tropical cyclones and is referred to as the Planetary Boundary Layer Model, or TC-96 (Thompson and Cardone, 1996). For each simulation in the suite a unique set of input conditions is defined. The data file includes the track position in space and time, the forward speed (V_f) and direction, central pressure, pressure scale radius (R_p), a rotation angle and a pressure profile peakedness parameter termed the *Holland B* factor (Holland, 1980). The wind and pressure field is generated and positioned on a fixed longitude/latitude grid system covering the Gulf of Mexico. Using continuity of the storm center, these snapshots are placed in time generating a complete account of the temporal and spatial evolution of a given hurricane. It should be noted that all storm simulations are synthetic conditions based on input criteria of the TC-96 model. Hence, no validation of the results can be pursued. This method has been used successfully for the past decade over a wide range of tropical storm scenarios for a number of studies (Cox and Cardone, 2000). Replacing the validation of the final wind and pressure fields, extensive quality control products (QA/QC) were generated to assure consistency and correctness of the forcing functions used for the wave and surge modeling efforts.

2.5.2 Methodology

The final wind and pressure fields resulting from TC-96 are targeted on a grid domain covering the entire Gulf of Mexico. The lateral boundaries are at -98º to -80º Longitude; 18º to 31º Latitude with a grid resolution of 0.05º (or 3´). The temporal variation in these fields is set to 1800-s, (30-min average wind) with lengths of storms ranging from 47- to 144-hr. All wind-fields are *marine-exposure* (no effective roughness variations for land/sea changes), generated at a 10-m elevation. The marine-exposure assumption will have implications as each of the tropical systems make landfall altering their state because of differences in the roughness lengths between open water and vegetated states. Each simulation retained consistent time of landfall at the identical date-time stamp of 080100 (month, day, hour). This effectively assured the surge and nearshore wave modeling efforts were synchronized in time. In addition each simulation has a unique name and internally a unique date-time stamp (incrementing the year for each run). The wind and pressure fields were generated for the entire 197-storm suite.

2.5.3 Results

The 197-storm suite was simulated with the TC-96 PBL model and the results were applied as forcing for the surge and wave modeling. Example results for storm 821 of the wind and pressure field generation are given in Figures 2.5-1 to 2.5-5. A series of seven individual graphical products are used in the evaluation of the wind and pressure fields. These identify any inconsistencies that would be attributed to incorrect input conditions to TC-96 since the model itself is very robust. These products include track position and maximum wind speed; maximum (wind magnitude) and minimum (pressure) field conditions over the entire simulation duration; wind and pressure field snapshot at landfall, wind field snapshot at the overall maximum speed in the simulation; and time variation of input.

Figure 2.5-1 plots the maximum wind speed found at each snapshot and the storm track position. For each individual snapshot (at 1800-s time step intervals) the maximum wind speed is determined. In general the wind maxima must be to the right of the storm track. If at any time the maximum wind speed location falls to the left of the track a potential error in the input to TC-96 is flagged. As in the case of storm 821 the locations of all maxima are to the right of the storm track. As this storm approaches the coastline and makes landfall, the wind speed decreases from nearly 60-m/s at its maximum to around 40-m/s indicative of filling of the pressure field.

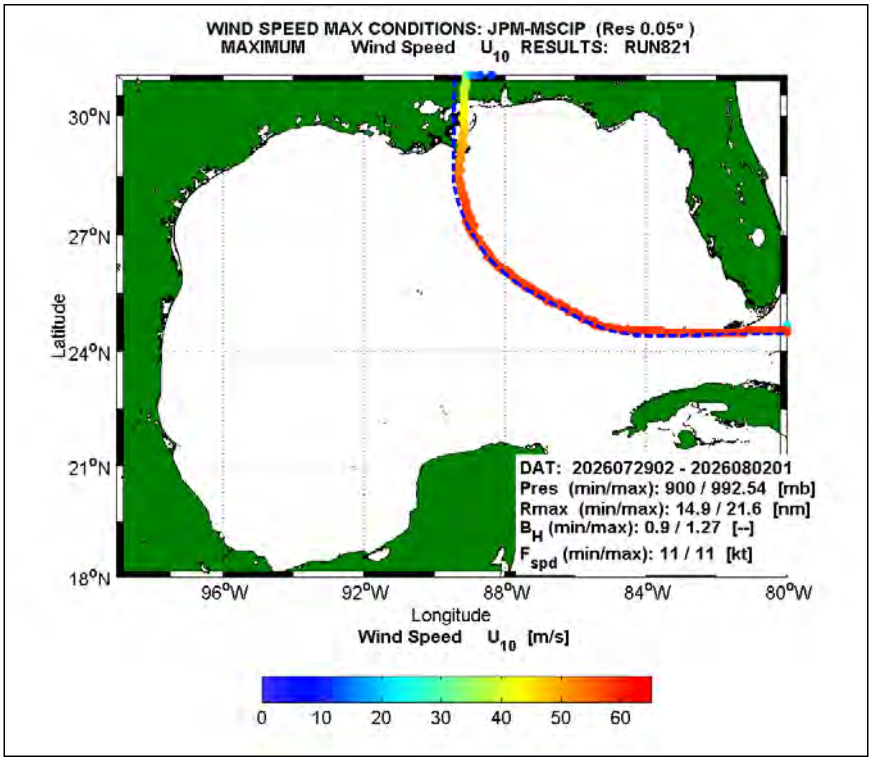


Figure 2.5-1. Location and value of maximum individual wind magnitude at every snapshot along with the storm track position for storm 821

Figure 2.5-2 represents the spatial variation of the maximum wind speed, and Figure 2.5-3 is the minimum overall pressure distribution. The wind field product (Figure 2.5-2) reflects the storm's path and displays the spatial coverage of high winds (for this case above 50-m/s), an indication of the breadth in the hurricane core. Figure 2.5-2 also shows the decay in the wind speed magnitude as it makes landfall. The minimum pressure distribution (Figure 2.5-3) clearly shows the storm track position, R_{max}, and where the filling of the pressure field occurs.

An example plot of the wind speed and wind direction vectors is shown in Figure 2.5-4. The wind direction vectors have been sub-sampled every 4 grid points, and are pointing toward which the winds are blowing. The directions also tend to reflect the base vortex in the TC-96 methodology. This is clearly evident as you move from the extremities of the storm to its center. The wind speed contouring clearly shows near continuous lines from the land to sea. This is indicative of generating an exclusive set of marine exposure wind fields. The wind speed maximum is found in the right front quadrant of the storm rotated about 45º counterclockwise from the orthogonal to the storm track.

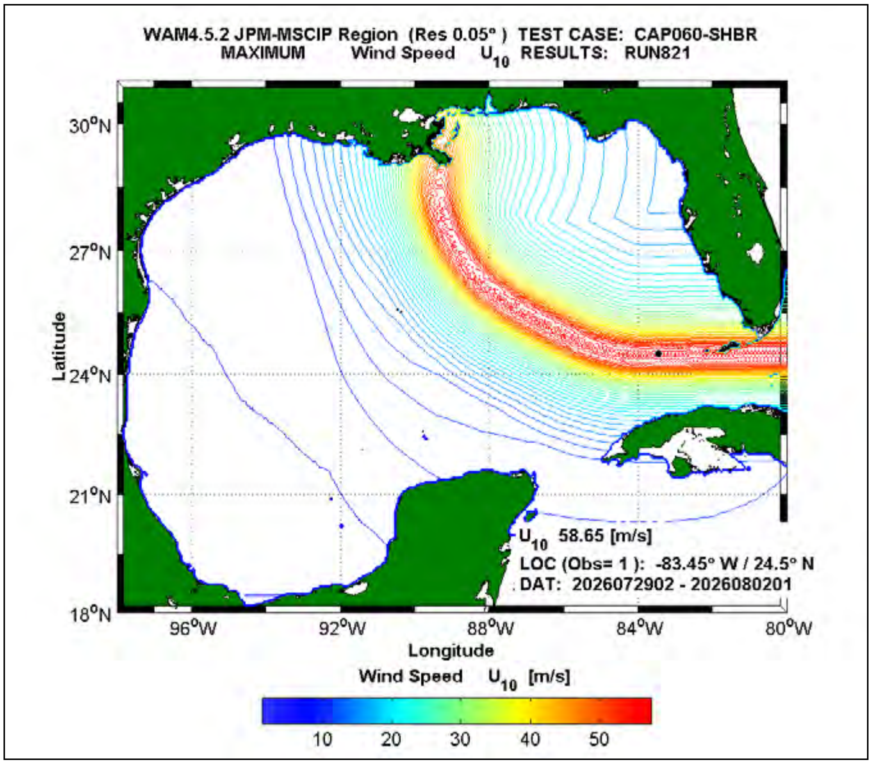


Figure 2.5-2. Maximum overall wind speed color contour for storm 821

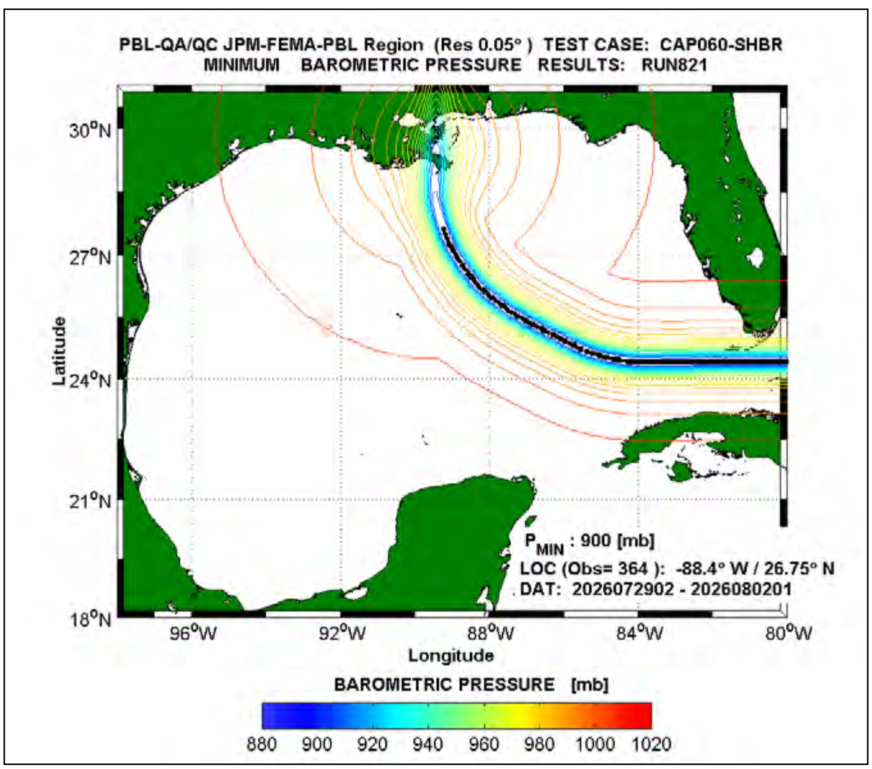


Figure 2.5-3. Minimum overall pressure field color contour for storm 821

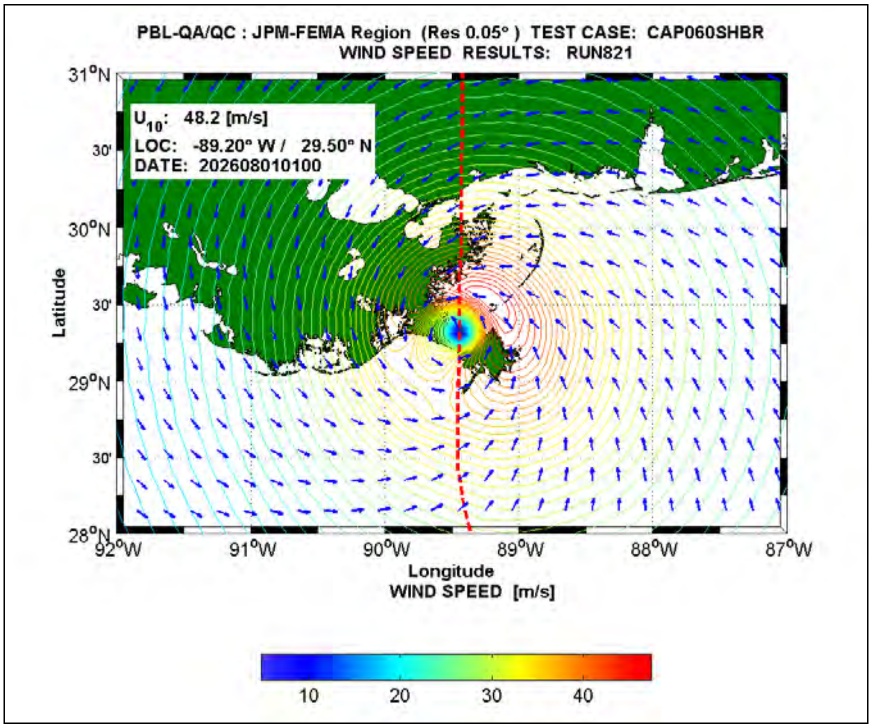


Figure 2.5-4. Snapshot of the wind speed (color contoured) and wind direction at the landfall time for storm 821

Figure 2.5-5 displays time plots and comparisons between the input files used to build the wind and pressure fields for TC-96 and results obtained from the resulting fields. It displays the minimum pressure from the input and output, the maximum wind speed (only from the output wind fields to check consistency); a comparison between R_p and a computed radius to maximum wind speed (R_{max}); the input Holland B (for consistency checks); and a comparison between the input forward speed and one computed from the field information. There are a few identifiable differences and similarities found in this product. The top panel of Figure 2.5-5 shows a large difference between the input and output minimum pressures. These differences will not influence any of the surge modeling efforts since they lie outside of the ADCIRC simulation times. These differences are attributed to the input file containing minimum pressures that are located outside of the defined grid domain. This also holds true for the comparison between the pressure scale radius (R_p) and the computed R_{max} at the start and end of the simulation. One must also note that R_p and R_{max} are not equivalent variables, but are relatively similar. In addition, the estimate of R_{max} is dependent on the modeled grid resolution of 0.05º or roughly 5.5-km. The value of R_p is a defined finite input value. In general though, despite these differences, any large deviation (more than 20-percent of the value) would be considered as questionable. The fourth panel displays the time variation in Holland B parameter analyzed directly from the input file. This variable is either constant (a value of 1.0) over time, or decreases as it does in this example just prior to landfall. This reflects the filling in the pressure field, as well as a decrease in the wind maximum. For all no constant cases, the Holland B is equal to 1.0 at landfall. The lower panel checks for the proper forward speed. The solid line is derived from the input file, while the symbols represent a computed forward speed derived from the output results. The noted oscillations result from the specified grid resolution used with accuracy levels on the order of about 5.5-km. Strong deviations from the input would suggest a phase error in the resulting wind fields and subject to either further testing or evaluation.

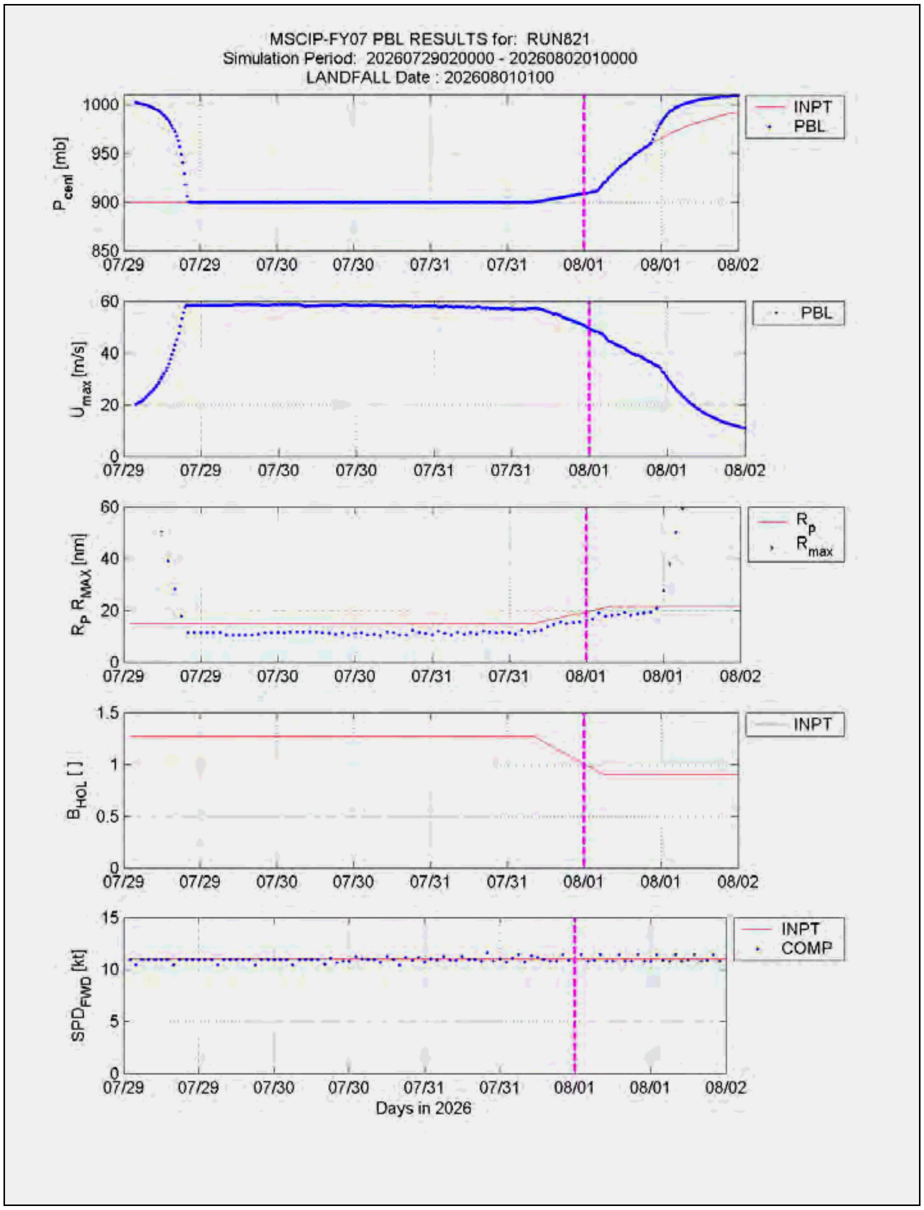


Figure 2.5-5. Time plot of input to TC-96 and output derived from the wind and pressure field files for storm 821

2.6 Offshore Wave Modeling

Offshore waves are required as a boundary condition for the nearshore wave modeling. This section describes the methodology to generate the offshore waves for the 197 storms in the JPM-OS suite. The offshore wave model is forced with the wind fields discussed in section 2.5.

2.6.1 Computational Model

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:

$$\frac{\partial N}{\partial t} + \overrightarrow{c_G} \cdot \frac{\partial N}{\partial \vec{x}} = \omega^{-1} \cdot \sum_i S_i \qquad \text{E2.6-1}$$

where N is the action density defined by $F(f,\theta,x_i,t)/\omega$, where F is the energy density spectrum defined in frequency, (f) direction (θ) over space, (x_i)and time, (t) and the radial frequency ω is equal to $2\pi f$. S_i represent the source-sink terms:

$$\sum_i S_i = S_{in} + S_{nl} + S_{ds} + S_{w-b} + S_{bk} \qquad \text{E2.6-2}$$

S_{in} is the atmospheric input, S_{nl} represents the nonlinear wave-wave interactions, S_{ds} is the high frequency breaking (white-capping), S_{w-b} is wave bottom effects (bottom friction), and S_{bk} is depth limited wave breaking. The solution is solved for the spatial and temporal variation of action in frequency and direction, over a fixed grid defined in x_i (generally a fixed longitude latitude geospatial grid).

Computationally E2.6-1 is solved in two steps. The advection term (second term in E2.6-1) 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. 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. As the water depth decreases, the full dispersion relationship is applied. Wave shoaling and refraction effects 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 (S_{in}) 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 (S_{ds}) 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 (S_{w-b}) and ultimately in very shallow water the spectrum releases much of its available energy due to breaking (S_{bk}). A more complete theoretical derivation and formulation of the source terms can be found in Komen et al. (1994).

2.6.2 Methodology

The goal of the offshore wave modeling was to provide two-dimensional wave spectra in the coastal area to be used as input boundary condition to the nearshore wave modeling (STWAVE, Smith et al. 2001). The spectral estimates contain all energy derived from the synthetic storm simulations.

Initial sensitivity tests (and past hurricane simulations) indicated that only one grid at a nominal resolution of 0.05º was required to provide quality wave estimates. The target domain is shown in Figure 2.6-1. Figure 2.6-2 identifies the save locations for the boundary conditions for the nearshore transformation modeling STWAVE (Smith et al 2001). Two sets are used in this study. The Alabama-Mississippi set (AL-MS, ST001-ST025) consist of the line parallel to the Alabama and Mississippi coastline. The second set is the diagonal line running from the northeast to southwest intersecting the AL-MS boundary at ST011 and ending at ST046. There are many distinct features that can affect the incoming wave energy, however most all, with exception to the Mississippi Canyon are landward of the defined output boundary for STWAVE. The Mississippi Canyon because of its deep water acts as a filter, attenuating wave energy.

Two time steps are applied in the wave model simulations. The propagation time step is set so that numerical stability is attained. The second time step the source term integration is set to the physical processes and relaxation times of S_{in} , S_{nl} , S_{ds} , $S_{w\text{-}b}$. In addition the time steps are required to be integer multiples of the wind input, and for the fine-scale grid also evenly divisible of the basin-scale propagation time step.

All simulations are initiated from simple fetch laws using the first wind field. Wave field information files are built for quality assurance, quality control graphical products displaying the temporal and spatial evolution of various wave related parameters for each of the 197 storms. The offshore WAM wave simulations supply the nearshore wave modeling effort supported by STWAVE (Smith, et al, 2001). The WAM directional wave spectra are output every 15-min at 28 discrete frequency bands (exponential distribution where $f_{n+1} = 1.1 \cdot f_n$ and $f_0 = 0.031384$), and 24 direction bands centered every 15-deg starting at $\theta_0 = 7.5$). The location of these special output locations are found in Figure 2.6-2.

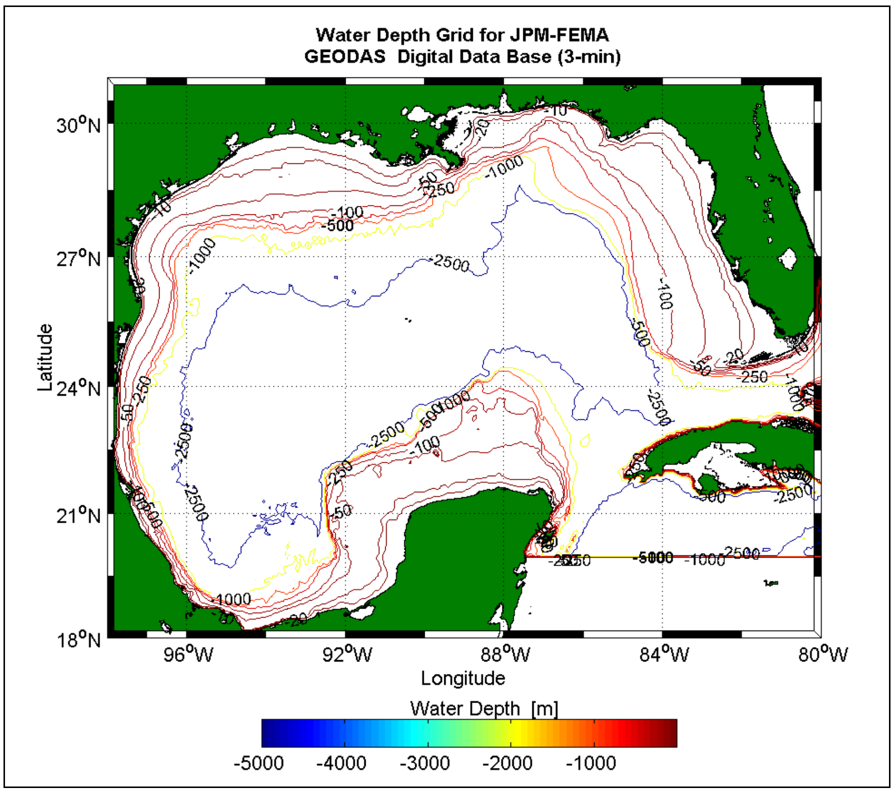


Figure 2.6-1. Water depth contours for offshore wave model simulations. Depths are in meters.

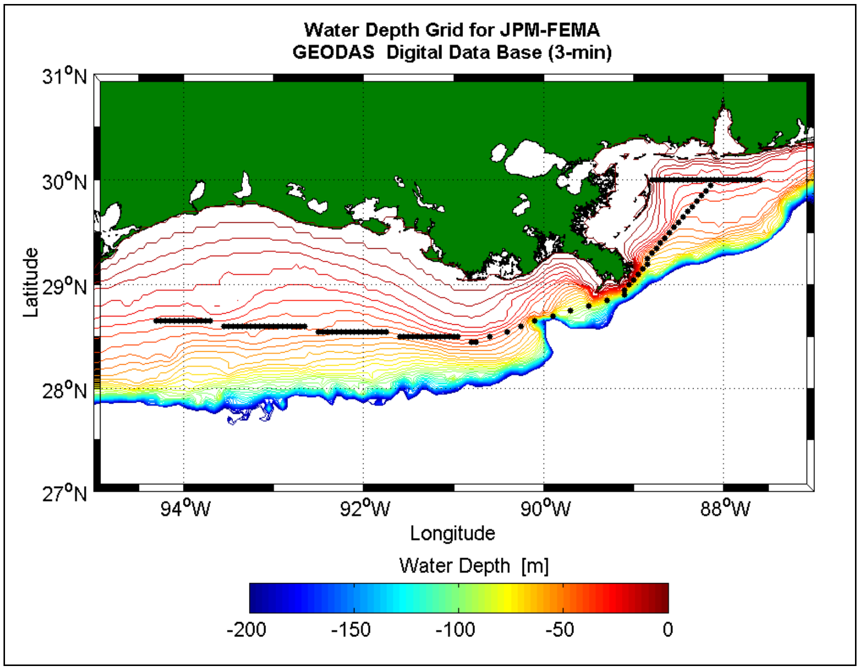


Figure 2.6-2. Refined version of the water depth grid used in offshore wave model simulations. Boundary points closed symbols, and depths are in meters.

2.6.3 Results

Generation of wave estimates based on synthetic storm simulations must be substantiated with verification/validation of the modeling results based on not only the technology used, but also the methods applied. This effort has been documented in previous studies (Interagency Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Project (IPET), https://ipet.wes.army.mil/) and more recently a Joint Coastal Surge Modeling Effort for the New Orleans District. These two reports focus on the verification of the WAM results using highly defined wind fields and also the PBL methods. A series of historical storms (Betsy, 1965, Rita 2005, Ivan 2004, Camille 1969, Katrina 2005 and Andrew 2002) were selected and analyzed.

Two-dimensional wave spectra in the coastal area were calculated and output by WAM to be applied as the input boundary condition to the nearshore wave model STWAVE for the entire 197-storm suite. Example results of the maximum total significant wave height field for RUN801 are given in Figure 2.6-3. The envelope of high waves coincides with that of the wind core (see Figure 2.5-2). The maximum overall H_{mo} for this simulation is 17-m and falls far south of any of the boundary points. However, there is an area of 10-m maxima running all along the SE-LA boundary extending well into the Mississippi coastline. The distribution is skewed toward the east sending what appears to be more energy into the coastline. In the nearshore area, the H_{mo} results diminish to a range near 8-m.

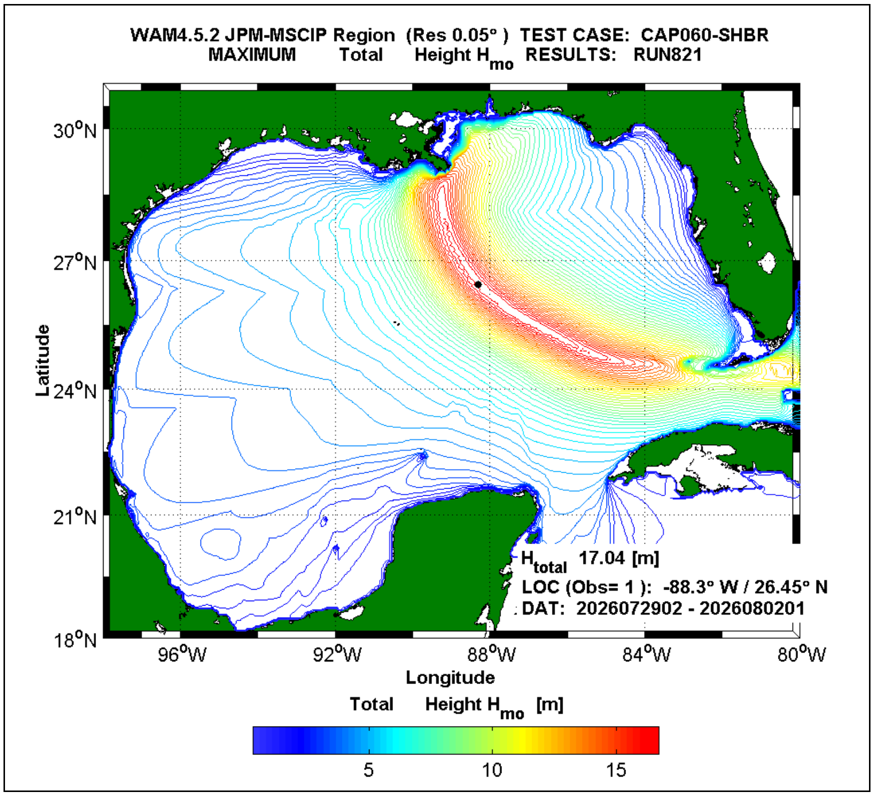


Figure 2.6-3. Maximum overall total significant wave height (in meters) color contour for storm 821

Figure 2.6-4 is an example color contour plot generated to depict the spatial (x axis) and temporal (y-axis) variation of the significant wave heights at each of the 119 output locations designated for STWAVE input boundary information. ST001 through ST046 are the points located along the Mississippi coast. Figure 2.6-4 represents the significant wave heights (integrated from the directional wave spectra of the STWAVE boundary conditions) for storm 821. Rather than isolate only station information (ST001 through ST046) along the Mississippi coast, all 119 output locations are contoured. This aids in the overall evaluation of the wave model's performance, and isolates any potential problems, not only in the local domain (along the Mississippi-Alabama coastline) but the entire shoreline reach. The well defined discontinuity around ST025 requires some explanation. The location of ST001 through ST025 represented in Figure 2.6-2 as the horizontal line seaward Mississippi Sound and extending to the west to the Chandelier Islands. The water depth decreases to about 5-m at the western extent. This causes the wave heights to diminish to near zero. ST026 is the start of the SE-LA boundary (Figure 2.6-2) and starts just offshore of ST012, and is oriented in a NE/SW direction extending to the tip of Louisiana. For storm 821, there is a distinct lobe of high energy values (upwards of nearly 16-m) along the SE-LA boundary. Along the Mississippi coast the wave climate is diminished to 10-m. Despite this reduction in energy level the duration of these high waves occurs over a 12-hour time span. One must realize some of the energy contained in the spectrum may not reach the coast, propagating outside of a ±90º directional plane at the boundary. These conditions emulate that found in the maximum wave height graphic (Figure 2.6-3). The skewed nature of the maximum H_{mo} distribution is evident in this plot, where the Mississippi coastline is affected despite the landfall being located well to the west.

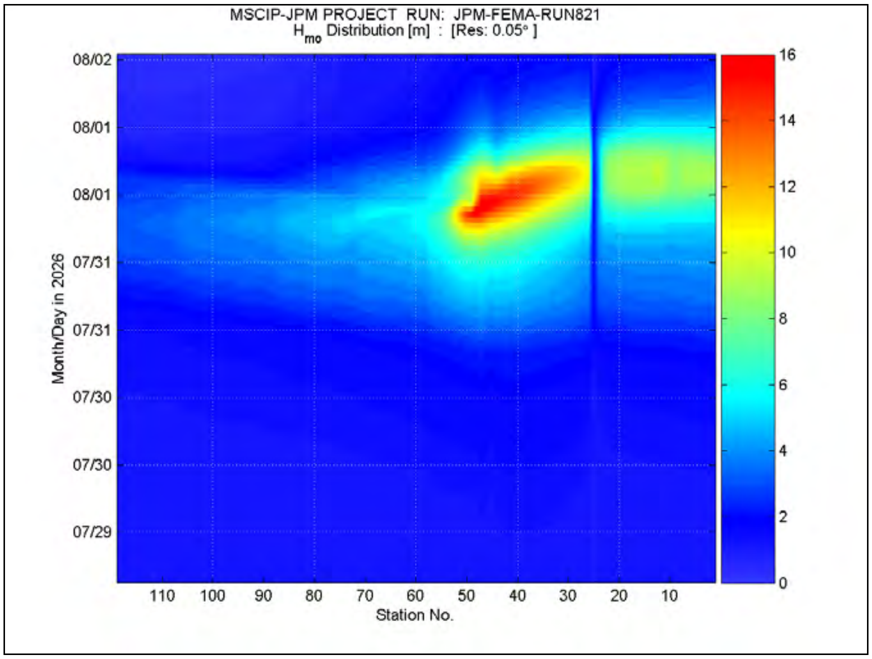


Figure 2.6-4. Spatial and temporal variation in the H_{mo} (in meters) for the 119 boundary output locations for storm 821

Directional spectral wave estimates were generated based on the 28 frequency and 24 direction bands at the 46 nearshore locations in the WAM grid domain for the entire 197-storm suite. This information consists of time (900-s) and spatial (0.05-deg) varying energy density (defined here as m^2-s) for the entire storm simulation period. This information is used as input criteria for STWAVE to estimate the nearshore wave environment.

2.7 Nearshore Wave Modeling

This section describes the numerical modeling of nearshore wave transformation and generation. Nearshore waves are required to calculate wave runup and overtopping on structures, and the wave momentum (radiation stress) contribution to elevated water levels (wave setup). First the nearshore wave model STWAVE and the Boussinesq wave model COULWAVE are briefly described, then the modeling methodology is outlined. Finally, example results are presented.

2.7.1 Computational Models

2.7.1.1 STWAVE

The numerical model STWAVE (Smith 2000; Smith, Sherlock, and Resio 2001; Smith and Smith 2001; Thompson, Smith, and Miller 2004; Smith and Zundel 2006; Smith and Sherlock in publication) was used to generate and transform waves to the shore. STWAVE numerically solves the steady-state conservation of spectral action balance along backward-traced wave rays:

$$(C_{ga})_x \frac{\partial}{\partial x}\frac{C_a C_{ga}\cos(\mu-\alpha)E(f,\alpha)}{\omega_r} + (C_{ga})_y \frac{\partial}{\partial y}\frac{C_a C_{ga}\cos(\mu-\alpha)E(f,\alpha)}{\omega_r} = \sum \frac{S}{\omega_r} \qquad \text{E2.7-1}$$

where

C_{ga} = absolute wave group celerity

x,y = spatial coordinates, subscripts indicate x and y components

C_a = absolute wave celerity

μ = current direction

α = propagation direction of spectral component

E = spectral energy density

f = frequency of spectral component

ω_r = 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 E2.7-1 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 include mild bottom slope and negligible wave reflection; steady waves, currents, and winds; linear refraction and shoaling, and 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:

$$H_{mo_{\max}} = 0.1L \tanh kd \qquad \text{E2.7-2}$$

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 (T_p), and mean wave direction (α_m) at all grid points and two-dimensional spectra at selected grid points. Recent upgrades to STWAVE include an option to input spatially 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.

The inputs required to execute STWAVE include the 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); and bottom friction coefficients (optional). The outputs generated by STWAVE include the 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).

2.7.1.2 COULWAVE

Numerical results based on the standard Boussinesq equations or the equivalent formulations have been shown to give predictions that compared quite well with field data (Elgar and Guza 1985) and laboratory data (Goring 1978, Liu *et al*. 1985). COULWAVE (Cornell University Long and Intermediate Wave model) is based on the Boussinesq-type equations, which are known to be accurate for inviscid wave propagation from fairly deep water (wavelength/depth ~2) all the way to the shoreline (Wei *et al*, 1995).

The model consists of a fairly complex set of partial differential equations that are integrated in time to solve for the free surface elevation. Fundamentally, the Boussinesq equations solved by COULWAVE are inviscid. To accommodate frictional effects, viscous submodels are integrated. To simulate the effects of wave breaking, the eddy viscosity model of Kennedy *et al* (2000) is used here with some modification as given in Lynett (2006b). Energy dissipation is added to the momentum equation when the wave slope exceeds some threshold value, and continues to dissipate until the wave slope reaches some minimum value when the dissipation is turned off.

The moving shoreline condition has been shown to capture shoreline motion due to a wide range of wave frequencies, wave heights, and beach slopes. The shoreline algorithm has been extensively compared with empirical runup laws and existing experimental data for runup due to regular waves. (Korycansky & Lynett, 2005). The model results have also been compared to time-averaged experimental data of overtopping of sloping structures (e.g. Kobayashi & Wurjanto, 1989; Dodd, 1998; Hu et al, 2000) with good agreement. The Boussinesq model results were compared with well-established empirical formulas such as those given by Owen (1984) and Van der Meer & Janssen (1995). A noteworthy result of these comparisons is the conclusion that, when using the wave height and water level at the toe of the last simple slope of the structure, there is no accuracy preference between the empirical formulas and the detailed hydrodynamics (Boussinesq). Thus, for relatively simple profiles where the wave height at the structure toe can be estimated with high confidence, the empirical formulas provide the same level of accuracy as the Boussinesq with significantly less computational expense. On the other hand, if the levee is fronted by a series of slopes or an arbitrary shaped protecting structure, some method must be used to provide the wave height at the toe of the last simple slope. For this situation, the Boussinesq can be used to provide this wave height; however the Boussinesq can also provide the overtopping for such a configuration and would be the logical choice for estimating overtopping, provided the computational resources and expertise required by the modeling are available.

2.7.2 Methodology

STWAVE was applied on two grids with 200 m resolution for the Mississippi coast: 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 wind applied in STWAVE is spatially and temporally variable for all domains. STWAVE was run at 30-min intervals for 93 quasi-time steps (46.5 hrs). The model output includes wave parameters (height, peak wave period, and mean direction) to provide wave parameters for the calculation wave runup and overtopping on structures 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. Figure 2.7-1 shows the bathymetry for the MS-AL grid and Figure 2.7-2 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.

Levees and other barriers such as seawalls and roadways are included in the STWAVE and COULWAVE grids as bathymetric/topographic features. The STWAVE grids bathymetry/topography was updated to include the lines of defense 3 and 4. However, the STWAVE model cannot resolve the wave setup that occurs near structures such as levees and seawalls due to the resolution of the grid and the assumption of negligible wave reflection. Local wave setup very near structures such as levees and seawalls can increase the water level at the structure. To capture the additional wave setup near the line of defense structures, COULWAVE was applied. COULWAVE was used to generate a lookup table that, given input wave boundary condition information from STWAVE and a representative profile, computes the additional wave setup and wave height at the toe of the structure.

Representative profile data was collected at Hancock, west Harrison, east Harrison, and Jackson counties. Profile data that extended from the mainland into Mississippi Sound at locations near Waveland, Pass Christian, Harrison County just west of the western end of Deer Island, and in the Pascagoula area were used to develop the representative profiles for the Mississippi coast. To evaluate lines of defense 3 and 4, which include both a levee and seawall structure , the structures were incorporated into the 4 representative profiles, giving a total of 8 profiles to be simulated. The seawall had a 15 ft (NAVD88 2004.65) elevation for all four profiles. The levee was modeled with a 30 ft elevation for the Waveland, Pass Christian, and west of Deer Island profiles; and modeled with a 15 ft elevation on the Pascagoula profile.

To develop the lookup table a set of independent parameters and their ranges were specified. The independent parameters are levee slope, levee crest height, incident wave height, peak wave period, and surge water elevation. All of the hydrodynamic parameters are specified at 600 ft from the levee toe, and represent information provided from STWAVE and ADCIRC runs. The levee slope evaluated was 1:3. The seawall was approximated as a very steep slope (5:1). The other parameters used to develop the lookup table are given in Table 2.7-1. For each parameter combination, a Boussinesq simulation was run for the 8 profiles. Save stations near proposed structures were associated with the most appropriate representative profile. For the Mississippi coast, a total of 8 x 3 x 3 X 3 = 216 simulations were run to create the lookup table.

For each simulation, time series of free surface elevation, depth-averaged velocity, and mass flux are recorded. Each time series is distilled to a significant wave height, a mean water level (from which the local wave setup is obtained), and a mean flux. Note that mean flux, when measured on the crest of a levee, is identical to the overtopping rate in units of water volume/time per unit length of crest. Using interpolation routines, the wave height, wave setup, and overtopping values for any combination of input conditions bracketed by the independent parameter ranges shown above can be obtained. The lookup table script outputs the wave setup at the structure toe, the wave height at the toe, and the overtopping rate predicted by COULWAVE.

The entire 197 storm suite was simulated with STWAVE forced with input boundary conditions calculated by the offshore wave model WAM and water level supplied by the surge model. 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. Radiation stress gradients were calculated and applied as a forcing condition to the surge model. To provide the wave height and period for boundary conditions to COULWAVE, the STWAVE output was processed to extract the significant wave height at the surge peak for save stations near the proposed structures.

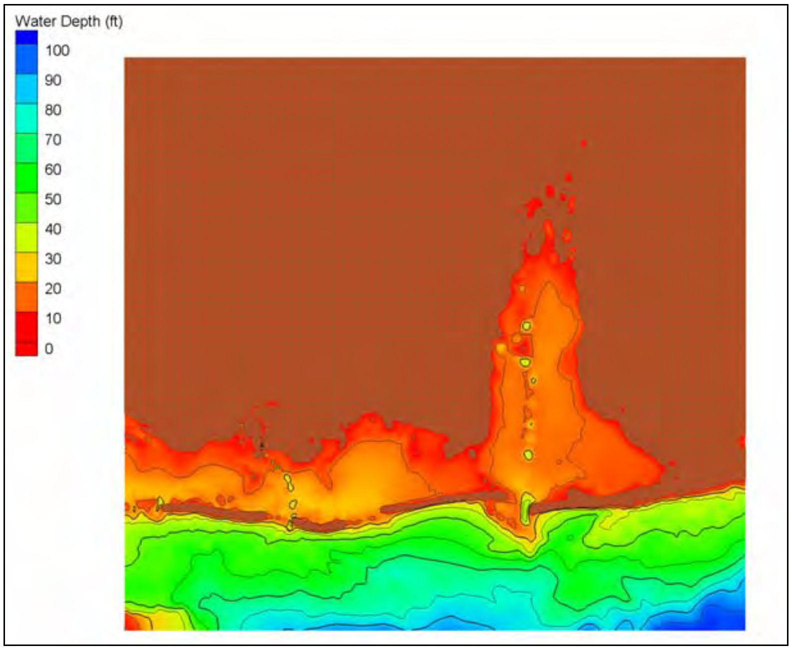


Figure 2.7-1. MS-AL Bathymetry Grid (depths in feet)

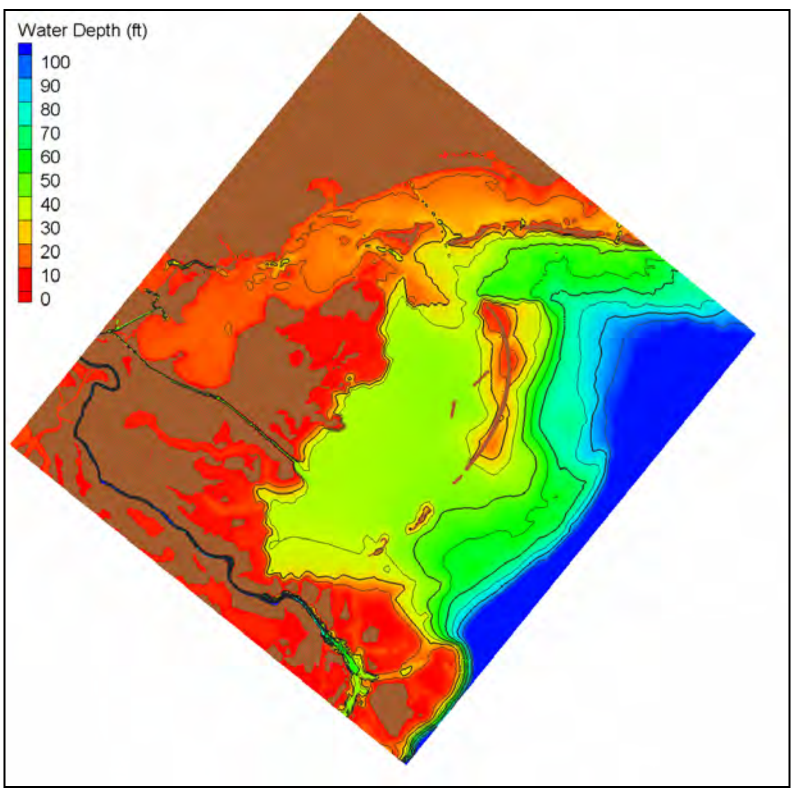


Figure 2.7-2. West MS-SE LA Bathymetry Grid (depths in feet)

Table 2.7-1.
Water Levels and Wave Parameters Modeled with COULWAVE

Water Level Relative to Structure Crest (ft)	Wave Heights (ft)	Peak Periods (sec)
0	2, 5, 8	8, 12, 16
-2	2, 5, 8	8, 12, 16
-4	2, 5, 8	8, 12, 16

2.7.3 Results

Example output generated from the STWAVE model results are provided in Figures 2.7-3 to 2.7-6. Figures 2.7-3 to 2.7-4 show the maximum significant wave height and coincident direction produced by storm 027 for the MS-AL and MS-SE LA grids, respectively. Figures 2.7-5 to 2.7-6 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 as boundary conditions for calculating wave runup and overtopping, wave forcing on structures, or other design purposes.

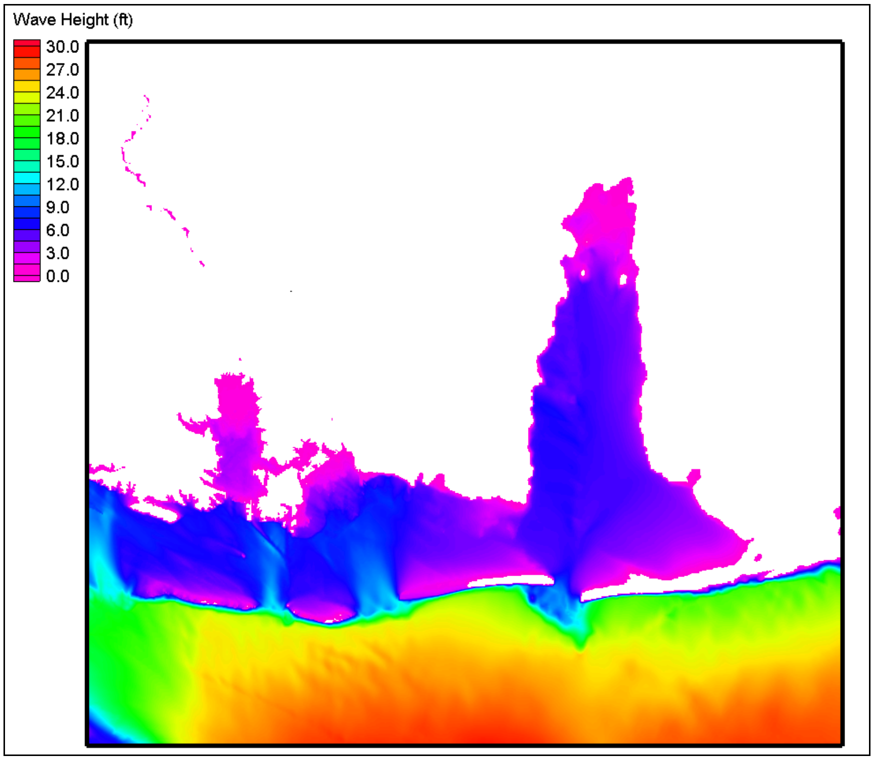


Figure 2.7-3. Maximum significant wave height for the MS-AL grid for storm 027

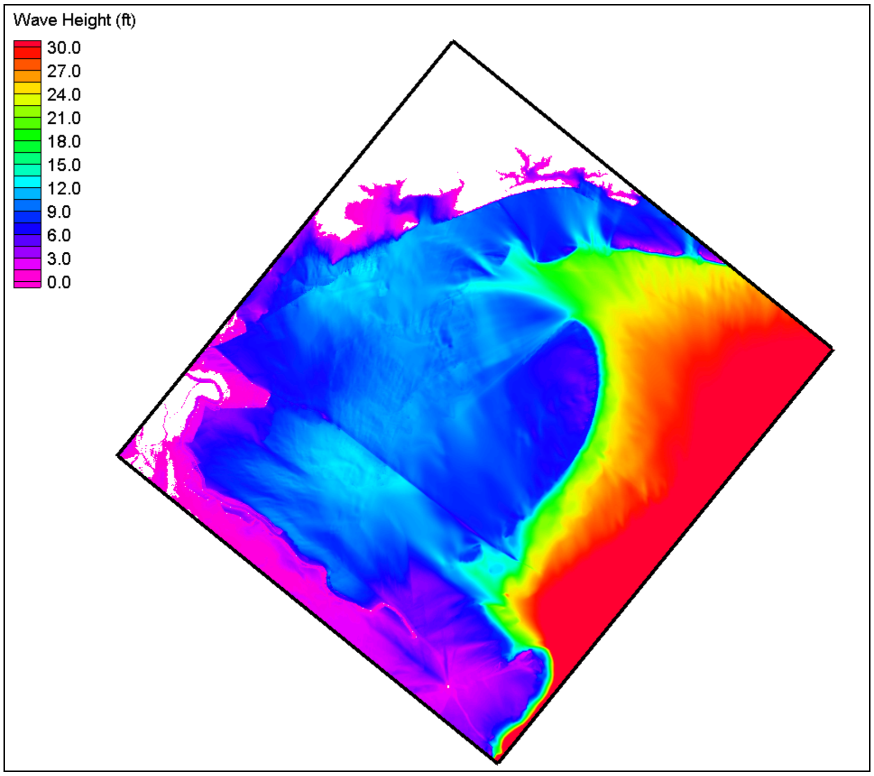


Figure 2.7-4. Maximum significant wave height for the MS-SE LA grid for storm 027

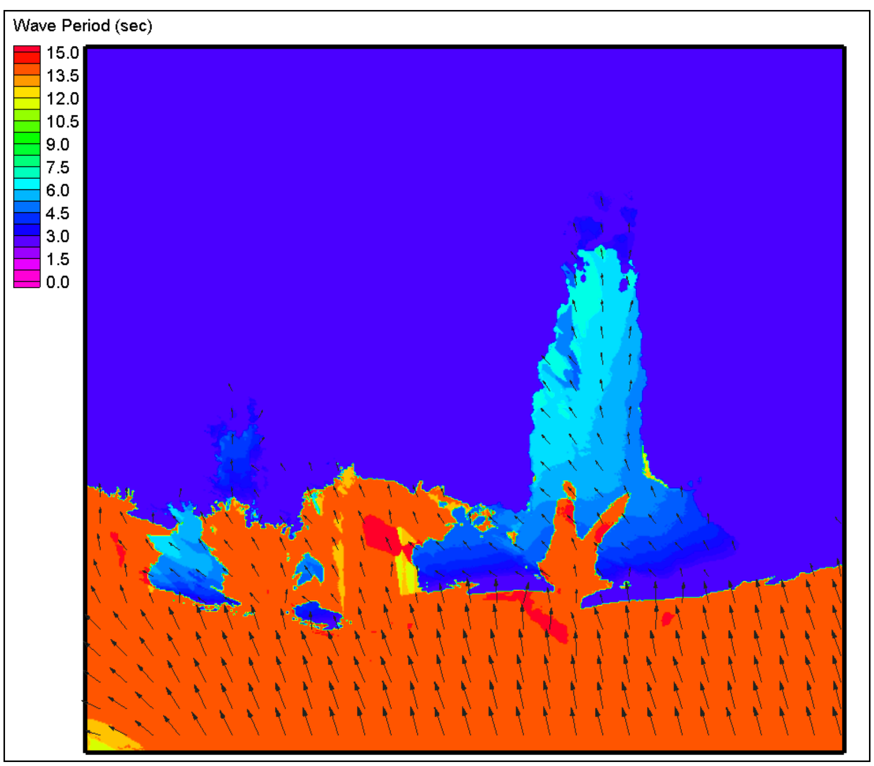


Figure 2.7-5. Peak wave period and direction at the time of maximum wave height for the MS-AL grid for storm 027

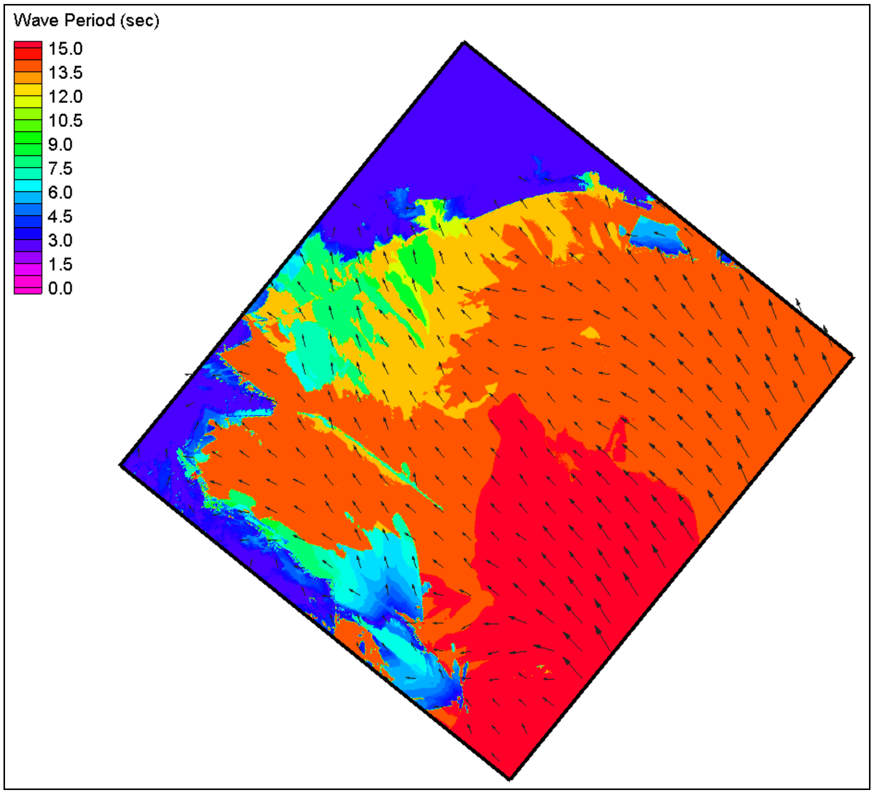


Figure 2.7-6. Peak wave period and direction at the time of maximum wave height for the MS-SE LA grid for storm 027

For storm 027 at a save location near a proposed ring levee in the Pascagoula area, the wave height and period calculated by STWAVE at the peak of the storm was approximately 3.5 ft and 13.5 sec, respectively. With these input parameters, the wave setup at the toe of the structure obtained from the COULWAVE generated lookup table was about 1.3 ft. These calculations were made for all 27 storms in the measure evaluation storm suite and the local wave setup was added to the water level. In general, the additional wave setup calculated near the structure was less than 1 ft, but occasionally was calculated to be as much as approximately 1.75 ft. Note that this additional wave setup was only applied to estimate water levels at the proposed lines of defense to assist in preliminary cost estimation.

2.8 Storm Surge Modeling

The Advanced CIRCulation Model (ADCIRC) was selected as the basis for the surge modeling effort. The domain and geometric/topographic description and resulting computational grid provides for a common domain and grid from the Sabine River to Mobile Bay which extends inland across the floodplains of Southern Louisiana and Mississippi (to the 30 to 75 ft contour NAVD88 2004.65) and extends seaward to the deep Atlantic Ocean. The grid, referred to as SL15, domain boundaries were selected to ensure the correct development, propagation and attenuation of storm surge without necessitating nesting solutions or specifying ad hoc boundary conditions for tides or storm surge. The grid will be used for all coastal analysis for Louisiana and Mississippi to ensure consistency and matching solutions at state line/region boundaries.

2.8.1 Computational Model

ADCIRC-2DDI, the two-dimensional, depth-integrated implementation of the ADCIRC coastal ocean model, was used to perform the hydrodynamic computations in this study (Luettich et al. 1992, Westerink et al. 1992, Westerink et al. 1993, Luettich and Fulcher 2004, Luettich and Westerink 2004). 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):

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

E2.8-1

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

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

E2.8-2

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

$$\frac{\partial \zeta}{\partial t} + \frac{1}{R\cos\phi}\left[\frac{\partial UH}{\partial \lambda} + \frac{\partial (UV\cos\phi)}{\partial \phi}\right] = 0 \qquad \text{E2.8-3}$$

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,

$H = \zeta + h$ = total water column depth,

h = bathymetric depth relative to the geoid,

$f = 2\Omega \sin \varphi$ = Coriolis parameter,

Ω = angular speed of the earth,

p_s = atmospheric pressure at free surface,

g = acceleration due to gravity,

η = effective Newtonian equilibrium tide-generating potential parameter,

ρ_0 = reference density of water,

$\tau_{s\lambda}$ and $\tau_{s\varphi}$ = applied free surface stresses in the longitudinal and latitudinal directions, respectively, and

τ = bottom shear stress and is given by the expression $C_f\,(U^2 + V^2)^{1/2}\,/H$ where C_f is the bottom friction coefficient.

The momentum equations (Equations 1 and 2) are differentiated with respect to λ and τ and substituted into the time differentiated continuity equation (Equation 3) to develop the following Generalized Wave Continuity Equation (GWCE):

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

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

$$-\frac{1}{R}\frac{\partial}{\partial\phi}\left[\frac{1}{R\cos\phi}\left(\frac{\partial HVV}{\partial\lambda} + \frac{\partial HVV\cos\phi}{\partial\phi}\right) + UUH\,\frac{\tan\phi}{R} + 2\,\omega\sin\phi\,HU\right] \qquad \text{E2.8-4}$$

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

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

The ADCIRC model solves the GWCE in conjunction with the primitive momentum equations given in Equations 1 and 2. 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.

2.8.2 Methodology

The ADCIRC grid utilized for this study is that which was calibrated and validated for IPET with Hurricane Katrina data and subsequently validated with data from Hurricane Rita for this and other coastal surge studies conducted by USACE. The development of an accurate unstructured grid storm surge model of Southern Louisiana and Mississippi requires appropriate selection of the model domain and optimal resolution of features controlling surge propagation. The SL15 model domain, shown in Figure 2.8-1, has an eastern open ocean boundary that lies along the 60° W meridian, extending south from the vicinity of Glace Bay in Nova Scotia, Canada to the vicinity of Coracora Island in eastern Venezuela (Westerink, Luettich and Muccino 1994, Blain et al. 1994, Mukai et al. 2002, Westerink et al., 2006, Ebersole et al., 2006). This domain has a superior open ocean boundary that is primarily located in the deep ocean and lies outside of any resonant basin. There is little geometric complexity along this boundary. Tidal response is dominated by the astronomical constituents and nonlinear energy is limited due to the depth. The boundary is not located near tidal amphidromes. Hurricane storm surge response along this boundary is essentially an inverted barometer pressure effect directly correlated to the atmospheric pressure deficit in the meteorological forcing; it can therefore be easily specified. This boundary allows the model to accurately capture basin-to-basin and shelf-to-basin physics. Hurricane forerunner and Gulf of Mexico resonant modes can be generated as the hurricane moves from the Atlantic into the Gulf.

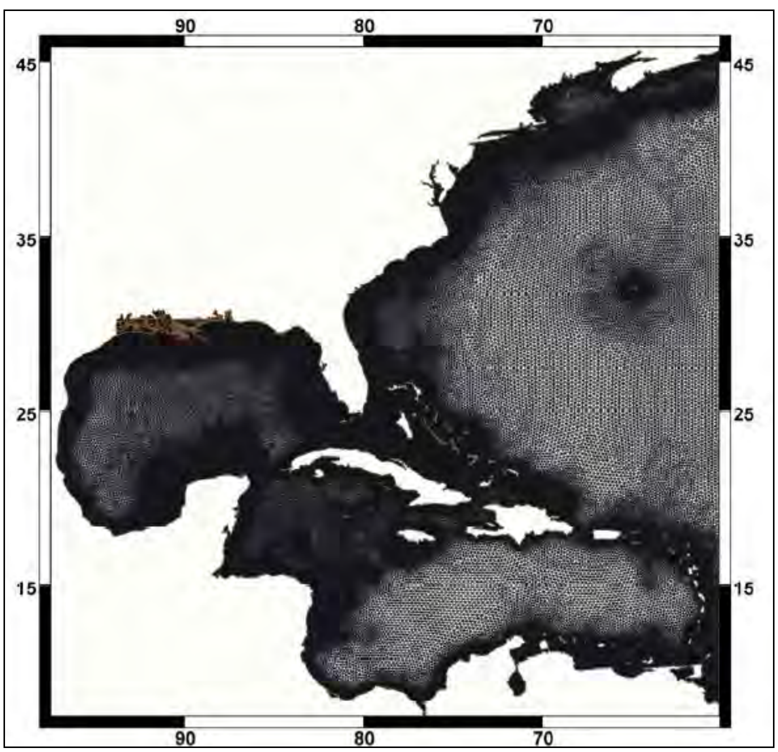


Figure 2.8-1. The ADCIRC SL15 Unstructured Grid

The grid design provides localized refinement of the coastal floodplains of Southern Louisiana and Mississippi and of the important hydraulic features. The level of detail in Southern Louisiana and Mississippi is unprecedented, with nodal spacing reaching as low as 100 ft in the most highly refined areas. Unstructured grids can resolve the critical features and the associated local flow processes with orders of magnitude fewer computational nodes than a structured grid, because the latter is limited in its ability to provide resolution on a localized basis and fine resolution generally extends far outside the necessary area. The SL15 grid is refined locally to resolve features such as inlets, rivers, navigation channels, levee systems and local topography/bathymetry. In addition, wave breaking zones have been identified based on local bathymetric gradients, and a swath of 150 to 700 ft grid resolution has been placed along the coast and over barrier islands to ensure that the grid scale of the flow model is consistent with that of the STWAVE models. The STWAVE forcing function is accommodated by adding a high level of resolution where significant gradients in the wave radiation stresses and forcing of surge through wave transformation and breaking are the largest. The high resolution zones allow for the strong wave radiation stress gradients to fully force the water body in these important regions and ensures that the resulting wave radiation stress induced set up is sufficiently accurate. Barrier islands were in particular very highly resolved to 150 to 250 ft due to the significant wave breaking and the resulting important wave radiations stresses as well as the very high currents that develop over the features. The SL15 computational grid contains 2,137,978 nodes and 4,184,778 elements. Grid resolution varies from approximately 12-15 mi in the deep Atlantic Ocean to about 100 ft in Louisiana and Mississippi. The high grid resolution required for the study region leads to a final grid with more than 90% of the computational nodes placed within or upon the shelf adjacent to Southern Louisiana and Mississippi, enabling sufficient resolution while minimizing the cost of including such an extensive domain. Geometry, topography and bathymetry in the SL15 model were all defined to replicate the prevailing conditions in August 2005 prior to Hurricane Katrina with the exceptions of some of the barrier islands and area between Lake Pontchartrain and Lake Borgne that were included as post Katrina September 2005 configurations. The bathymetric and topographic data was interpolated to the SL15 computational mesh by moving progressively from the coarsest and deepest to finest and shallowest areas of the computational domain.

Levee and road systems that are barriers to flood propagation are features that generally fall below the defined grid scale and represent a non-hydrostatic flow scenario. It is most effective to treat these structures as sub-grid scale parameterized weirs within the domain. ADCIRC defines these as barrier boundaries by a pair of computational nodes with a specified crown height (Westerink et al. 2001). Once water level reaches a height exceeding the crown height, the flow across the structure is computed according to basic weir formulae. This is accomplished by examining each node in the defined pair for their respective water surface heights and computing flow according to the difference in water elevation. The resulting flux is specified as a normal flow from the node with the higher water level to the node with the lower water level for each node pair. Lines of Defense 3 and 4, as described in Section 2.1, were incorporated into the ADCIRC grid as sub-grid scale weirs. Weir boundary conditions also are implemented for external barrier boundaries, which permit surge that overtops levee structures at the edge of the domain to transmit flow out of the computational area.

The entire JPM-OS synthetic storm suite was simulated for the no project condition forced with the wind and pressure fields discussed in section 2.5 and radiation stress gradients calculated by STWAVE (see section 2.7). The ADCIRC and STWAVE models were coupled in that wind and water levels computed by ADCIRC were applied as a boundary condition for STWAVE, STWAVE was run and the resulting radiation stress gradients were then applied as forcing to ADCIRC to compute the final water level.

2.8.3 Results

The primary goal of the ADCIRC simulations was to estimate overall peak water level for each storm in the JPM-OS suite for the calculation of stage-frequency curves for the no project condition and with proposed protection measures in place. 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. Example output generated from the ADCIRC model results are provided in Figures 2.8-2 to 2.8-6 and discussed below. The peak surge elevations were saved at stations along the Mississippi coast for the entire JPM-OS storm suite and the computed water levels used as input for the JPM analysis.

Figure 2.8-2 is the envelope of maximum water level for storm 027 for the no project condition, Figure 2.8-3 is the envelope of maximum water level for the same storm with line of defense 3 in place, and Figure 2.8-4 is the difference between the line of defense 3 and the no project condition. For this particular storm, the maximum water level envelope for the no project condition (Figure 2.8-2) shows that the highest water levels are in the vicinity of Saint Louis Bay where the water level reaches 26 ft NAVD88 (2004.65). Approximately 25 miles to the east, Biloxi Bay water levels are at 18 ft NAVD88 (2004.65) and Pascagoula is at 10 ft NAVD88 (2004.65). The bays overflow their banks and the surrounding low-lying areas are inundated. The maximum water level envelope for the line of defense 3 (Figure 2.8-3) shows that the highest water levels are *seaward* of the line of defense 3 in the vicinity of Bay Saint Louis where the water level in the Gulf reaches 27 ft NAVD88 (2004.65) along the shoreline west of the entrance to Saint Louis Bay. Water within Saint Louis Bay is locally affected by the winds with water levels of 3-5 ft, but generally remains within its banks. Approximately 25 miles to the east, Biloxi Bay water levels are at 3 ft NAVD88 (2004.65) and Pascagoula remains at 10 ft NAVD88 (2004.65) since it is unprotected by the line of defense 3. The difference between the maximum water levels with line 3 of defense and the no project condition (Figure 2.8-4) shows areas in blue (Saint Louis Bay and Biloxi Bay) where water levels are reduced, indicating that the line of defense 3 provides protection to these regions. Water levels are reduced by 18-23 ft in St Louis Bay and 14 ft in Biloxi Bay. Figure 2.8-4 also shows slightly (~1-ft) higher water levels in the Gulf as indicated by the yellow and orange areas.

Figure 2.8-5 is the envelope of maximum water level for the storm 027 with line of defense 4 in place, and Figure 2.8-6 is the difference between the line of defense 4 and the no project condition. The results are very similar to the line of defense 3 results. The maximum water level envelope for the line of defense 4 (Figure 2.8-5) also shows that the highest water levels are *seaward* of the line of defense 4 in the vicinity of Bay Saint Louis where the water level in the Gulf reaches 27 ft NAVD88 (2004.65) along the shoreline west of the entrance to Saint Louis Bay. Water within Saint Louis Bay is again locally affected by the winds with water levels of 2-5 ft, but generally remains within its banks. Approximately 25 miles to the east, Biloxi Bay water levels are at 3 ft NAVD88 (2004.65) and Pascagoula remains at 10 ft NAVD88 (2004.65) since it is unprotected by the line of defense 4. The difference between the maximum water levels with line 4 of defense and the no project condition (Figure 2.8-6) shows areas in blue (Saint Louis Bay and Biloxi Bay) where water levels are reduced, indicating that the line of defense 4 provides protection to these regions. Water levels are reduced by 18-23 ft in St Louis Bay and 14 ft in Biloxi Bay. Figure 2.8-6 also shows slightly (~1-ft) higher water levels in the Gulf as indicated by the yellow and orange areas.

The peak surge elevations were saved at stations along the Mississippi coast for the entire JPM-OS storm suite and the computed water levels used as input for the JPM analysis. The peak surge elevations for the set of storms run with the lines of defense in place were also saved at stations along the coast and stage frequency curves developed with the methodology discussed in section 2.4.3. The resulting stage frequency relationships are given in section 2.9.

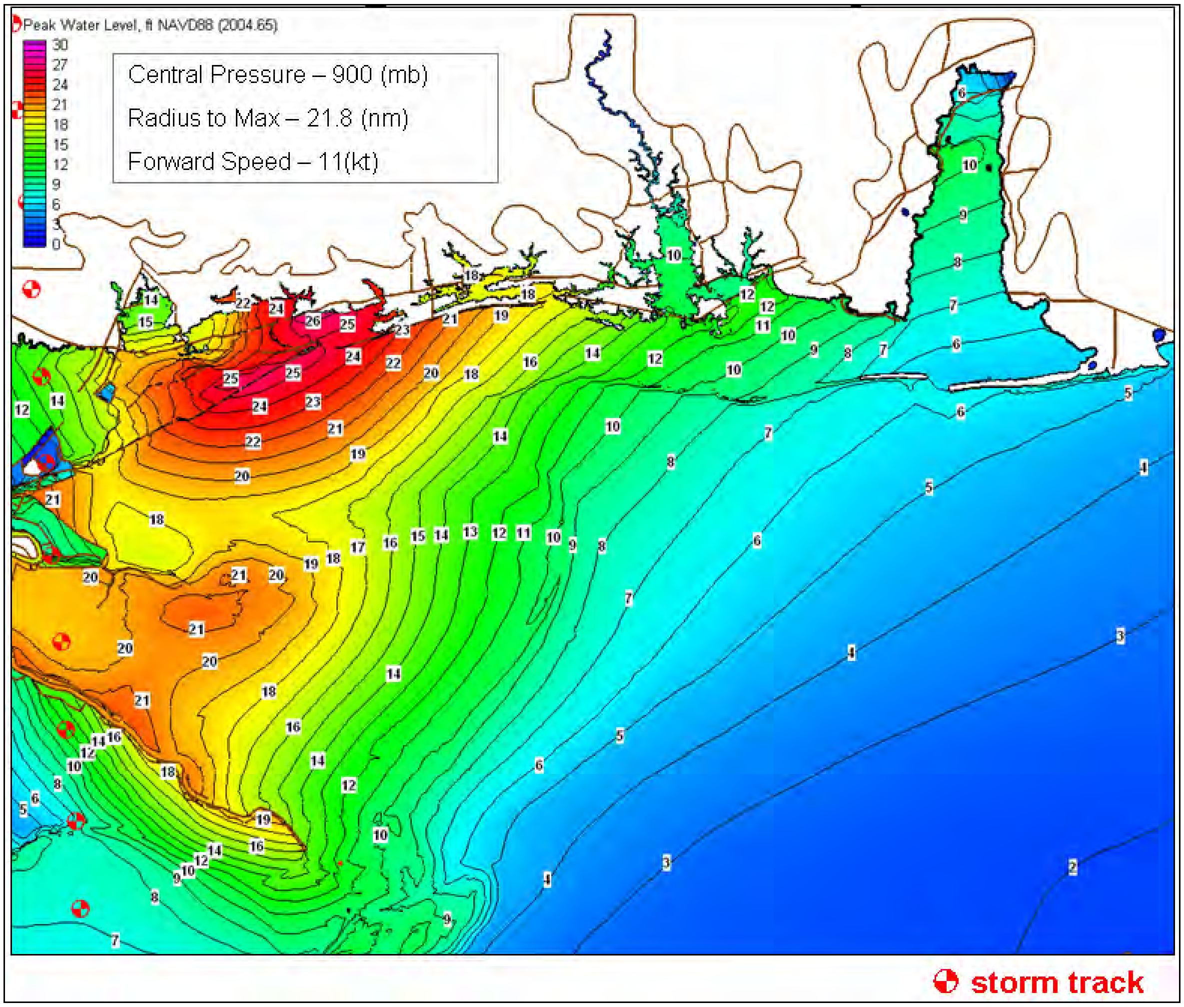


Figure 2.8-2. Envelope of maximum water level for storm 027 for the no project condition

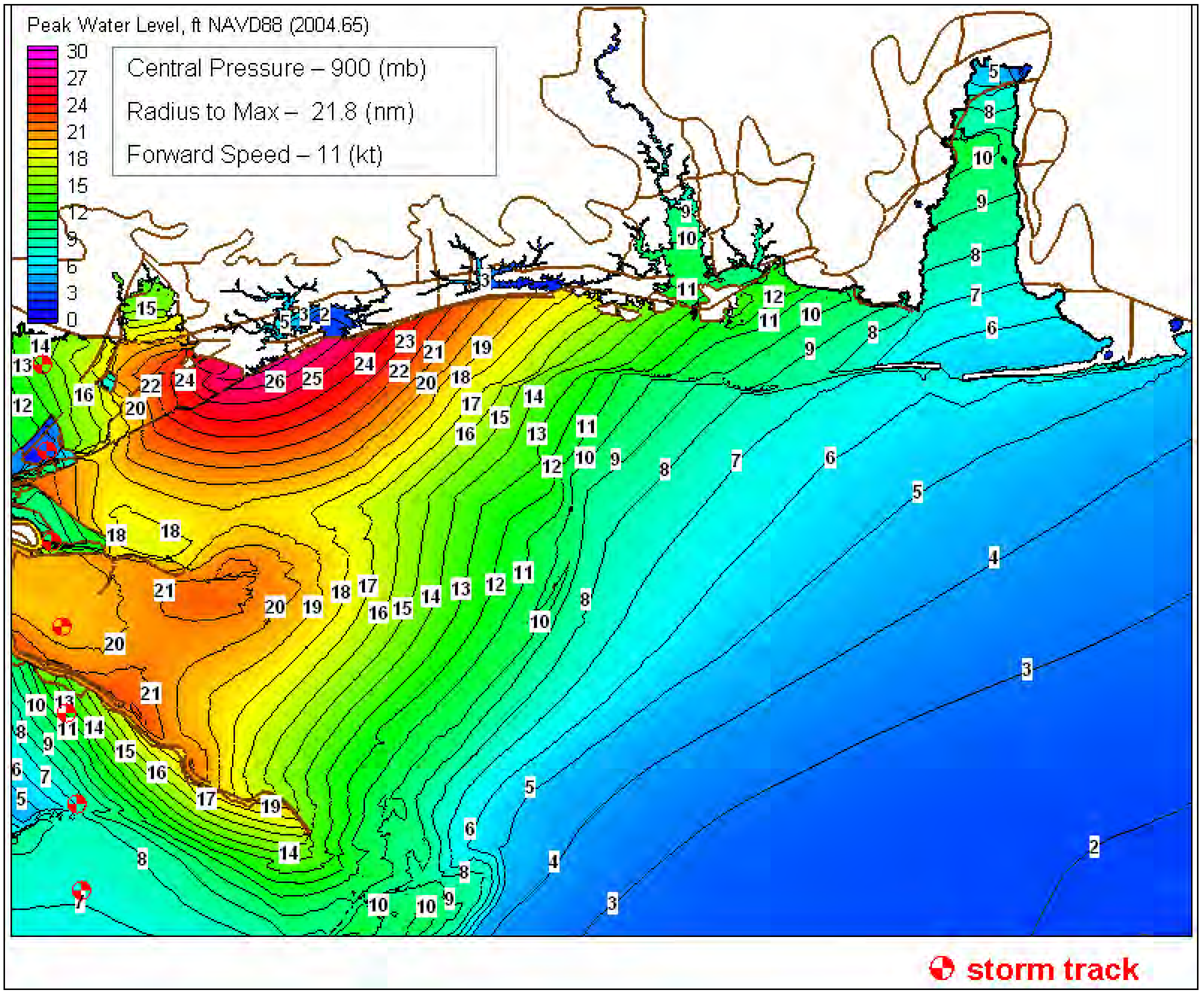


Figure 2.8-3. Envelope of maximum water level for storm 027 for the line of defense 3

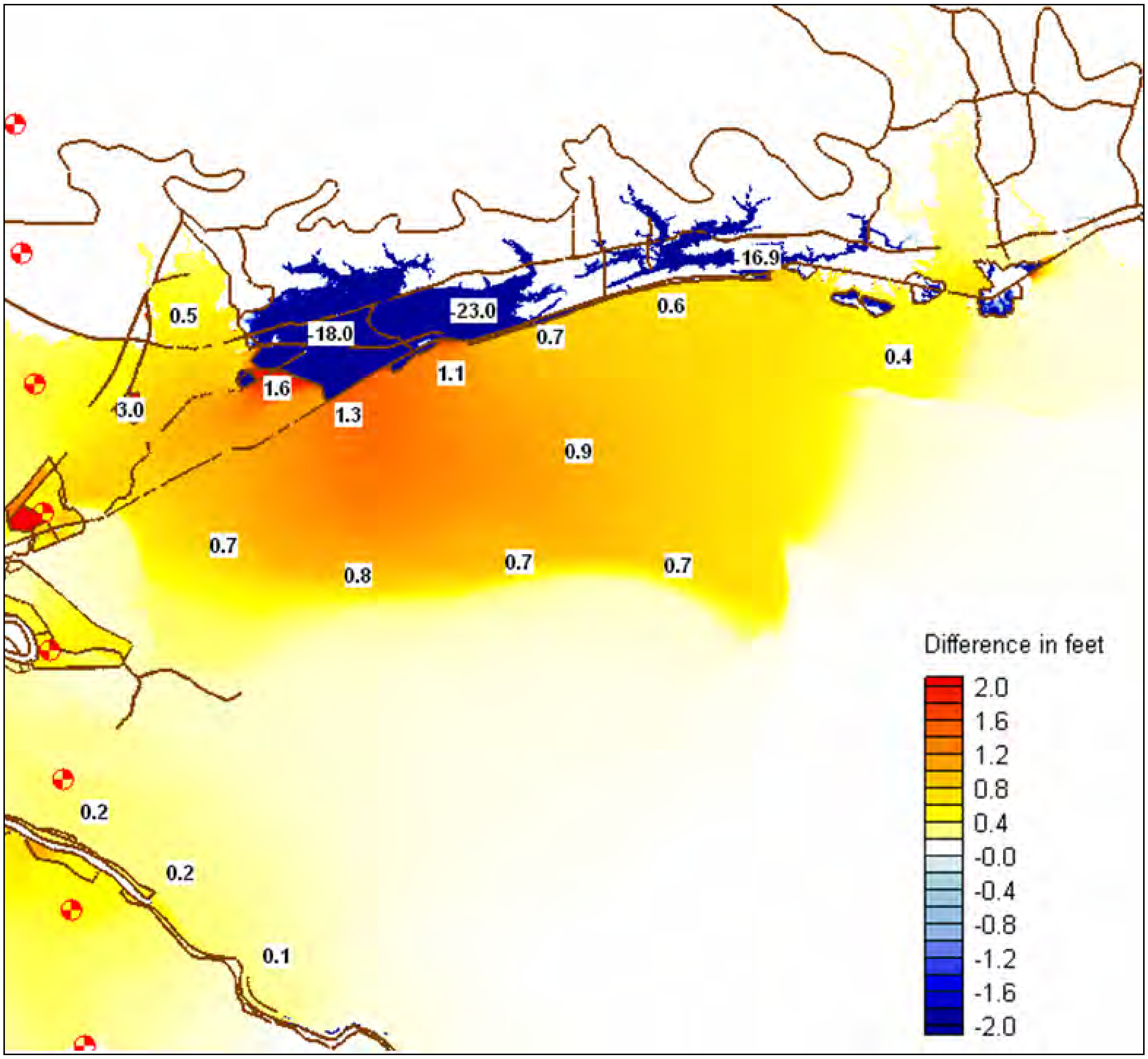


Figure 2.8-4. Difference in maximum water level between line of defense 3 and the no project condition for storm 027

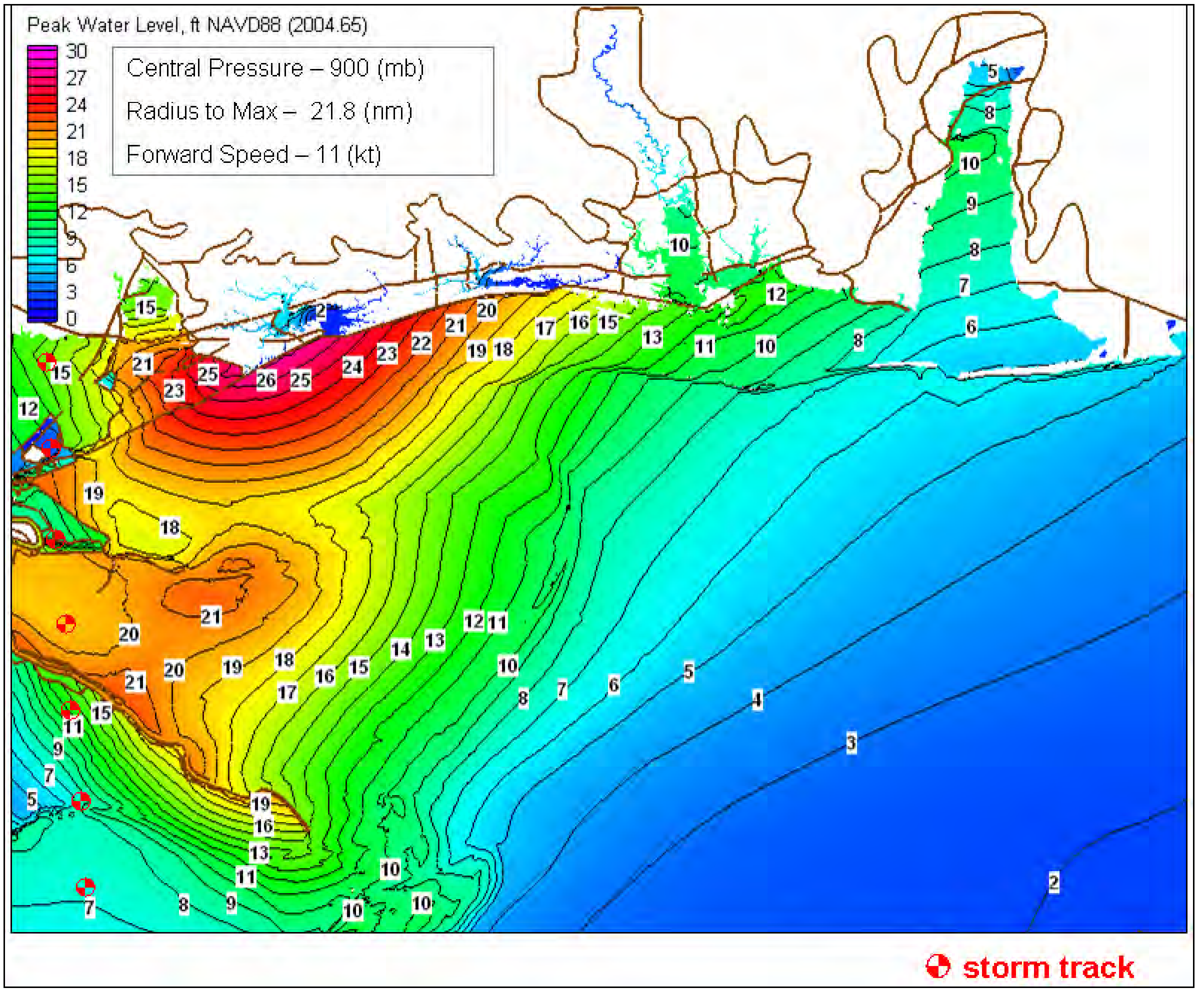


Figure 2.8-5. Envelope of maximum water level for storm 027 for the line of defense 4

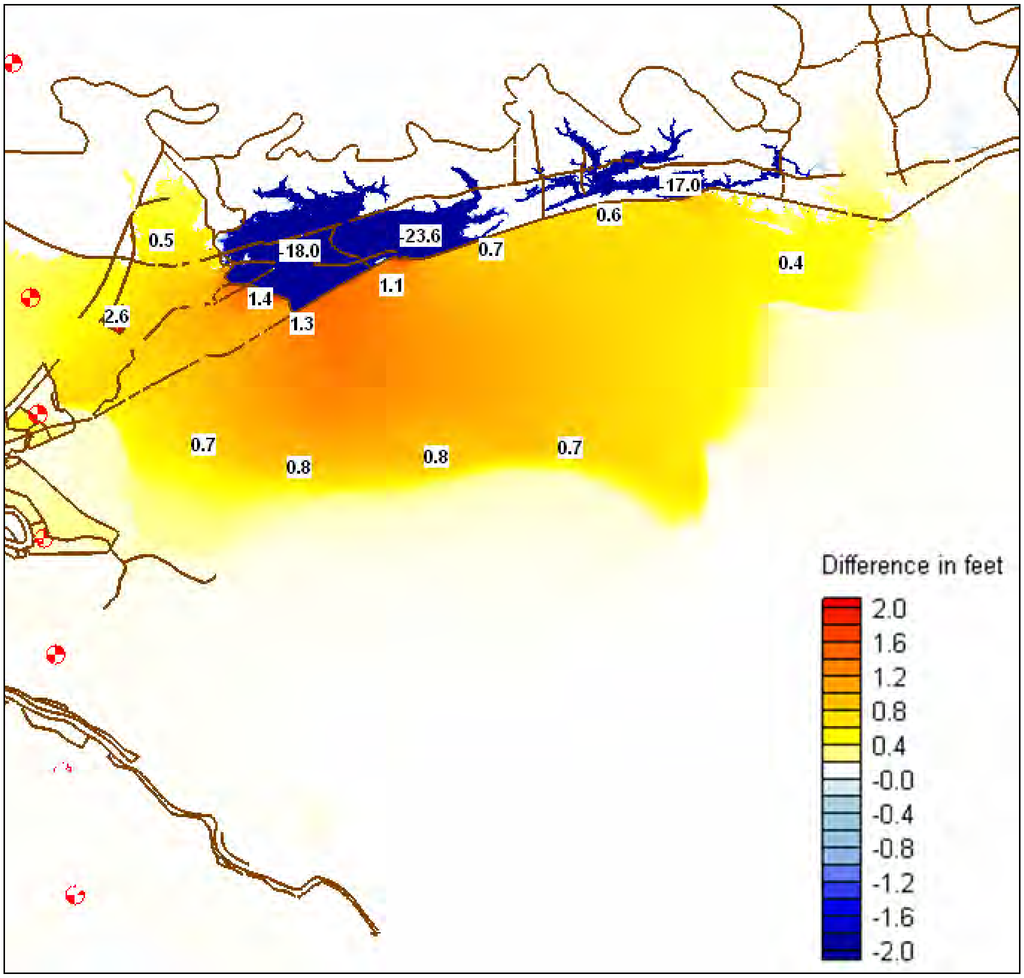


Figure 2.8-6. Difference in maximum water level between line of defense 4 and the no project condition for storm 027

2.8.4 Line of Defense 5 Results

Six maximum possible intensity (MPI) storms with landfall points along the Mississippi coast were simulated to determine inundation limits for the Mississippi coastline. These limits are used to establish line of defense 5. The six MPI storms made landfall at various points along the coast as shown in Figure 2.8-7. All MPI storms were defined at their most intense point as having a minimum central pressure of 880 mb, radius to maximum winds of 36 n mi, and a forward speed of 11 kt.

Peak water level envelopes from each of the six MPI simulations were computed. The six peak water level envelopes were then compared to compute the “peak of peaks”, which is considered the inundation limit along the entire Mississippi coastline (Figure 2.8-8). The maximum water level along the Mississippi coastline was determined to be approximately 30 ft along the entire western half of the state and east of Pascagoula. The landward extent of the inundation indicates the storm surge reaches Interstate 10 for much of the western portion of the state. Lower peaks near Biloxi and Mobile Bay (24-27 ft) may be attributed to the protection afforded by the barrier islands.

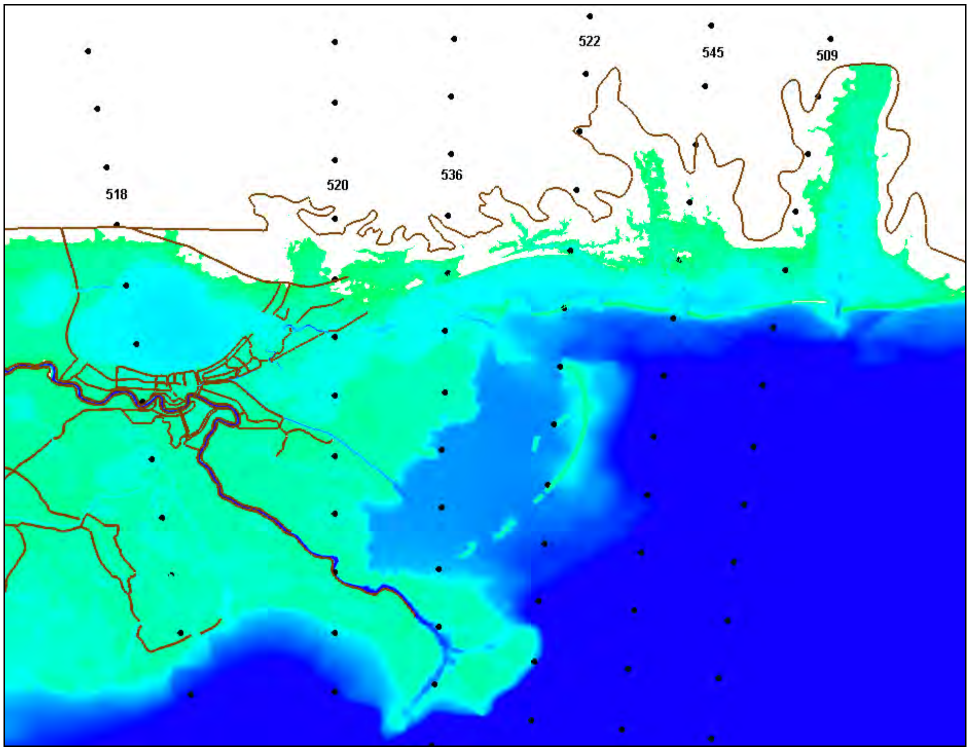


Figure 2.8-7. Storm Tracks for Maximum Possible Intensity Storms

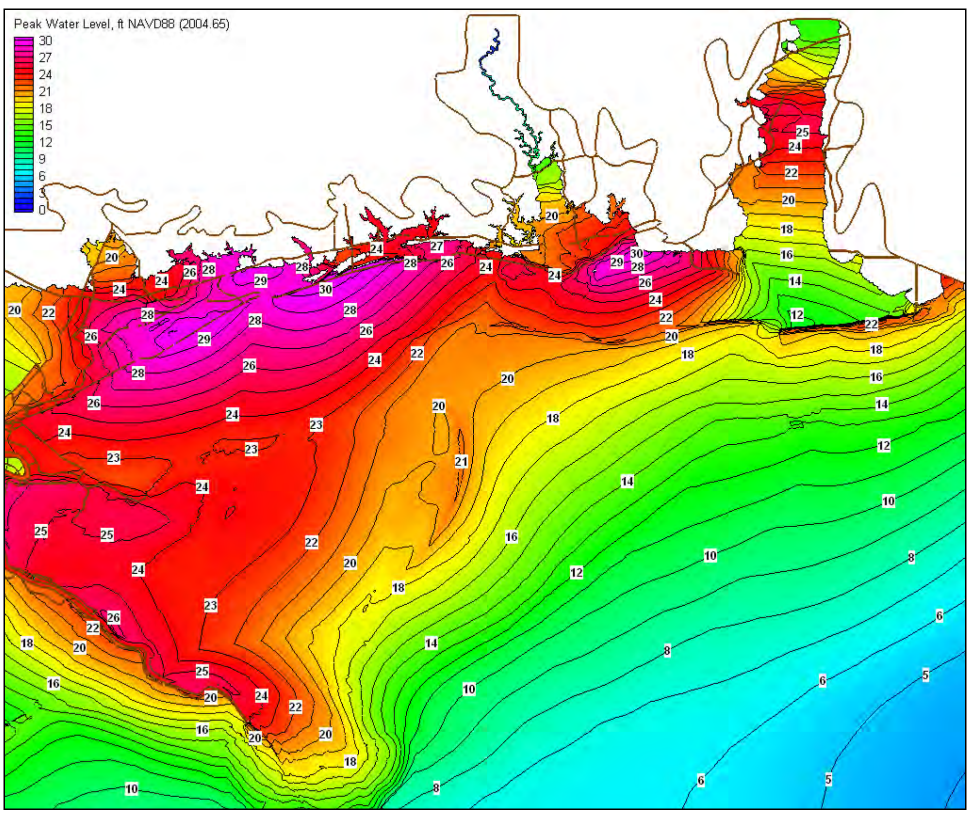


Figure 2.8-8. Envelope of Maximum Water Level for all MPI Storms

2.9 Stage Frequency Curves

The purpose of hydrodynamic modeling was to estimate the surge and wave conditions for the no project condition and with lines of defense 3 and 4 in place. The expected return periods for those surge and wave conditions is also required to quantify the risk for the existing condition and the level of protection that might be possible with the proposed protection measures. Sixty-two save locations were selected to evaluate damage reaches across Mississippi. The surge and wave conditions at these 62 locations, plus 18 additional locations in the Mississippi Sound and seaward of the barrier islands were saved and analyzed with the JPM-OS methodology described in section 2.4. The calculation of the hydrodynamic conditions has been detailed in sections 2.5 to 2.8. In this section, a description of the integrated modeling system is given and the location of the save stations identified. Finally, the frequency results for both the surge and waves are presented.

2.9.1 Integrated Modeling System

Section 2.4 described the statistical methodology and sections 2.5 to 2.8 detailed the models and methodologies applied for computing the surge and wave estimates for the Mississippi coast. Each component is part of an integrated modeling system. For completeness, the integrated system is presented. A schematic diagram of the system is shown in Figure 2.9-1.

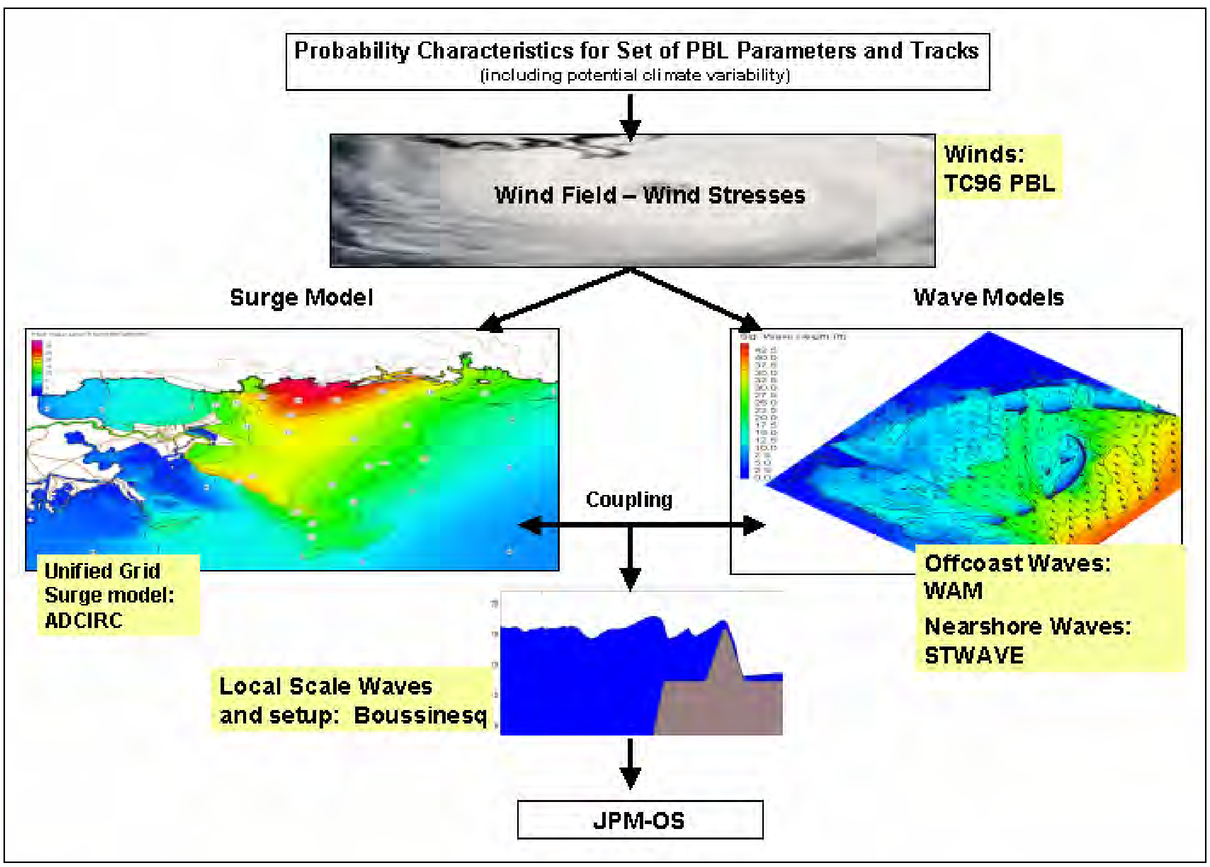


Figure 2.9-1. Diagram of Modeling System for Coastal Inundation Applications

First, for each defined storm (a track and its time-varying wind field parameters) the TC96 PBL model (Thompson and Cardone, 1996) is used to construct 15-minute snapshots of wind and pressure fields for driving surge and wave models. ADCIRC is then run to compute the wind-driven surge component. In parallel with the initial ADCIRC runs, the large-domain, discrete, time-dependent spectral wave model WAM is run to calculate directional wave spectra that serve as boundary conditions for local-domain, near-coast wave model STWAVE. Using initial water levels from ADCIRC, winds that include the effects of sheltering due to land boundaries, and spectral boundary conditions from the large-domain wave model, STWAVE is run to produce wave fields and estimated radiation stress fields. The radiation stress fields are added to the PBL-estimated wind stresses, and the ADCIRC model is run again for the time period during which the radiation stresses potentially make a significant contribution to the water levels.

For simulations with the proposed structures, a method based on Boussinesq modeling (using a lookup table based on interpolations from generic runs) is used to provide estimates of the incremental contribution to the water level at the structure. The water levels from the second ADCIRC run and waves from STWAVE in locations adjacent to structures are provided as the boundary conditions for driving the Boussinesq-based runs.

2.9.2 Save Stations

Sixty-two save stations were identified to evaluate damage reaches across Mississippi. The surge and wave conditions at these 62 stations, plus 18 additional stations in the Mississippi Sound and seaward of the barrier islands were saved and analyzed. Figure 2.9-2 shows the location of each station.

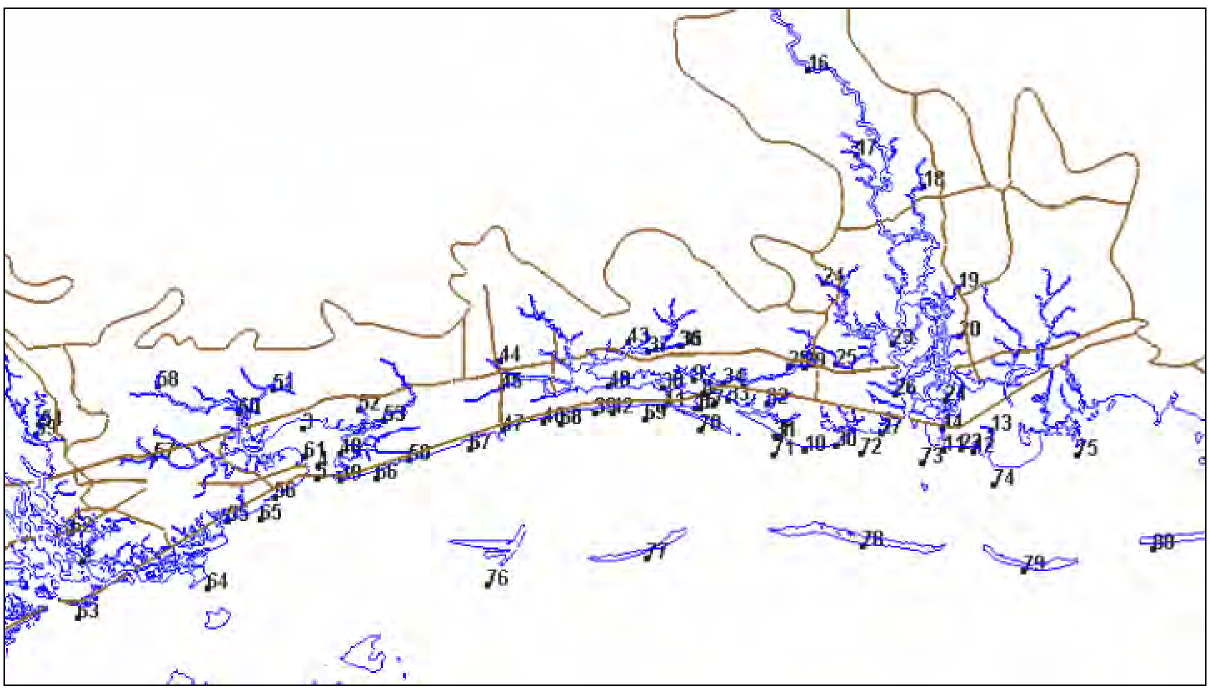


Figure 2.9-2. Save Station Locations

2.9.3 Results

2.9.3.1 Without-Project

The peak surge elevations, maximum wave heights, and peak wave periods were saved at stations along the Mississippi coast for the entire JPM-OS storm suite and used as input for the JPM analysis. The resulting frequency relationships are provided by save station in Tables 2.9-1 to 2.9-3 below for water level, wave height, and wave period, respectively. Note that there are no waves at some of the inland points.

Table 2.9-1.
Stage-Frequency Relationships – Without Project

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	8.1	11.3	14.2	19.1	20.8
2	10	13.8	16.3	20.1	21.3
3	10.8	14.9	17.9	23	24.6
4	10.4	14.4	17.3	22.3	23.9
5	10.4	14.4	17.3	22.4	24
6	8.9	12.4	15.4	20.3	22.1
7	8.8	12.3	15.4	20.3	22.1
8	9	12.6	15.6	20.6	22.4
9	9.2	12.9	16	21	22.9
10	8.1	11.2	13.9	18.7	20.4
11	7.7	11.2	13.8	18.3	19.8
12	7.4	10.9	13.6	17.9	19.4
13	7.6	11.3	14	18.3	19.7
14	7.1	9.9	12.1	16.3	17.8
15	9	12.5	15.5	20.3	22.1
16	2.6	3.3	3.8	4.5	4.8
17	3.2	4.2	4.8	6.1	6.5
18	4.2	5.5	6.6	8.5	9.3
19	5.6	7.4	8.9	11.8	13
20	5.9	7.9	9.6	12.9	14.2
21	6.3	8.6	10.6	14.5	15.9
22	7.7	11.2	13.9	18.2	19.8
23	5.8	7.7	9.3	12.4	13.6
24	5.7	7.5	9	12	13.5
25	6.1	8.1	9.8	12.6	13.7
26	6.3	8.4	10.2	13.8	15.1
27	8.1	11.4	13.9	18.7	20.3
28	7.1	9.7	11.6	15	16.3
29	6.9	9.3	11.2	14.4	15.5
30	8.2	11.6	14.2	19	20.7
31	8.1	11.3	14.2	19.1	20.8
32	8.4	11.8	14.8	19.8	21.6
33	8.8	12.3	15.4	20.4	22.3
34	8.7	12.2	15.2	20	21.8
35	8.3	11.3	13.5	17.2	18.5
36	8.3	11.3	13.5	17.2	18.5
37	8.7	11.8	14.1	17.9	19.2
38	9.1	12.6	15.5	20.1	21.8
39	9.7	13.5	16.4	21.2	23
40	9	12.1	14.4	18.3	19.7
41	9.1	12.8	15.7	20.5	22.3

Table 2.9-1.
Stage-Frequency Relationships – Without Project (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
42	9.5	13.3	16.2	21	22.8
43	8.9	12	14.3	18.3	19.6
44	9.1	12	14.1	17.7	18.8
45	8.8	11.6	13.7	17.2	18.3
46	10.2	13.9	16.7	21.6	23.4
47	10.7	14.2	17	22.1	23.9
48	10.4	14.3	17.2	22.2	23.8
49	10.5	14.3	17.3	22.6	24.3
50	10.8	14.3	17.2	22.6	24.4
51	10.5	14.4	17.1	21.6	23
52	10.4	14.3	17.3	22.4	24
53	10.3	14.1	17.1	22.2	23.8
54	6.8	9.7	11.6	14.6	15.6
55	10.9	15.1	18.1	22.9	24.3
56	10.7	14.8	17.8	22.8	24.4
57	8.9	12.1	14.3	18.2	19.4
58	10.1	13.7	16.3	20.4	21.6
59	6.9	9.9	11.8	14.9	16
60	10.6	14.5	17.2	21.6	23
61	10.6	14.5	17.5	22.4	24
62	9.6	13.5	16	20.1	21.4
63	10.1	14.1	16.5	20.3	21.5
64	10.3	14.2	16.9	21.2	22.5
65	10.6	14.7	17.6	22.5	24
66	10.6	14.2	17.1	22.4	24.2
67	10.6	14.1	16.8	22	23.8
68	10	13.7	16.5	21.4	23.1
69	9.1	12.7	15.7	20.4	22.2
70	8.5	12	14.9	19.6	21.3
71	8	11.1	13.8	18.5	20.2
72	7.9	11.2	13.7	18.4	20
73	7.5	10.8	13.3	17.7	19.2
74	7	10.3	13.2	17.4	18.8
75	6.9	10.3	14	18.7	20.2
76	9.4	12.2	14.3	17.8	19.1
77	8	11	13.2	16.8	18
78	6.7	9.6	12.1	15.9	17.3
79	5.8	8.7	11.7	15.7	17
80	5.6	8.3	11.7	16.1	17.3

Table 2.9-2.
Wave Height-Frequency Relationships – Without Project

Station Number	Significant Wave Height (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	3.5	4.9	5.9	7.3	7.7
2	1.3	2.7	3.8	6	6.8
3	2.4	3.3	4	5.5	6
4	2	3.4	4.6	7.3	8.1
5	2.8	4.2	5.6	8	8.7
6	2.9	4.6	5.9	8.1	8.8
7	3.4	5.2	6.7	9.1	9.9
8	2.6	4.4	5.7	8	8.7
9	1.8	3.1	4	5.5	6
10	4.4	5.6	6.6	8	8.3
11	2.1	3.2	4.1	6.1	6.5
12	5	6.4	7.5	9.5	10
13	0.2	0.5	0.9	1.3	1.5
14	0.4	0.8	1.3	2.2	2.5
15	2.2	4.1	5.3	7.3	8
16	0	0	0	0	0
17	0	0	0	0	0
18	0	0	0	0	0
19	0	0	0	0	0
20	0.1	0.2	0.3	0.6	0.7
21	0.6	1.3	1.9	3.1	3.3
22	4.3	6	7.3	9.5	10.2
23	0.5	1	1.4	2.2	2.6
24	0	0	0	0	0
25	0	0	0	0	0
26	0.8	1.6	2.3	3.4	3.7
27	3.3	4.4	5.6	7.2	7.6
28	0	0	0	0	0
29	0	0	0	0	0
30	4.4	5.8	6.7	8.3	8.9
31	3.8	5.1	6.1	7.4	7.8
32	0.3	0.8	1.3	2.3	2.7
33	2.7	4.4	5.8	7.8	8.5
34	0	0.1	0.1	0.2	0.3
35	0	0	0	0	0
36	0	0	0	0	0
37	0	0	0	0	0
38	0.8	1.2	1.6	2.3	2.7
39	5.1	6.2	7	8.1	8.6
40	1	1.4	1.8	2.6	2.9
41	4.6	6.1	7.2	8.9	9.4
42	5.5	6.7	7.4	8.6	9.1
43	0	0	0	0	0
44	0	0	0	0	0

Table 2.9-2.
Wave Height-Frequency Relationships – Without Project (continued)

Station Number	Significant Wave Height (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
45	0	0	0	0	0
46	4.5	5.5	6.4	8	8.5
47	4.4	5.8	6.5	7.8	8.4
48	0.5	1.1	1.8	4	4.7
49	3.1	4.4	5.6	8.2	8.9
50	0.5	1.8	3.9	7.3	8.5
51	0	0	0	0	0
52	0	0.1	0.1	0.2	0.3
53	0.1	0.2	0.3	0.6	0.7
54	0	0	0	0	0
55	2.7	4	5.3	7.4	8.1
56	3	4.4	5.8	8.4	9.1
57	0	0	0	0	0
58	0	0	0	0	0
59	0	0	0	0	0
60	0.6	1.2	1.8	2.6	3
61	2	3.1	4	5.7	6.3
62	0.5	1.2	1.8	3.8	4.5
63	2.5	4.1	5.5	8.1	9.2
64	3	4.2	5.2	7.6	8.4
65	3.7	5.1	6.3	9	9.8
66	4.2	5.1	6.1	8.4	9.2
67	5.1	6.1	6.7	8.1	8.7
68	4.4	5.5	6.4	8	8.4
69	6.3	7.6	8.4	9.8	10.2
70	5.5	6.9	8	9.6	10.2
71	5	6.1	7	8.3	8.7
72	5.2	6.6	7.7	9.8	10.5
73	4.8	6	6.8	8.3	8.8
74	4.7	6	6.8	8.1	8.4
75	6.4	7.7	9.6	11.3	11.7
76	8.3	9.9	11	13	13.6
77	10.6	12.4	13.6	15.5	16.1
78	10.4	12.2	13.6	16	17
79	11.9	13.6	15.1	17	17.5
80	10	11.5	13.3	15.4	15.9

Table 2.9-3.
Wave Period-Frequency Relationships

Station Number	Peak Wave Period (sec)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	8.4	11.6	13.9	16.3	16.3
2	3.4	6.3	7.9	12.6	14.4
3	3.8	5.1	6.7	9.9	11.1
4	5.5	7.0	8.0	10.2	11.0
5	7.0	9.2	11.1	14.6	14.9
6	6.2	10.1	13.0	15.9	16.3
7	9.5	12.7	14.2	16.3	16.3
8	5.9	8.9	12.1	14.7	15.6
9	6.0	8.3	11.7	14.8	15.6
10	9.5	12.6	14.3	16.3	16.3
11	10.7	13.1	14.9	16.3	16.3
12	11.7	13.0	14.0	14.9	14.9
13	2.3	2.6	3.0	3.5	3.8
14	4.4	5.2	5.8	7.0	7.4
15	5.3	7.5	9.8	13.6	14.6
16	2.2	2.4	2.4	2.4	2.4
17	2.2	2.4	2.4	2.4	2.4
18	2.2	2.4	2.4	2.4	2.4
19	2.2	2.4	2.4	2.4	2.4
20	2.2	2.4	2.6	2.9	2.9
21	2.4	2.8	3.2	4.1	4.5
22	12.1	13.2	14.1	14.9	14.9
23	3.0	3.5	3.9	4.3	4.3
24	2.2	2.4	2.4	2.4	2.4
25	2.2	2.4	2.4	2.4	2.4
26	2.4	2.9	3.3	3.9	4.2
27	8.5	11.6	14.0	16.3	16.3
28	2.2	2.4	2.4	2.4	2.4
29	2.2	2.4	2.4	2.4	2.4
30	5.2	7.1	10.7	14.9	14.9
31	10.4	12.3	13.6	16.1	16.3
32	2.6	3.4	4.3	8.5	11.2
33	10.2	12.7	13.9	16.3	16.3
34	2.2	2.4	2.6	2.9	2.9
35	2.2	2.4	2.4	2.4	2.4
36	2.2	2.4	2.4	2.4	2.4
37	2.2	2.4	2.4	2.4	2.4
38	2.7	3.2	3.7	4.4	4.7
39	11.4	12.8	13.6	15.1	15.6
40	2.5	3.0	3.3	4.0	4.1
41	11.3	12.6	13.5	14.9	15.2
42	11.4	12.8	13.7	15.5	16.0
43	2.2	2.4	2.4	2.4	2.4
44	2.2	2.4	2.4	2.4	2.4

Table 2.9-3.
Wave Period-Frequency Relationships (continued)

Station Number	Peak Wave Period (sec)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
45	2.2	2.4	2.4	2.4	2.4
46	11.4	13.0	14.1	16.3	16.3
47	10.2	12.7	13.9	15.8	16.3
48	2.6	3.4	4.4	6.2	6.8
49	7.1	9.6	11.3	15.5	16.3
50	2.8	6.5	10.3	13.5	13.5
51	2.2	2.4	2.4	2.4	2.4
52	2.4	2.8	3.2	3.8	3.9
53	2.2	2.4	2.6	2.7	2.7
54	0.0	0.0	0.0	0.0	0.0
55	8.0	10.8	12.4	15.3	16.3
56	7.0	8.9	10.3	12.7	13.8
57	2.2	2.4	2.6	2.7	2.7
58	2.2	2.4	2.4	2.4	2.4
59	0.0	0.0	0.0	0.0	0.0
60	2.3	2.8	3.2	3.9	4.2
61	3.7	6.0	7.7	10.8	12.2
62	2.6	3.6	4.9	7.5	8.3
63	5.4	6.7	7.7	9.3	9.9
64	7.0	8.6	10.0	12.0	12.7
65	6.1	7.7	9.0	10.8	11.4
66	6.1	9.7	12.4	14.9	14.9
67	10.7	13.7	14.9	14.9	14.9
68	11.3	12.9	13.9	14.9	14.9
69	11.5	13.2	14.5	16.3	16.3
70	11.3	12.9	14.0	14.9	14.9
71	5.3	8.8	13.1	16.2	16.3
72	5.7	8.9	12.6	14.9	14.9
73	11.6	13.3	14.3	14.9	14.9
74	7.2	10.6	12.9	16.3	16.3
75	12.1	13.9	14.9	14.9	14.9
76	12.4	14.1	15.1	16.3	16.3
77	12.0	13.3	14.3	14.9	14.9
78	12.4	13.8	14.6	14.9	14.9
79	12.7	14.0	14.7	14.9	14.9
80	12.4	13.8	14.6	14.9	14.9

2.9.3.2 Line of Defense 3

The peak water level, maximum wave height, and wave period for the set of storms run with the lines of defense in place were also saved at stations along the coast. The water level at save stations adjacent to the proposed structures was increased by the amount predicted from the Boussinesq modeling. The waves were not calculated for stations behind the proposed line of defense. The frequency relationships were estimated from the 27 storm subset and methodology discussed in section 2.4.3. The wave periods were not altered by the presence of the line of defense and thus are the same as for the no project condition. The frequency relationships for water level and wave height are provided by save station in Tables 2.9-4 and 2.9-5, respectively.

Table 2.9-4.
Stage-Frequency Relationships – LOD 3

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	8.5	12	15	19.8	21.6
2	10.2	14.3	16.8	21	22.3
3	1.3	1.3	1.5	2.5	2.6
4	1.2	1.2	1.4	1.8	2
5	11.9	15.5	18.3	23.9	25.5
6	9.2	13.2	16.4	21.5	23.4
7	9	12.8	16	21	22.9
8	9.3	13.2	16.3	21.5	23.4
9	1.3	1.3	1.4	1.4	1.5
10	8.1	11.4	14.3	18.9	20.7
11	8.1	11.5	14.3	18.9	20.4
12	8	11.1	14.3	18.5	20
13	8.2	12	14.9	19	20.4
14	7.1	9.9	12.1	16.5	18
15	9.3	13.3	16.4	21.4	23.3
16	2.6	3.2	3.8	4.5	4.8
17	3	4.3	4.9	6.2	6.6
18	4.2	5.6	6.7	8.5	9.3
19	5.6	7.6	9.1	12	13.1
20	6	7.9	9.8	13.1	14.3
21	6.6	9	11.1	14.9	16.3
22	8.2	11.8	14.6	18.8	20.4
23	6	7.9	9.5	12.6	13.7
24	5.8	7.6	9.1	12.1	13.6
25	4.2	5.4	6.5	8	8.1
26	6.5	8.7	10.6	14.1	15.4
27	8.4	11.9	14.6	19.2	20.8
28	1.6	1.7	2	2.3	2.4
29	1.6	1.8	2.2	2.9	2.9
30	8.6	12.2	15	19.6	21.4
31	8.5	12	15	19.8	21.6
32	8.7	12.1	15.3	20.1	22
33	9.1	12.9	16.1	21.2	23.2
34	1.3	1.3	1.4	1.4	1.5
35	1.2	1.9	2	2.2	2.4
36	1.2	1.9	2	2.2	2.4
37	1.3	2	2.1	2.5	2.7
38	1.1	1.3	1.3	1.4	1.4
39	10	14	16.8	21.8	23.7

Table 2.9-4.
Stage-Frequency Relationships – LOD 3 (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
40	1.3	1.6	1.6	1.9	2
41	9.5	13.4	16.3	21.2	23.1
42	9.8	13.8	16.6	21.6	23.5
43	1.5	2.2	2.3	2.9	3.1
44	3	4.5	4.8	6.3	6.4
45	2.3	3.1	3.5	4.7	4.7
46	10.5	14.3	16.9	22.4	24.1
47	11	14.6	17.4	22.9	24.6
48	1.4	1.5	1.6	1.8	2
49	10.9	15.2	18.2	23.8	25.5
50	11	14.5	17.7	23.6	25.6
51	2.2	2.6	3.1	4.6	4.8
52	1.9	2	2.1	2.2	2.3
53	0	0	0.1	0.1	0.1
54	6.9	9.9	11.8	14.9	15.9
55	11.3	15.9	19.2	24.2	25.8
56	11.1	15.7	18.7	24.1	25.7
57	2.1	2.5	2.6	3.5	3.7
58	2.2	2.4	3.4	4.7	4.8
59	7	10.1	12.1	15.3	16.4
60	2.4	3.2	4	5.6	5.8
61	1.1	1.2	1.5	1.8	2
62	0	0	0	0	0
63	10.4	14.5	17.1	21.1	22.3
64	10.6	14.8	17.7	22.4	23.7
65	11	15.4	18.6	23.6	25.1
66	10.9	14.6	17.7	23.2	25
67	10.7	14.3	17.1	22.7	24.3
68	10.1	14	16.6	22	23.6
69	9.2	13	16.1	20.8	22.7
70	8.6	12.3	15.3	20	21.8
71	8	11.4	14.2	18.8	20.6
72	8	11.5	14	18.7	20.3
73	7.5	10.9	13.4	17.9	19.4
74	7	10.4	13.2	17.5	18.9
75	6.9	10.3	14	18.7	20.2
76	9.5	12.2	14.7	18.7	19.8
77	8	11.2	13.3	16.9	18.2
78	6.7	9.6	12.1	16	17.6
79	5.8	8.7	11.7	15.7	17
80	5.6	8.3	11.7	16.1	17.3

Table 2.9-5.
Wave Height-Frequency Relationships – LOD 3

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	3.6	4.9	6.1	7.8	8.3
2	1.4	2.9	4.0	6.2	7.0
5	2.8	4.4	5.9	8.4	9.1
6	3.0	4.8	6.2	8.4	9.1
7	3.5	5.2	6.9	9.7	10.7
8	2.8	4.5	5.9	8.4	9.2
10	4.6	5.7	6.7	8.2	8.6
11	2.1	3.2	4.2	6.4	6.9
12	5.0	6.4	7.5	9.4	9.9
13	0.2	0.5	0.8	1.0	1.1
14	0.4	0.7	1.1	1.8	2.0
15	2.5	4.0	5.6	7.6	8.3
16	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0
18	0.0	0.0	0.0	0.0	0.0
19	0.0	0.0	0.0	0.0	0.0
20	0.1	0.2	0.3	0.6	0.7
21	0.1	0.8	0.9	0.9	0.5
22	4.3	6.1	7.4	9.6	10.3
23	0.4	0.8	1.1	1.9	2.2
24	0.0	0.0	0.0	0.0	0.0
25	0.0	0.0	0.0	0.0	0.0
26	0.8	1.6	2.4	3.5	3.8
27	4.0	5.1	6.1	7.2	7.4
28	0.0	0.0	0.0	0.0	0.0
29	0.0	0.0	0.0	0.0	0.0
30	4.4	6.0	6.9	8.4	9.0
31	3.9	5.1	6.2	7.6	7.9
32	0.2	0.5	1.0	1.8	2.2
33	2.8	4.5	5.9	7.9	8.6
39	5.1	6.3	7.1	8.2	8.7
41	4.5	5.9	6.2	6.0	5.7
42	5.5	6.8	7.5	8.7	9.2
46	4.6	5.7	6.6	8.2	8.7
47	4.5	6.0	6.6	8.0	8.6
49	3.0	4.6	5.2	8.4	9.3
50	3.0	3.4	4.9	7.7	8.6
54	0.0	0.0	0.0	0.0	0.0
55	2.7	4.2	5.5	7.6	8.3
56	3.1	4.6	6.0	8.7	9.4
59	0.0	0.0	0.0	0.0	0.0
63	2.6	4.1	5.5	7.9	9.0
64	3.0	4.2	5.3	7.8	8.6
65	3.7	5.2	6.5	9.2	10.1

Table 2.9-5.
Wave Height-Frequency Relationships – LOD 3 (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
66	4.3	5.2	6.2	8.6	9.4
67	5.1	6.3	6.8	8.2	8.8
68	4.4	5.6	6.5	8.1	8.5
69	6.3	7.7	8.3	9.4	9.6
70	5.5	7.0	8.1	9.8	10.4
71	5.1	6.0	7.2	8.4	8.7
72	5.2	6.6	7.8	10.0	10.8
73	4.8	5.9	7.0	9.1	9.9
74	4.8	6.0	6.7	7.7	7.9
75	6.5	7.8	9.7	11.4	11.8
76	8.3	9.9	10.9	12.7	13.2
77	10.7	12.4	13.6	15.5	16.1
78	10.2	12.2	13.3	15.0	15.7
79	11.9	13.6	15.1	16.8	17.2
80	10.0	11.4	13.3	15.7	16.4

2.9.3.3 Line of Defense 4

The frequency relationships for line of defense 4 were also estimated from the 27 storm subset and methodology discussed in section 2.4.3. The waves were not calculated for stations behind the proposed line of defense. The wave periods were not altered by the presence of the line of defense and thus are the same as for the no project condition. The frequency relationships for water level and wave height are provided by save station in Tables 2.9-6 and 2.9-7, respectively.

Table 2.9-6.
Stage-Frequency Relationships – LOD 4

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	8.2	11.6	14.6	19.4	21.1
2	10.2	14.2	16.8	20.7	22
3	1.2	1.2	1.4	2.3	2.4
4	1.2	1.2	1.3	1.3	1.4
5	11.9	15.5	18.2	23.8	25.4
6	9.1	13.1	16.4	21.5	23.4
7	9	12.8	16	21	22.9
8	9.3	13.2	16.3	21.5	23.4
9	1.2	1.2	1.3	1.3	1.4
10	8.1	11.4	14.3	18.9	20.7
11	7.7	11.2	13.9	18.5	19.7
12	7.5	10.9	13.7	18	19.3
13	7.6	11.4	14	18.4	19.8
14	7.1	9.9	12.1	16.5	18
15	9.2	13.2	16.4	21.4	23.3

Table 2.9-6.
Stage-Frequency Relationships – LOD 4 (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
16	2.6	3.2	3.8	4.5	4.8
17	3.2	4.2	4.8	6.1	6.5
18	4.2	5.5	6.6	8.5	9.3
19	5.6	7.4	9	11.8	13.1
20	5.9	7.9	9.7	13	14.3
21	6.4	8.7	10.8	14.6	16
22	7.7	11.2	14	18.3	19.9
23	5.8	7.7	9.4	12.5	13.7
24	5.7	7.5	9	12	13.5
25	5.2	6.6	8	9.7	9.8
26	6.3	8.5	10.4	13.9	15.2
27	8.1	11.4	14.2	18.9	20.5
28	1	1	1.1	1.1	1.1
29	0.8	0.8	0.8	0.9	0.9
30	8.2	11.7	14.6	19.2	20.9
31	8.2	11.6	14.6	19.4	21.1
32	8.7	12.2	15.3	20.2	22.1
33	9.1	13	16.3	21.3	23.3
34	1.2	1.2	1.2	1.3	1.3
35	1.2	1.9	2	2.2	2.4
36	1.2	1.9	2	2.2	2.4
37	1.3	2	2.1	2.5	2.7
38	1	1.2	1.2	1.2	1.2
39	10	14	16.9	22	23.9
40	1.3	1.6	1.6	1.8	1.9
41	9.5	13.6	16.6	21.3	23.3
42	9.8	13.9	16.9	21.9	23.8
43	1.5	2.4	2.5	2.9	3.1
44	3	4.4	4.8	6.4	6.4
45	2.3	3.1	3.5	4.6	4.7
46	10.5	14.4	17	22.7	24.4
47	11	14.7	17.6	23.2	24.9
48	1	1.4	1.4	1.4	1.4
49	10.9	15.3	18.2	23.9	25.7
50	11.3	14.7	18.1	24.4	26.2
51	2.2	2.6	3.1	4.6	4.8
52	1.5	1.5	1.5	1.6	1.7
53	0	0	0.1	0.1	0.1
54	6.9	9.9	11.8	14.9	15.9
55	11.3	16	19.2	24.4	25.9
56	11.1	15.8	18.8	24.3	25.9
57	2.1	3	3.2	3.5	3.7
58	2.2	2.4	3.4	4.7	4.8
59	7	10.1	12.1	15.3	16.4

Table 2.9-6.
Stage-Frequency Relationships – LOD 4 (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
60	2.4	3.3	4	5.6	5.8
61	1.3	1.3	1.4	1.4	1.4
62	9.7	13.8	16.4	20.7	22.1
63	10.4	14.5	17	21	22.3
64	10.6	14.7	17.6	22.3	23.6
65	10.9	15.4	18.5	23.6	25.1
66	10.8	14.6	17.6	23.2	25
67	10.8	14.3	17.1	22.7	24.3
68	10.1	14	16.6	22	23.6
69	9.2	13	16.1	20.8	22.7
70	8.6	12.3	15.3	20	21.8
71	8	11.4	14.2	18.8	20.5
72	8	11.5	14	18.6	20.2
73	7.5	10.9	13.4	17.9	19.4
74	7	10.4	13.2	17.5	18.9
75	6.9	10.3	14	18.7	20.2
76	9.5	12.2	14.7	18.6	19.8
77	8	11.2	13.3	16.9	18.2
78	6.7	9.6	12.1	15.9	17.1
79	5.8	8.7	11.7	15.7	17
80	5.6	8.3	11.7	16.1	17.3

Table 2.9-7.
Wave Height-Frequency Relationships – LOD 4

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
1	3.6	4.9	6.1	8	8.6
2	1.3	2.9	4	6.3	7.1
5	2.8	4.3	5.7	8.1	9
6	3	4.8	6.2	8.7	9.5
7	3.4	5.2	6.9	9.7	10.7
8	2.8	4.5	6.2	9.4	10.5
10	4.6	5.7	6.8	8.5	9
11	2.1	3.2	4.2	6.4	6.9
12	4.8	6.2	7.2	9.1	9.6
13	0.1	0.3	0.5	0.8	1
14	0.2	0.7	1.1	1.8	2
15	2.4	4	5.5	7.8	8.6
16	0	0	0	0	0
17	0	0	0	0	0
18	0	0	0	0	0
19	0	0	0	0	0
20	0.1	0.2	0.3	0.6	0.7

Table 2.9-7.
Wave Height-Frequency Relationships – LOD 4 (continued)

Station Number	Water Level (ft)				
	25-yr	50-yr	100-yr	500-yr	1000-yr
21	0.3	1	1.6	2.8	3
22	4.3	6.1	7.4	9.6	10.3
23	0.3	1	1.3	1.7	2
24	0	0	0	0	0
25	0	0	0	0	0
26	0.8	1.6	2.4	3.5	3.8
27	3.8	5	6	7.1	7.2
28	0	0	0	0	0
29	0	0	0	0	0
30	4.4	6	6.8	8.2	8.7
31	3.9	5.1	6.3	7.7	8.1
32	0.2	0.4	0.7	1	1.9
33	2.9	4.5	6	8.2	9
39	5.1	6.3	7.1	8.3	8.8
41	5.1	7.1	7.5	7.5	7.3
42	5.5	6.8	7.5	8.7	9.2
46	4.5	5.5	6.6	8.2	8.7
47	4.5	6	6.3	8	8.6
49	3.1	4.6	5.7	8.4	9.2
50	0.5	1.9	4.1	7.6	8.9
54	0	0	0	0	0
55	2.7	4.2	5.5	7.5	8.1
56	3	4.5	6	8.6	9.4
59	0	0	0	0	0
62	0.5	1.3	1.9	4	4.7
63	2.6	4.2	5.6	8.1	9.2
64	3	4.2	5.3	7.8	8.6
65	3.7	5.1	6.4	9.2	10.1
66	4.4	5.1	6.1	8.5	9.3
67	5.1	6.3	6.8	8.2	8.8
68	4.4	5.6	6.6	8.3	8.8
69	6.4	7.7	8.4	9.7	10.1
70	5.5	7	8.1	9.6	10.2
71	5.1	6	7.3	8.7	9.2
72	5.2	6.6	7.8	10.1	10.9
73	4.8	5.9	6.7	8.2	8.7
74	4.8	6.1	6.8	7.9	8.2
75	6.5	7.8	9.7	11.4	11.8
76	8.3	9.9	10.9	12.7	13.2
77	10.7	12.3	13.7	16.1	16.9
78	10.3	12.2	13.3	15	15.7
79	11.9	13.6	15.1	17	17.5
80	10	11.3	13.2	15.5	16.1

2.10 Barrier Island Sensitivity

Topography, landscape features, and vegetation 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) even when below the surge elevation have the potential to create friction and slow the forward speed of the storm surge. The barrier islands serve as the first line of defense for the Mississippi coast. The purpose of this section is to document a sensitivity study of various barrier island configurations to qualitatively assess the impact of barrier island restoration on storm surge at the mainland coast for storms of varying intensities.

The barrier island sensitivity study was conducted on a grid consistent with that applied for the Interagency Performance Evaluation Team (IPET) study. The analysis provides valuable information on trends and relative performance but one should be cautious about making quantitative assessments of surge reduction. It should be noted that the analysis does not consider the morphologic changes to the barrier islands caused by erosion that occur during a storms passage. The analysis also does not consider changes in the structure of the hurricane itself due to landfall infilling phenomenon that may be influenced by landscape features such as barrier islands.

2.10.1 Storm Suite

Eleven storms were identified for evaluating storm surge response to changes in barrier island configuration: two historical storms and nine hypothetical storms (Table 2.10-1). The two historical storms, Camille and Katrina, were selected because those hurricanes did in fact make landfall on the Mississippi coast in 1969 and 2005, respectively. A suite of storms making landfall on the Mississippi coast were also designed and selected for simulation. The first hypothetical storm (HST001) was designed to produce a 22 ft surge potential seaward of the barrier islands on the Mississippi coast. The storm had a central pressure of 890 mb and a radius to maximum winds of approximately 11 nm, that of Hurricane Camille. Two additional storms were defined by scaling HST001 to produce storms with a surge potential of 13 ft seaward of the barrier islands (HST002) and a surge potential of 8 ft (HST003). The hypothetical storms followed the Katrina track (both geographically and temporally), but were shifted eastward to make landfall at the three locations shown in Figure 2.10-1.

Table 2.10-1.
Barrier Island Sensitivity Storm Suite

Storm Number	Storm Name	Track	Barrier Island Configuration
1	Katrina	Historical	Post-Katrina
			Restored-High
2	Camille	Historical	Post-Katrina
			Restored-High
3	HST001-04	04	Post-Katrina
			Restored-High
			Restored-Low
4	HST003-04	04	Post-Katrina
			Restored-High
5	HST001-06	06	Post-Katrina
			Restored-High
			Restored-Low
6	HST003-06	06	Post-Katrina
			Restored-High
7	HST002-04	04	Post-Katrina
			Restored-High

Table 2.10-1.
Barrier Island Sensitivity Storm Suite (continued)

Storm Number	Storm Name	Track	Barrier Island Configuration
8	HST002-06	06	Post-Katrina
			Restored-High
9	HST001-05	05	Post-Katrina
			Restored-High
			Restored-Low
10	HST002-05	05	Post-Katrina
			Restored-High
11	HST003-05	05	Post-Katrina
			Restored-High

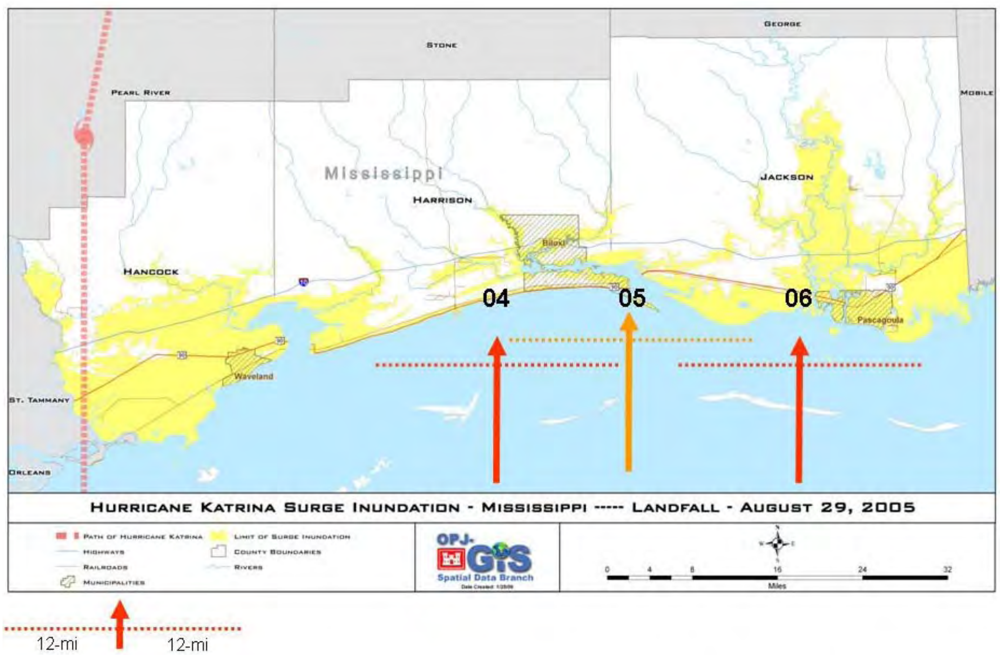


Figure 2.10-1. Hypothetical Storm Tracks

2.10.2 Barrier Island Configuration

The sensitivity storm suite consisting of the eleven storms described in Section 2.10.1 was used to simulate storm surge on three barrier island configurations. The barrier island configurations modeled were: 1) the existing Post-Katrina degraded condition (elevations ranging from approximately 2 to 6 ft (NAVD88 2004.65)); 2) a Restored-Hight barrier island configuration with an extended (pre-Camille) footprint and an elevation of 20 ft NAVD88 2004.65; and 3) a Restored-Low configuration with a footprint representative of the islands pre-Katrina and elevations ranging from approximately 5 to 10 ft (NAVD88 2004.65). The Restored-High configuration represents a massive barrier island configuration that would be difficult to achieve and was modeled for sensitivity purposes. The Restored-Low is a more likely restoration scenario with pre-Katrina footprints and heights of 10 ft or less. Figures 2.10-2 shows the topography of each of the five Mississippi barrier islands for the Post-Katrina and the Restored condition. Note that for the Restored-High condition, the gaps in Ship Island and Dauphin Island have been repaired to the pre-Camille configuration. Bathymetry for the Post-Katrina degraded condition was derived from a SHOALS air-borne LIDAR survey taken in September/October 2005. Figure 2.10-3 shows the difference between the Restored-Low and Post-Katrina conditions.

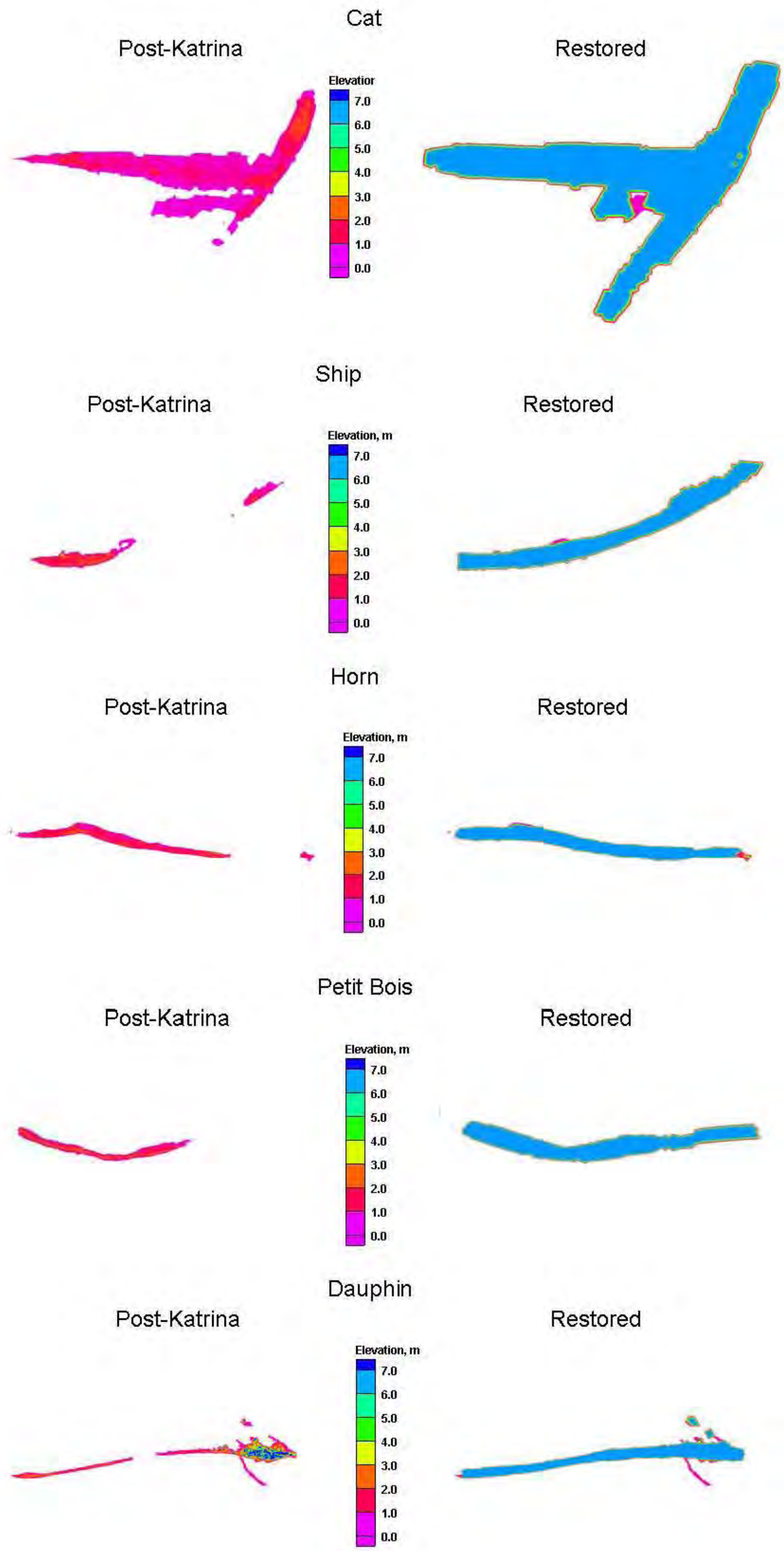


Figure 2.10-2. Mississippi barrier island Post-Katrina and Restored-High configurations

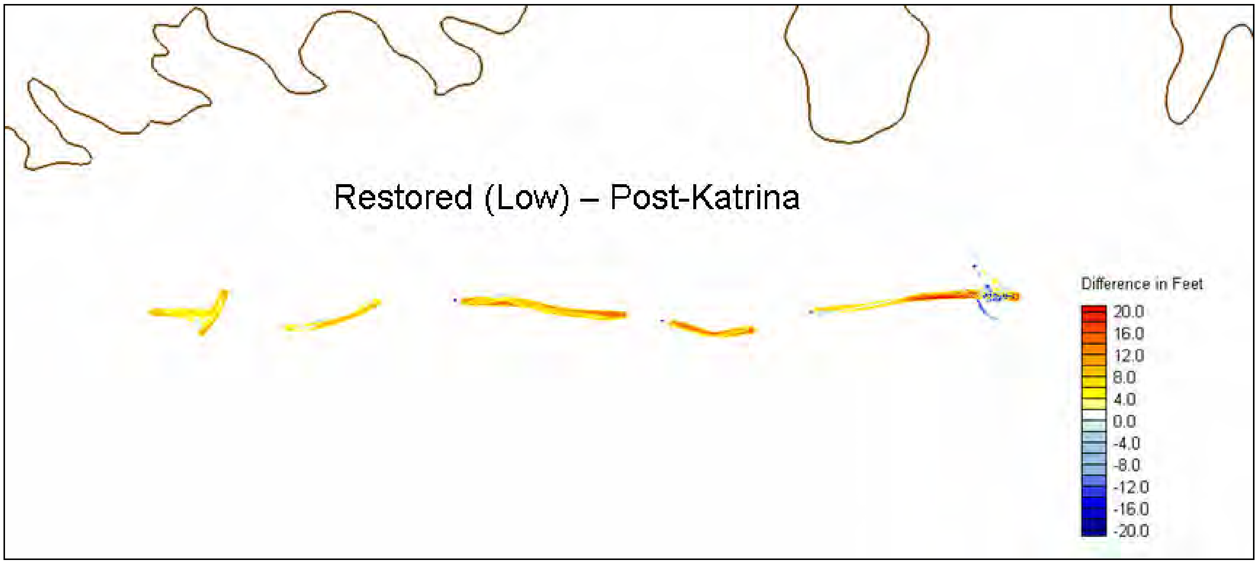


Figure 2.10-3. Difference between Restored-Low and Post-Katrina Mississippi barrier island configurations

The entire eleven storm suite was simulated on the Post-Katrina and Restored-High grids. Storms HST001-04, HST001-05, and HST001-06 were also simulated on the Restored-Low grid (see Table 2.10-1).

2.10.3 Results

Peak water level maps for each of the 11 storms simulated on the existing Post-Katrina configuration were compared to the same storms simulated on the Restored-High barrier island configuration; and the three storms simulated on the Restored-Low configuration were also compared to Post-Katrina. In general, raising the barrier islands caused a decrease in peak water level landward of the barrier islands when compared to the peak water level for the baseline Post-Katrina configuration and an increase in peak water level seaward of the barrier islands. Table 2.10-2 shows the peak water levels for each simulation with Post-Katrina and the Restored-High barrier island configurations. Reduction in the peak water level landward of the barrier islands is as much as 10 ft.

Table 2.10-2.
Peak Water Level for Barrier Island Sensitivity Storms

		Peak Water Level, ft					
		Waveland		Biloxi		Pascagoula	
Storm Name	Track	Post-Katrina	Restored	Post-Katrina	Restored	Post-Katrina	Restored
Katrina	Historical	26-28	26-28	20	18	16	12
Camille	Historical	28	26	22	20	12	10
HST001-04	04	8-12	8-10	40	35	28	18
HST003-04	04	6	6	14	13	6	6
HST001-06	06	3	3	8	8	40	31
HST003-06	06	2	2	6	6	12	6
HST002-04	04	8	6	24	20	14	9
HST002-06	06	2	2	8	6	16	14
HST001-05	05	3	3	24	22	32-33	26-27
HST002-05	05	3	3	18	14	16	11
HST003-05	05	2	2	12	10	7	6

For the purposes of discussion and comparison, Figures 2.10-4 through 2.10-12 show peak water levels for simulations of Hurricane Katrina, HST001-05, HST002-05, HST003-05 storms for both the Post-Katrina and Restored-High barrier island configurations and HST001-05 for the Restored-Low configuration. Peak water levels for Hurricane Katrina show maximum water levels of approximately 26-28 ft for the Waveland area for both the Post-Katrina and Restored-High barrier island configurations (Figures 2.10-4 and 2.10-5). This area of maximum water level is west of all barrier islands that protect the Mississippi coast, therefore little change is observed in this region when the barrier islands are raised. Peak water levels near Biloxi are approximately 20 ft for Post-Katrina and 18 ft for the Restored condition. This area is afforded 1-2 ft of surge protection from the raised Ship and Horn Islands. Further to the east, water levels in Pascagoula are reduced from 12-16 ft to 10-12 ft with the presence of the raised Horn, Petit Bois, and Dauphin Islands. Note that the barrier islands are completely inundated for the Post-Katrina configuration and remain dry for the Restored-High barrier island configuration.

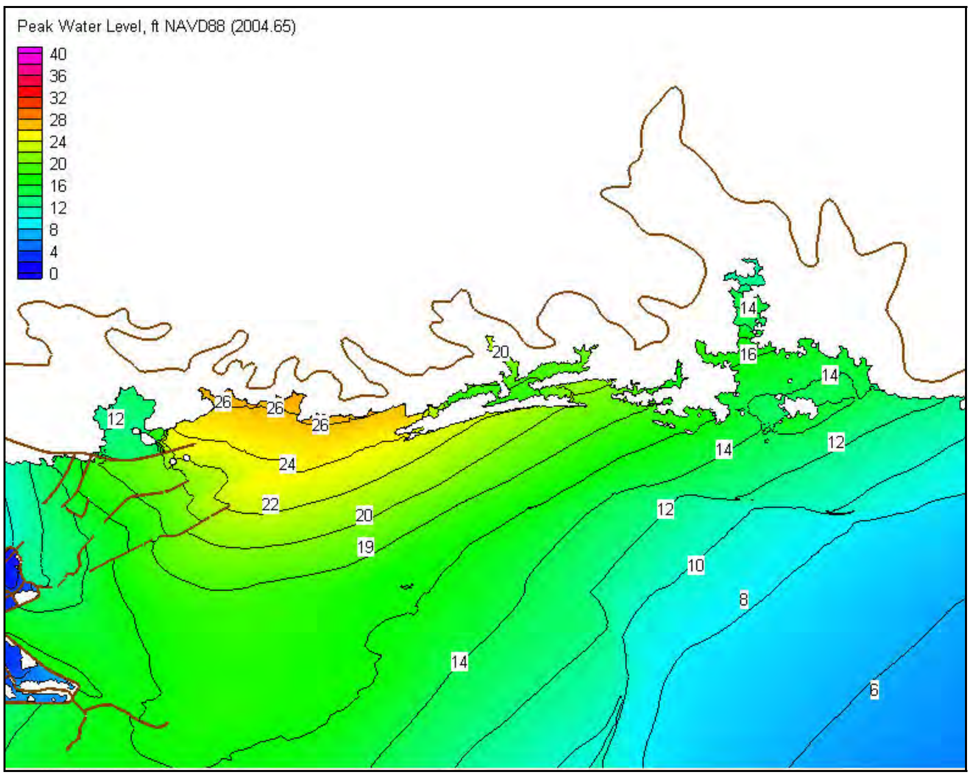


Figure 2.10-4. Katrina peak storm surge with waves; Post-Katrina configuration

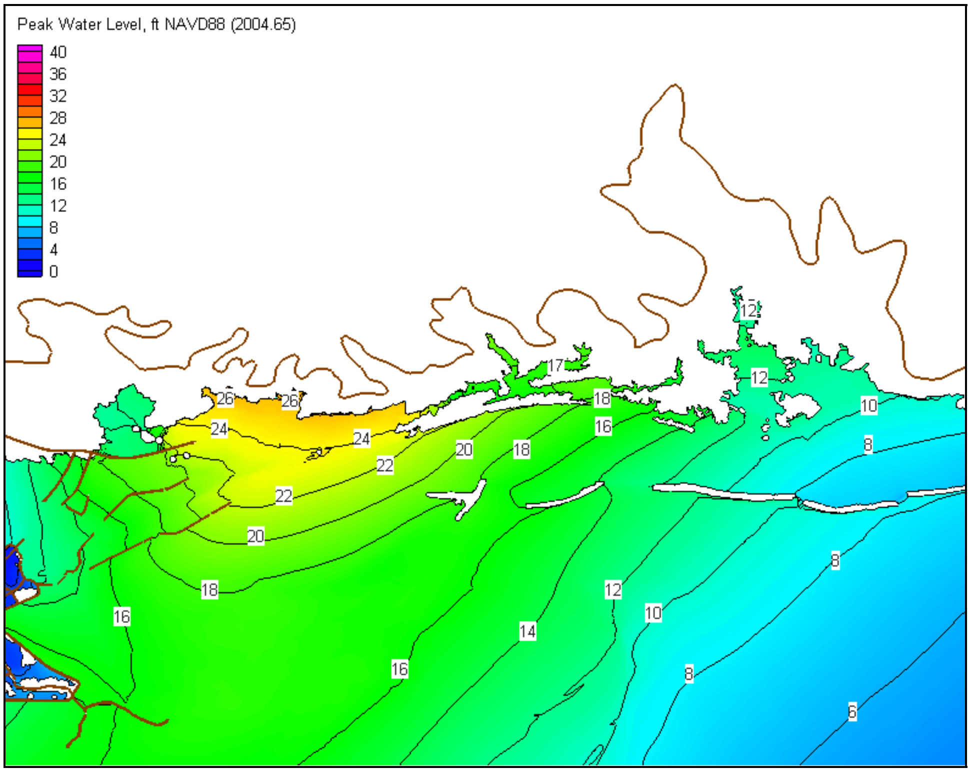


Figure 2.10-5. Katrina peak storm surge with waves; Restored-High configuration

Peak water levels for the HST001-05 storm are higher than the Hurricane Katrina water levels, with the greatest inundation levels in Biloxi and Pascagoula (Figures 2.10-6 to 2.10-8). Post-Katrina peak water levels are 32-33 ft and extend well into the Pascagoula basin with water levels of 24-26 ft. Peak water levels are a maximum of 26-27 ft for the Restored-High barrier island configuration and are 28-30 ft for the Restored-Low. The water levels up the Pascagoula basin reach 18 ft for the Restored-High barrier island configuration and reach 22 ft for the Restored-Low. Water levels at the entrance to Biloxi Bay are 22-24 ft for Post-Katrina, 20-22 ft with the Restored-High barrier islands, and 22-24 ft for the Restored-Low. Water levels seaward of the raised barrier islands are elevated compared to the Post-Katrina barrier islands. That is, the raised barrier islands effectively block some of the surge and it piles up seaward of the islands. Water levels near St. Louis Bay are nearly identical with and without raised barrier islands.

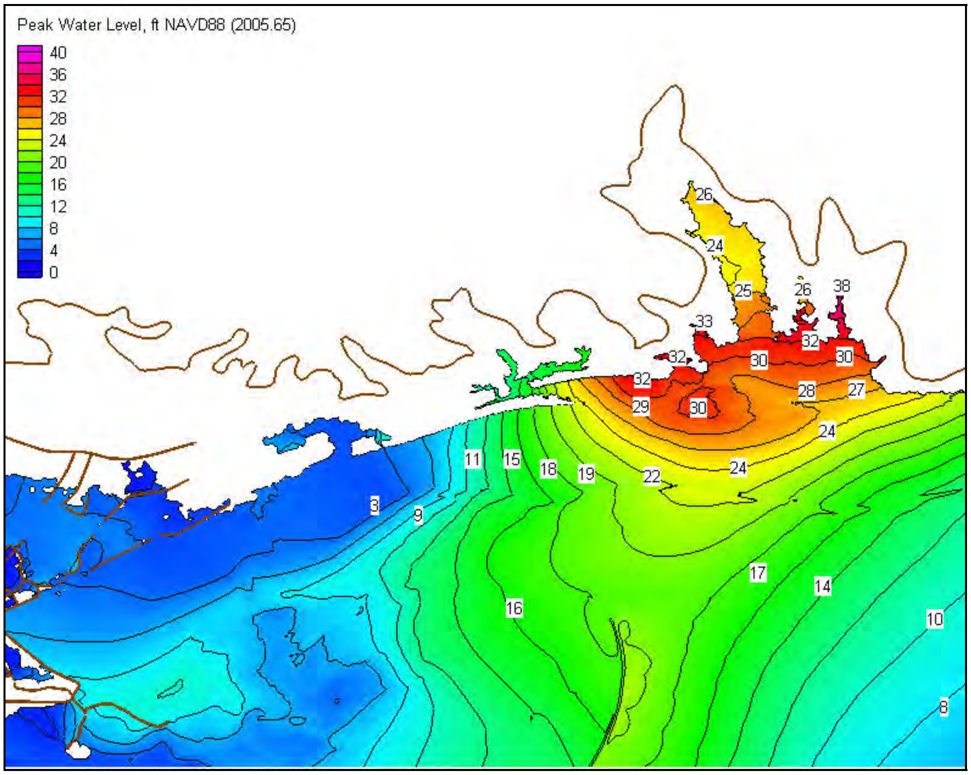


Figure 2.10-6. HST001-05 peak storm surge; Post-Katrina configuration

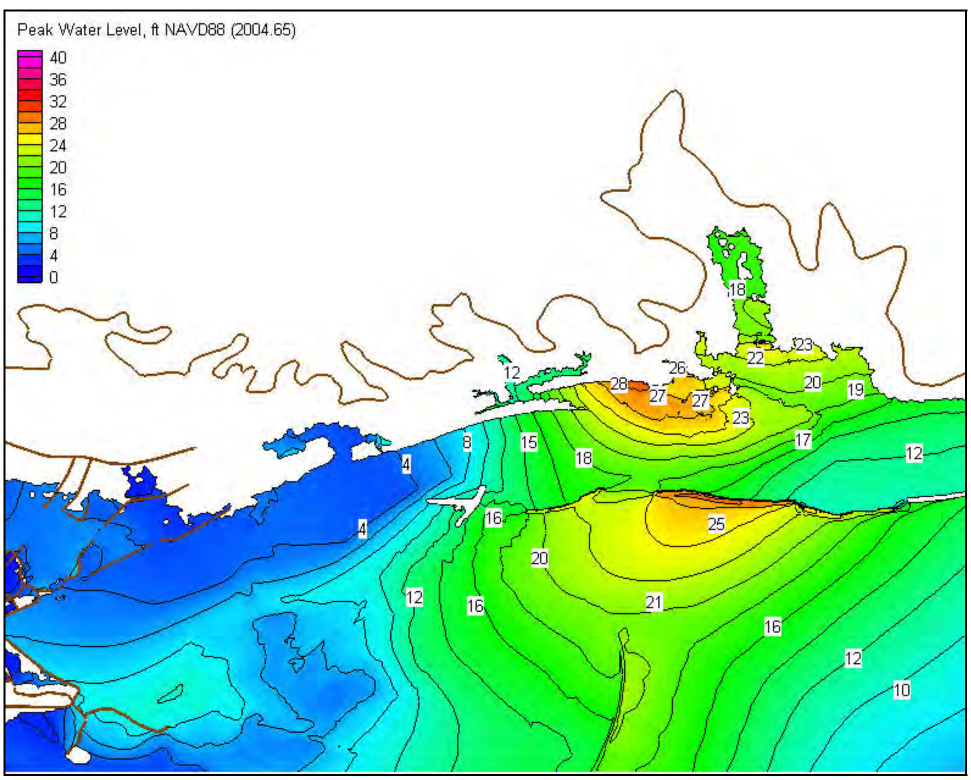


Figure 2.10-7. HST001-05 peak storm surge; Restored-High configuration

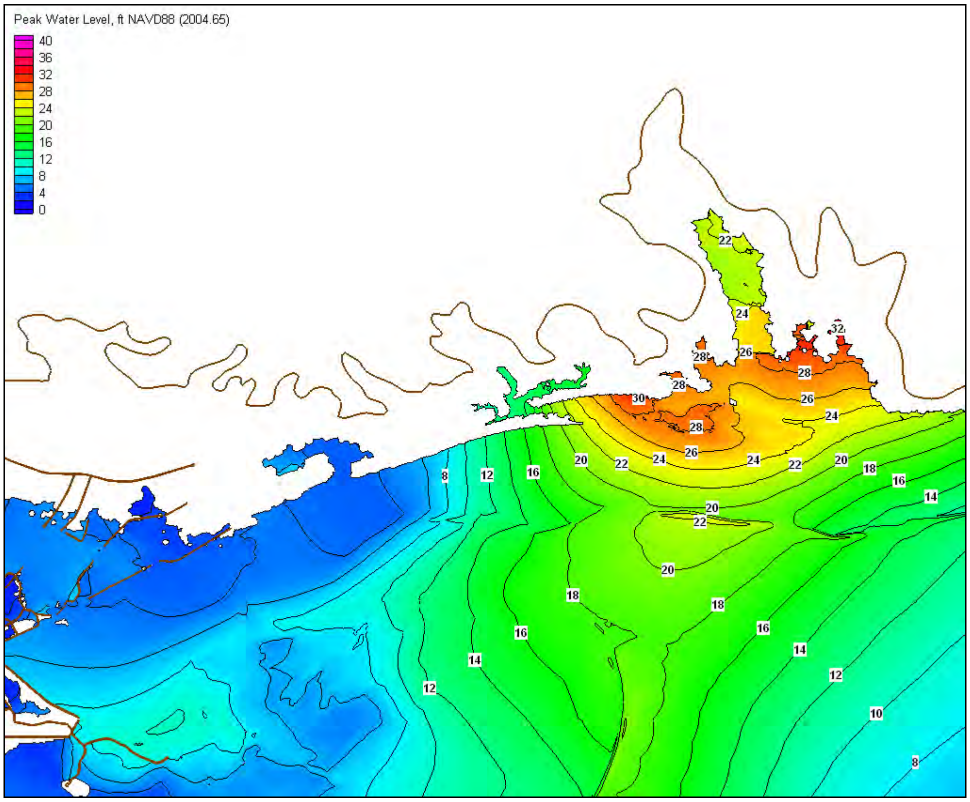


Figure 2.10-8. HST001-05 peak storm surge; Restored-Low configuration

Peak water levels for the HST002-05 storm (Figures 2.10-9 and 2.10-10) are less than the HST001-05 water levels, as expected. The general geographical area of maximum surge however is in the same location (Biloxi and Pascagoula). Peak water levels for the Post-Katrina barrier islands are 14-16 ft in Pascagoula and 16-18 ft near Biloxi. Penetration distance into the Pascagoula basin for the less intense storm is less, as expected. Peak water levels for the Restored-High barrier islands are 10-11 ft in Pascagoula and 14 ft at Biloxi. Water levels seaward of the raised barrier islands are elevated compared to the Post-Katrina barrier islands. Note that even for the 13 ft surge potential storm, the barrier islands are I inundated for the Post-Katrina configuration and remain dry for the Restored barrier island configuration.

Peak water levels for the HST003-05 storm (Figures 2.10-11 and 2.10-12) are less than the HST001-05 and HST002-05 water levels, as expected. The penetration distance into the Pascagoula basin is shorter and peak water levels are only 5-7 ft. Water levels at the entrance to Biloxi Bay are 10-12 ft for the Post-Katrina configuration and 10 ft for the Restored-High barrier islands. The difference between the Post-Katrina and Raised-High barrier island peak surges level is approximately 1-2 ft. Surge build-up seaward of the raised barrier islands is observed near Ship and Horn Islands and the barrier islands are still inundated for the Post-Katrina configuration for the 8 ft surge potential storm.

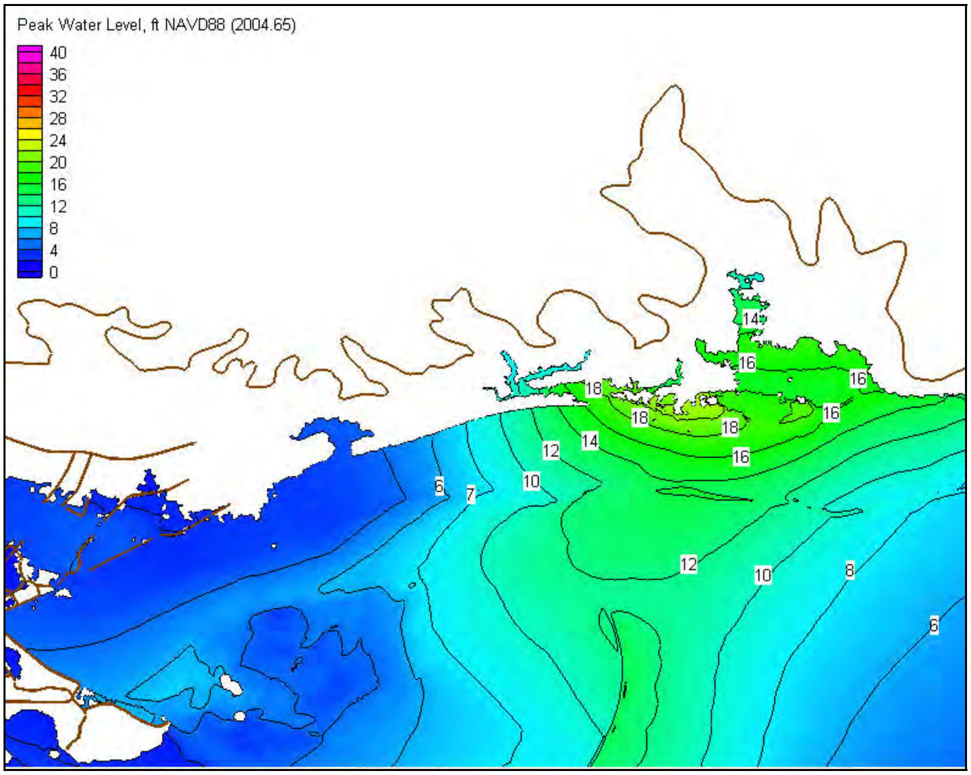


Figure 2.10-9. HST002-05 peak storm surge; Post Katrina configuration

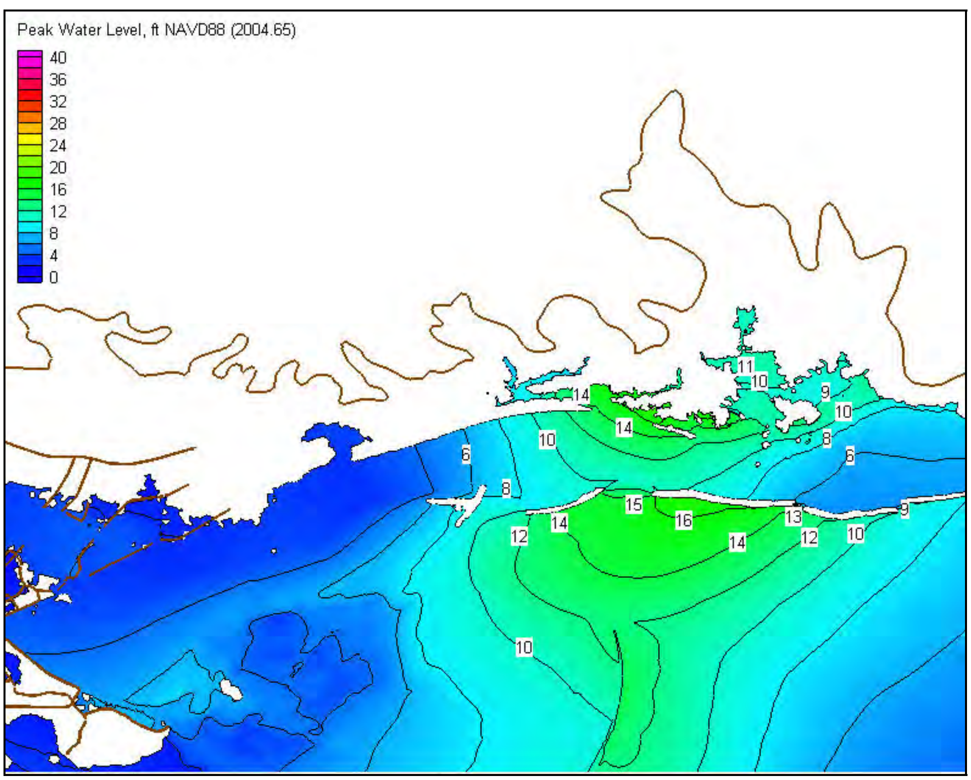


Figure 2.10-10. HST002-05 peak storm surge; Restored-High configuration

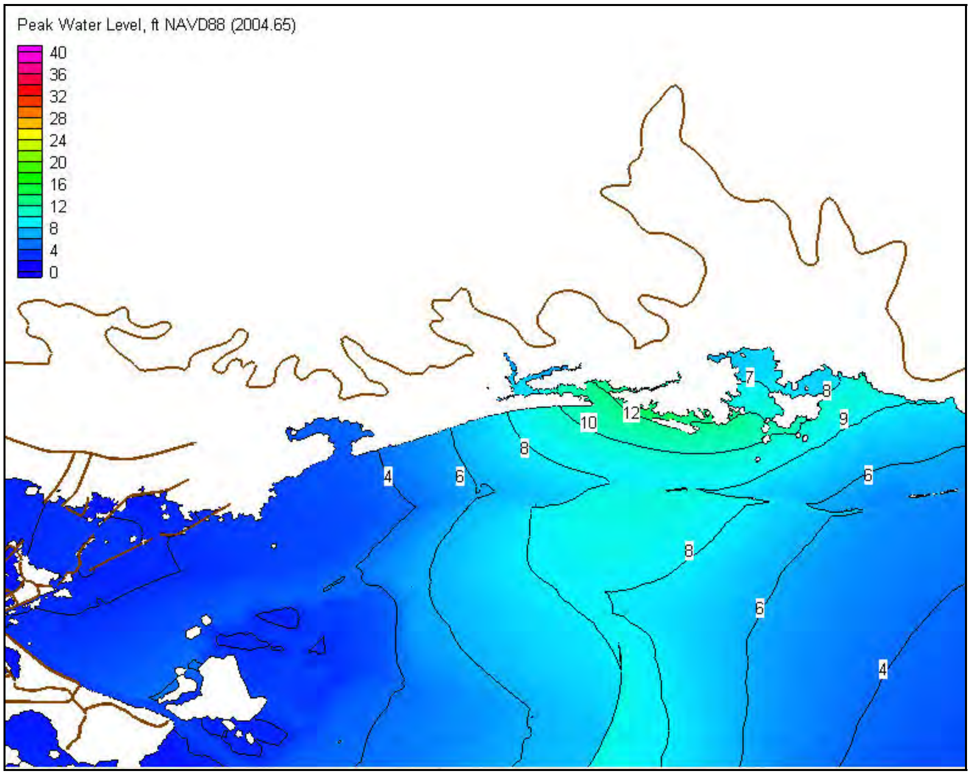


Figure 2.10-11. HST003-05 peak storm surge; Post-Katrina configuration

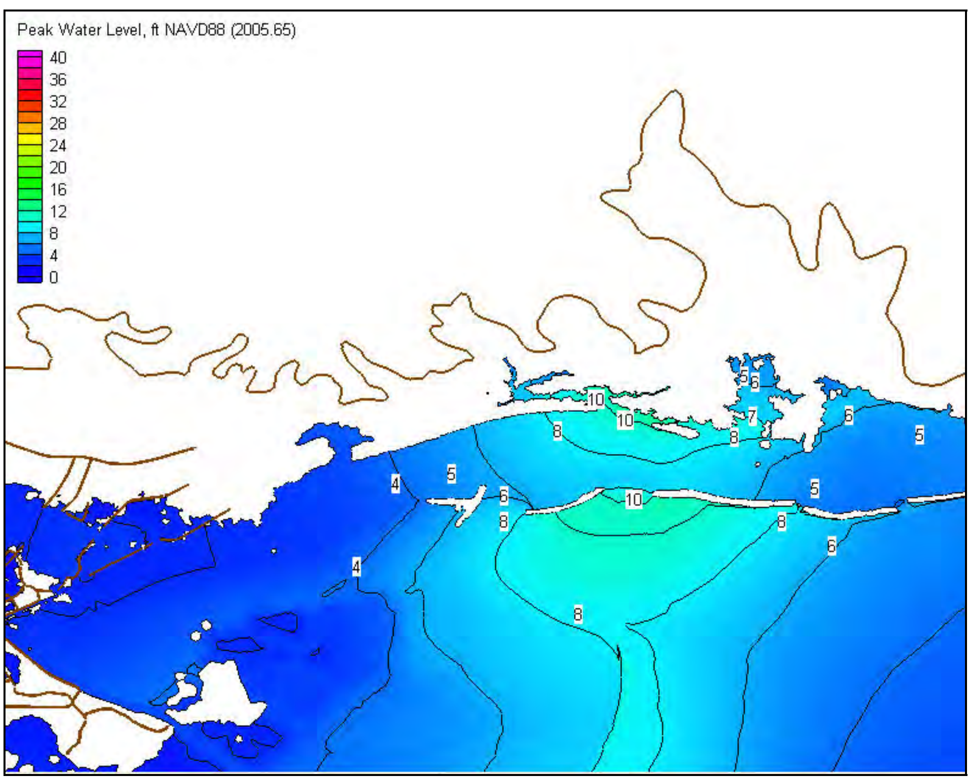


Figure 2.10-12. HST003-05 peak storm surge; Restored-High configuration

The difference in Hurricane Katrina peak water levels for the Restored-High barrier islands versus the Post-Katrina barrier islands (Figure 2.10-13) shows a reduction in water level of 1.0 to 3.5 ft landward of the barrier islands and an increase in water level of less than 1 ft seaward of the barrier islands. The most significant change in water level is in the Pascagoula basin where water levels are reduced 1-3 ft. Note that the area of maximum water level for Hurricane Katrina is in Waveland, which is west of all barrier islands that protect the Mississippi coast. Therefore, little change in peak water level (less than 0.5 ft) is observed in this region when the barrier islands are raised. Peak water levels near Biloxi are approximately 20 ft for Post-Katrina and 18 ft for the Restored-High condition. This area is afforded 1-2 ft of surge protection from the raised Ship and Horn Islands. Further to the east, water levels in Pascagoula are reduced from 12-16 ft to 10-12 ft with the presence of the raised Horn, Petit Bois, and Dauphin Islands. The percent reduction in peak water level landward of the barrier islands is greatest in the eastern part of the state (behind Horn and Petit Bois islands) and decreases to the west. Surge reductions were approximately 20% in the Pascagoula area, 5 to 10% in the central part of the state, and less than 5% in Waveland. Similar percent reductions in surge were calculated for Hurricane Camille.

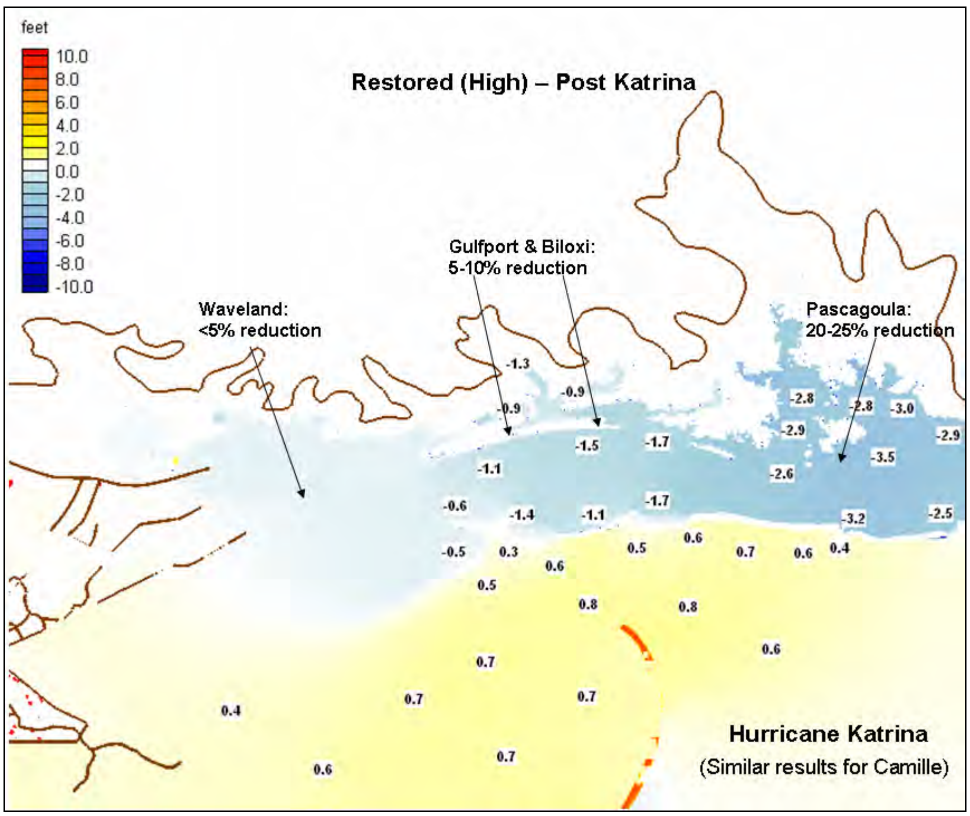


Figure 2.10-13. Difference in peak surge (Restored-High – Post Katrina) for Hurricane Katrina

A greater change in water level from the Post-Katrina to the Restored-High barrier island configuration was observed for the HST001-05 (Figure 2.10-14). HST001-05 made landfall at Biloxi and maximum water levels are observed east of the landfall point, where hurricane winds are strongest. This track passes between Ship and Horn Islands, therefore restoring Horn Island causes a major buildup of surge seaward of this island. Water levels landward of the barrier islands are greatly reduced for the Restored-High barrier island configuration. Post-Katrina flooding extends well into the Pascagoula basin. Water levels are 6-8 ft less in Pascagoula and 2-3 ft less near Biloxi for the Restored-High barrier island configuration. Seaward of the barrier islands surge levels increase 2-4 ft for the Restored-High barrier island configuration. The percent reduction in peak water level landward of the barrier islands is greatest in the eastern part of the state (behind Horn and Petit Bois islands) and decreases to the west. Surge reductions were approximately 20-40% in the Pascagoula area, 5 to 15% in the central part of the state, and less than 5% in Waveland. Similar percent reductions in surge were calculated for other tracks.

A much smaller change in water level is observed between the Post-Katrina and Restored-Low barrier island configurations (Figure 2.10-15). Water level reductions are cut in half relative to the high restoration with 3-4 ft less surge compared to Post-Katrina in Pascagoula and 0-2 ft less in the central part of the state. The percent reduction in peak water level landward of the barrier islands is again greatest in the eastern part of the state (behind Horn and Petit Bois islands) and decreases to the west. However, surge reductions are approximately 10% or less coast –wide. Similar percent reductions in surge were calculated for other tracks.

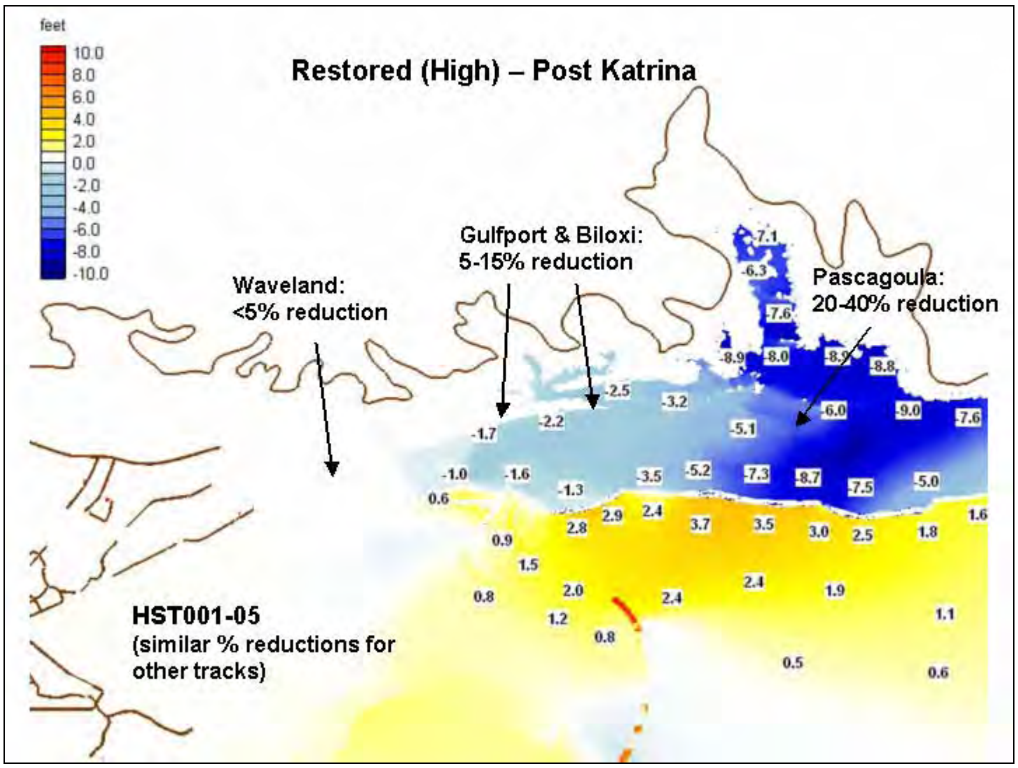


Figure 2.10-14. Difference in peak surge water level (Restored-High – Post Katrina) for HST001-05

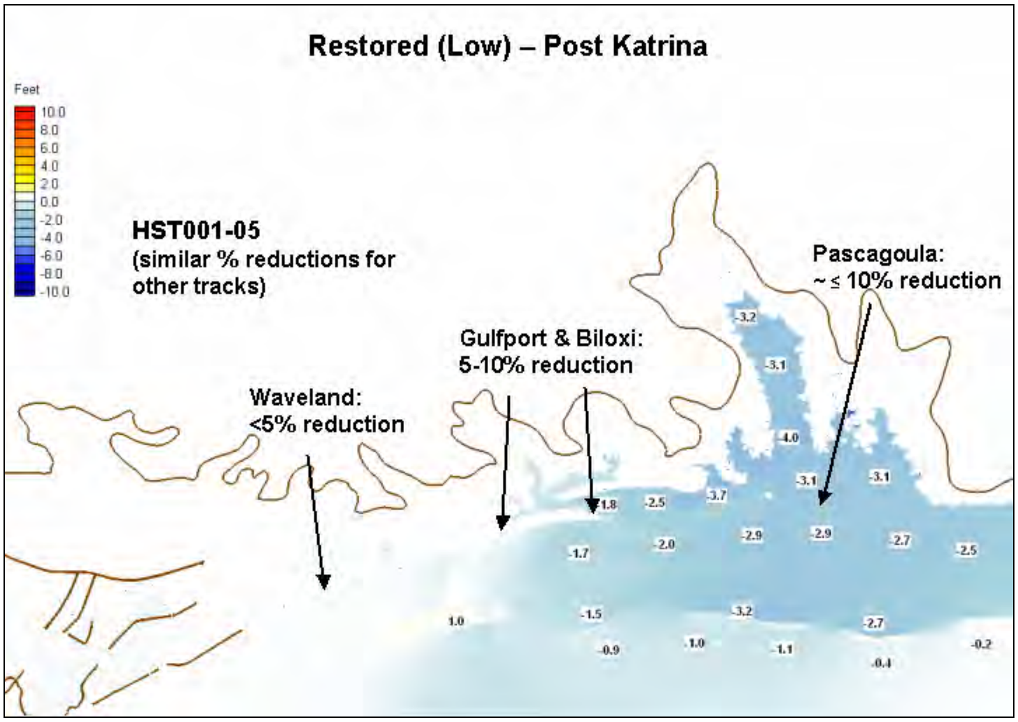


Figure 2.10-15. Difference in peak surge water level (Restored-Low – Post Katrina) for HST001-05

For HST002-05, the difference in the peak water level for the Restored-High barrier island configuration versus the Post-Katrina shows that water levels landward of the barrier islands are reduced 5-6 ft (Figure 2.10-16). Note that the inundation was less for the 13 ft surge potential storm compared to the 22 ft surge potential storm and the reduction in water level is also less for the weaker storm. Reduction in water level with the Restored-High barrier islands is most significant in Pascagoula (5-6 ft) and Biloxi (3 ft). Increased water level seaward of the barrier islands is most significant near Horn Island, but the build up is not as intense as for HST001-05. The percent reduction in surge was greatest for this storm.

With HST003-05, peak water levels were only 10-12 ft at the entrance to Biloxi Bay for the Post-Katrina barrier islands and 10 ft with the Restored-High barrier islands. The difference in peak water level for the Restored-High versus Post-Katrina barrier island configurations shows the protection afforded by the raised barrier islands is 1 to about 2.5 ft at the coast (Figure 2.10-17). The general pattern/area that is protected by the raised barrier islands is still the Pascagoula (~2 ft) and Biloxi (~1.5 ft) areas and surge build-up is observed seaward of Horn Island (~1 ft).

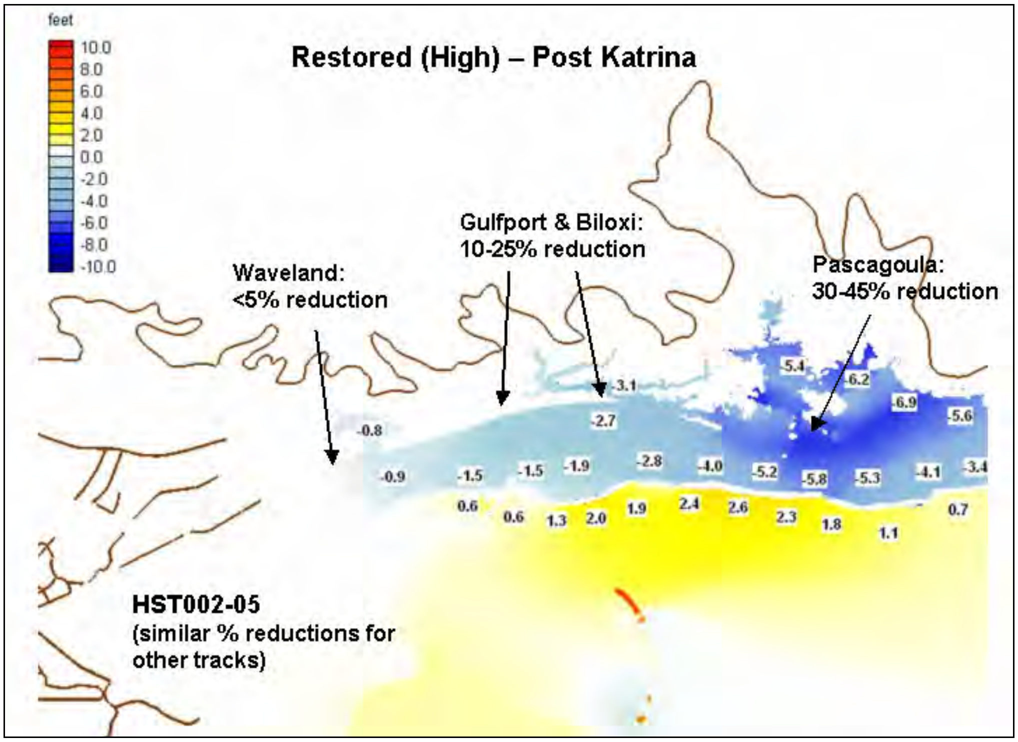


Figure 2.10-16. Difference in peak surge water level (Restored-High – Post Katrina) for HST002-05

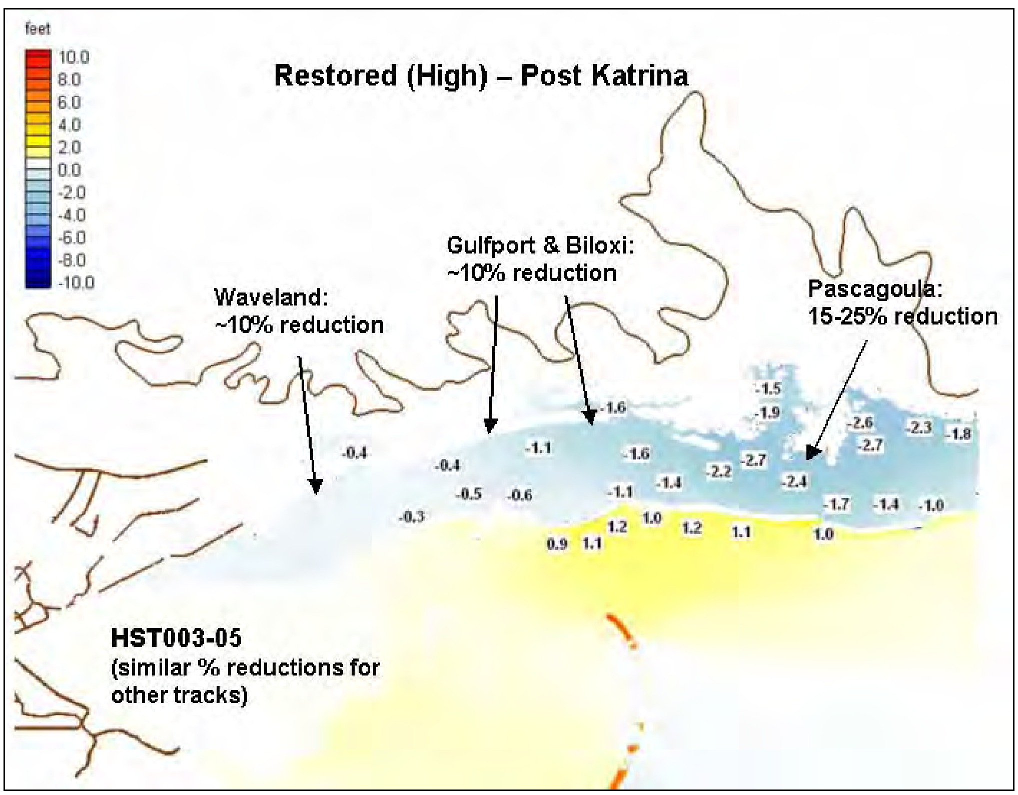


Figure 2.10-17. Difference in peak surge water level (Restored-High – Post Katrina) for HST003-05

2.10.4 Summary

The model results indicate that the barrier islands do provide some level of protection for most of the Mississippi coast, and that restoration of the islands will reduce surges at the mainland coast. The higher and greater in planform extent the islands are, the greater amount of protection the islands provide. The barrier islands do not significantly reduce surges in Hancock County and the surge reductions increase moving from west to east, with the greatest reductions in Jackson County. While model results showed that surge in Jackson County was reduced by as much as 40% for the Restored-High configuration, this represents a massive barrier island configuration that would be difficult to achieve. The Restored-Low is a more likely restoration scenario with pre-Katrina footprints and heights of 10 ft or less, the percent reduction in Jackson County for this configuration was much less at approximately 10%.

2.11 Wetlands, Landscape Features, and Storm Surge

Topography, landscape features, and vegetation 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 section is to present a literature review that documents studies that have measured and modeled storm surge elevations with the goal of understanding how landscape features and vegetation modify the surge elevation. A sensitivity study of a degraded and restored Biloxi marsh utilizing the modeling tools applied for this study is also presented. Sensitivity to barrier islands is discussed in Section 2.10.

2.11.1 Literature Review

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.

2.11.1.1 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. Obtaining reliable data from field observations is difficult as many factors control the elevation of the surge. To properly characterize the influence of landscape features and vegetation on storm surge, measurements should be (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 2.11-1). The relationships developed from the limited data available and discussed below do not, in general, adhere to these requirements and therefore should not be relied upon for engineering design.

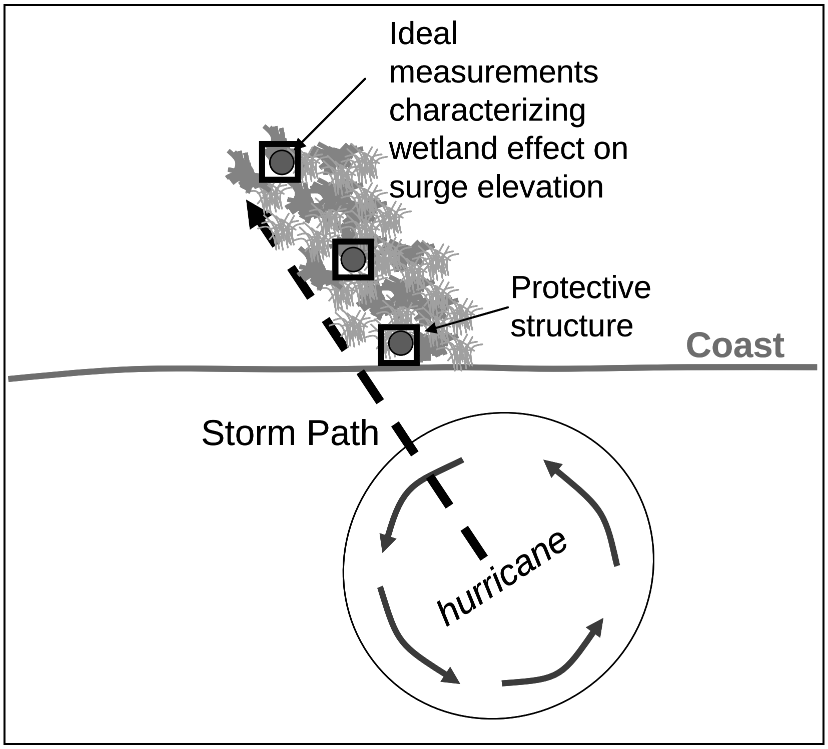


Figure 2.11-1. Ideal measurements for isolating the influence of landscape features on storm surge elevations

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 2.11-2). 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).

Lovelace (1994) documented storm surge elevations after Hurricane Andrew in Louisiana. 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 by 69,000 and 49,000 acres, respectively, between 1950 and 1990. Surge levels greater than 4.6 m (15 ft) were not significantly different between the two time periods.

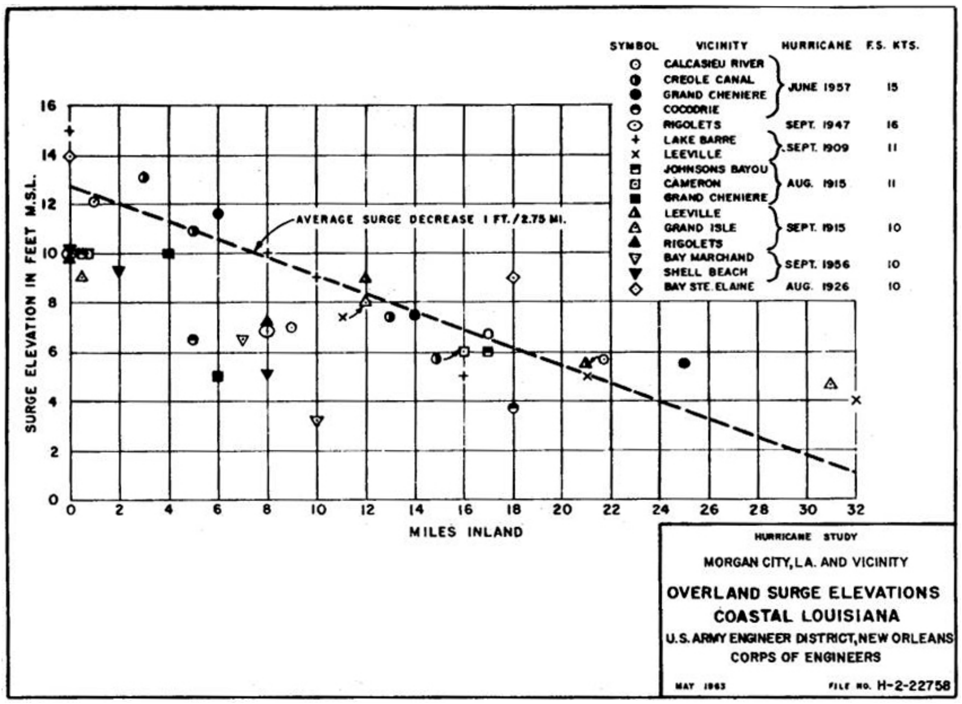


Figure 2.11-2. Observed maximum surge high water marks versus distance inland (USACE 1965)

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 a 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.

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.

2.11.1.2 Engineering Relationships

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.

2.11.1.2.1 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 number 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.

2.11.1.2.2 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:

$$E_{i,j} = \frac{\int_0^T \int_0^{h_i} \left| F_{i,j} u \right| dz\, dt}{T} \qquad \text{E 2.11-1}$$

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:

$$F_{i,j} = \frac{\rho C_D D_{i,j} \left| u \right| u}{2} \qquad \text{E 2.11-2}$$

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.

2.11.1.2.3 Currents and Storm Surge Elevation

Currents and surge are affected by coastal landscape features through two mechanisms. Bottom friction is 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). Bottom friction only 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,

$$\tau = \frac{g|U|u}{C^2}, \ \ and \ \ C = \frac{1.486h^{1/6}}{N} \qquad \text{E 2.11-3}$$

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),

$$N = Ah^{-B} \qquad \text{E 2.11-4}$$

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.

Flow-drag resistance also occurs in vegetated areas and represents flow resistance within the water column. 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

$$F_d = \frac{\rho C_d n D h_p V^2}{2} \qquad \text{E 2.11-5}$$

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,

$$F_d = \frac{\rho f V^2}{8} \qquad \text{E 2.11-6}$$

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

$$f = a\sigma^b,\ and\ \sigma = nDh_p \qquad \text{E 2.11-7}$$

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.

2.11.2 Sensitivity Analysis

A sensitivity analysis was performed to assess the impact of bathymetric and frictional resistance changes on ADCIRC-simulated peak surge elevations and STWAVE-simulated waves. The impact of coastal landscape features on surge propagation and waves is a relatively new application for surge and wave models and an area of active research that suffers from a lack of quality data. The purpose of the analysis is to qualitatively assess the potential of coastal features for reducing storm surge and waves for hurricanes with varying intensity. The analysis provides valuable information on trends and relative performance but should not be taken as a quantitative assessment of surge and wave reduction. It should be noted that the analysis does not consider the changes to the landscape that occur during a storms passage, where vegetation cover can be stripped away and land masses eroded. The analysis also does not consider changes in the structure of the hurricane itself due to landfall infilling phenomenon that may be influenced by landscape features. The physics of this process are not well known and research in this area is required. The analysis was performed for differing configurations of the Biloxi marsh on the Louisiana coast, south of the Mississippi coast.

A base configuration consistent with that applied for the Interagency Performance Evaluation Team (IPET) study and two Biloxi marsh conditions, one degraded and one improved from the base case, are evaluated. The marsh elevations for both the improved and degraded cases were provided by the environmental group at the USACE New Orleans District. Two storms were simulated for each configuration: 1) Hurricane Katrina as it occurred in August 2005, making landfall as a Saffir-Simpson scale Category 3 (CAT3) storm and 2) Hurricane Katrina reduced to a Category 1 (CAT1) storm at landfall. The smaller CAT1 storm was only simulated with ADCIRC without radiation stress forcing. STWAVE simulations were performed for both the CAT3 and CAT1 storms. The analysis first discusses the friction formulations in the models that impact surge and waves. The peak water levels and waves for each marsh configuration are then compared to the base condition. Finally, results for storms with varying intensity are compared.

2.11.2.1 Landscape Feature Roughness and Frictional Resistance

Coastal landscape features can reduce surge potential by reducing surface winds due to higher sub-aerial surface roughness and slow surge propagation due to bottom friction in shallow flow at the inundation front. The base condition coastal feature landscape land cover type was taken from the USGS National Land Cover Dataset (NLCD) classification raster map based upon Landsat imagery. Each NLCD classification has an associated land roughness length ($z_{0_{land}}$) and Manning's n value as defined by Federal Emergency Management Association (2005). These values are applied in the models as described below to reduce wind and water speeds. The values used for this analysis are summarized in Table 2.11-1. It should be noted that the passage of large storms can alter the landscape, stripping away vegetation cover is some areas and this impact is not considered in this analysis.

Table 2.11-1.
$Z_{0\text{-land}}$ Factors and Manning's n Values for NLCD Classifications

NLCD Class	Description	$Z_{0\text{-land}}$	Manning's n
11	Open Water	0.001	0.020
12	Ice/Snow	0.012	0.020
21	Low Residential	0.330	0.070
22	High Residential	0.500	0.140
23	Commercial	0.390	0.050
31	Bare Rock/Sand	0.090	0.040
32	Gravel Pit	0.180	0.060
33	Transitional	0.180	0.100
41	Deciduous Forest	0.650	0.120
42	Evergreen Forest	0.720	0.150
43	Mixed Forest	0.710	0.120
51	Shrub Land	0.120	0.050
61	Orchard/Vineyard	0.270	0.100
71	Grassland	0.040	0.034
81	Pasture	0.060	0.030
82	Row Crops	0.060	0.035
83	Small Grains	0.050	0.035
84	Fallow	0.050	0.030
85	Recreational Grass	0.050	0.025
91	Woody Wetland	0.550	0.100
92	Herbaceous Wetland	0.110	0.035
95	Cypress Forest	0.550	0.100

The winds input to the ADCIRC and STWAVE models are reduced to account for the higher surface roughness through a directional land masking procedure. Since the wind boundary layer does not adjust to surface roughness instantaneously, wind reduction factors are computed based on the weighted average of roughness coefficients (z_{0land}) within 10 km in the upwind direction. The wind reduction factor (f_r) is calculated as (Powell et al. 1996, Simiu and Scanlan 1986):

$$f_r = \left(\frac{z_{0_{marine}}}{z_{0_{land}}} \right)^{0.0706} \qquad \text{E 2.11-8}$$

where $z_{0marine}$ is the marine roughness length that is computed based on the Charnock relationship (Charnock 1955) and the relationship between the friction velocity and the applied drag law (Hsu 1988):

$$z_{0_{marine}} = \frac{\alpha_c C_d W_{10}^2}{g} \qquad \text{E 2.11-9}$$

where the Charnock parameter (α_c) is set to 0.018, C_d is the air-sea drag coefficient, W_{10} is the wind speed sampled at a 10 m height over a 10 min period, and g is the acceleration due to gravity. As inundation takes place, landscape features are submerged and their roughness is reduced. In the model, the roughness length is reduced according to (Simiu and Scanlan 1986):

$$z_0^{'} = z_{0land} - \frac{d}{30} \qquad \text{for } z_0^{'} \geq z_{0_{marine}} \qquad \text{E 2.11-10}$$

where d is the local water depth. The reduced roughness length is limited to the marine roughness value.

In addition to reducing wind speeds, coastal landscape features can also inhibit wind from penetrating through the features and shelter the water surface from wind stress. Features such as heavily forested canopies allow little momentum transfer from wind fields to the water column (Reid and Whitaker 1976) and thus areas classified as NLCD forest do not apply a wind stress in the model.

The speed at which a storm surge propagates and thus surge water level is affected by coastal landscape features through bottom friction and form drag. Bottom friction is the generated by fluid shear stresses at the water bottom and flow-drag resistance is generated by fluid stresses on objects extending through the water column (FEMA 1985). Bottom friction occurs in relatively shallow areas and bottom friction and flow-drag resistance can occur in vegetated areas. The ADCIRC and STWAVE models presently only account for bottom friction, the effect of form drag can only be approximated by increasing the bottom friction coefficient. The ADCIRC and STWAVE models apply a Manning's-type bottom friction formulation with the bottom friction coefficient specified as:

$$C_f = g \frac{n^2}{d^{1/3}} \qquad \text{E 2.11-11}$$

where n is the Manning roughness coefficient with values based on the USGS land use factors. The Manning n values applied for this analysis are summarized in Table 2.11-1. The values applied in the model for both the Manning n values and the roughness coefficients were validated through comparison of model hindcast results and measured high water marks for Hurricanes Katrina and Rita.

2.11.2.2 Sensitivity of Peak Water Level and Maximum Waves to Marsh Condition

The CAT3 and CAT1 storms were simulated on a base configuration, a case with Biloxi marsh raised to 1.05 ft NAVD 88 (2004.65) and restored to herbaceous wetland, and a case with Biloxi marsh lowered to -2 ft NAVD 88 (2004.65) and represented as open water. For the degraded case, the entire Biloxi marsh area was lowered and for the restored case only two strips of Biloxi marsh were altered (see Figure 2.11-3). Discussion of the peak water levels refers to the surge-plus-wave simulation results for the CAT3 storm only. Similarly, peak wave height maps for each of the CAT3 simulations were compared to the baseline configuration. In addition, maximum wave height maps were compared for the CAT1 simulations.

Figure 2.11-3. Outline of marsh areas restored (red) and deteriorated (blue)

Biloxi marsh was raised and restored to herbaceous wetland along two 100,000 ft by 15,000 ft strips of marshland for a total raised area of 116 sq mi. The change in bathymetry and frictional resistance slowed surge propagation resulting in an increase in water level seaward of the change and a decrease in water level landward of the change. The greatest change in peak water level was observed landward of the marsh feature where the decrease in peak water level was 0.9 to 1.3 ft (Figure 2.11-4). Changes on the Mississippi coast were less than 1 ft. The percent reduction in peak water level was as much as 8% in the New Orleans area and less than 3% on the Mississippi coast. The original STWAVE grid was also modified to represent the raised Biloxi marsh configuration and changes in maximum wave height are shown in Figure 2.11-5. The greatest change in maximum wave height was observed landward of the marsh feature where the decrease in maximum wave height was 1.0 to 2.0 ft for the CAT3 simulation. No changes in wave height were estimated on the Mississippi coast. Similar result patterns were observed for the reduced Katrina CAT1 simulation. The wave change patterns are consistent with the water level changes suggesting that the waves are depth limited.

The spatial extent where Biloxi marsh was lowered greatly exceeds the area that was raised. The lowered Biloxi marsh area encompassed 507 sq mi and was degraded (lowered) to -2.0 ft (NAVD88 2004.65) and returned to open water. The marsh reduction allows surge to propagate more rapidly, resulting in a rise in peak water level (relative to the original simulation peak water level) that extended beyond the Biloxi marsh region. The Biloxi marsh area increased in peak water level by 2.4 ft at the MRGO-GIWW junction (Figure 2.11-6). Peak water level south of English Turn increased 1.1 ft, Lake Pontchartrain peak water levels increased approximately 0.8 ft, and water levels on the Mississippi coast increased by less than 0.5 ft. The percent increase in peak water level was 8-13% near the New Orleans area levees but was less than 3 percent on the Mississippi coast. A map of changes in maximum wave height is shown in Figure 2.11-7. The greatest change in maximum wave height was observed landward of the marsh feature where the increase in maximum wave height was 2.0 to 4.5 ft for the Katrina CAT3 simulation. Similar result patterns were observed for the reduced Katrina CAT1 simulation. The wave change patterns are consistent with the water level changes suggesting that the waves are depth limited.

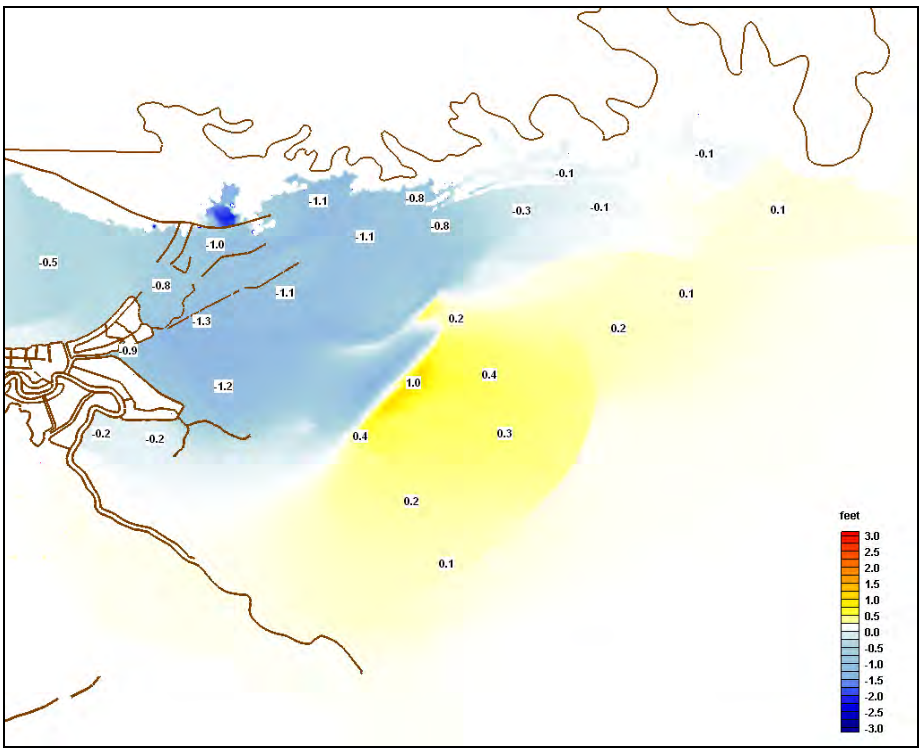


Figure 2.11-4. Difference in peak surge: Biloxi Marsh raised to 1.05 ft minus base configuration.

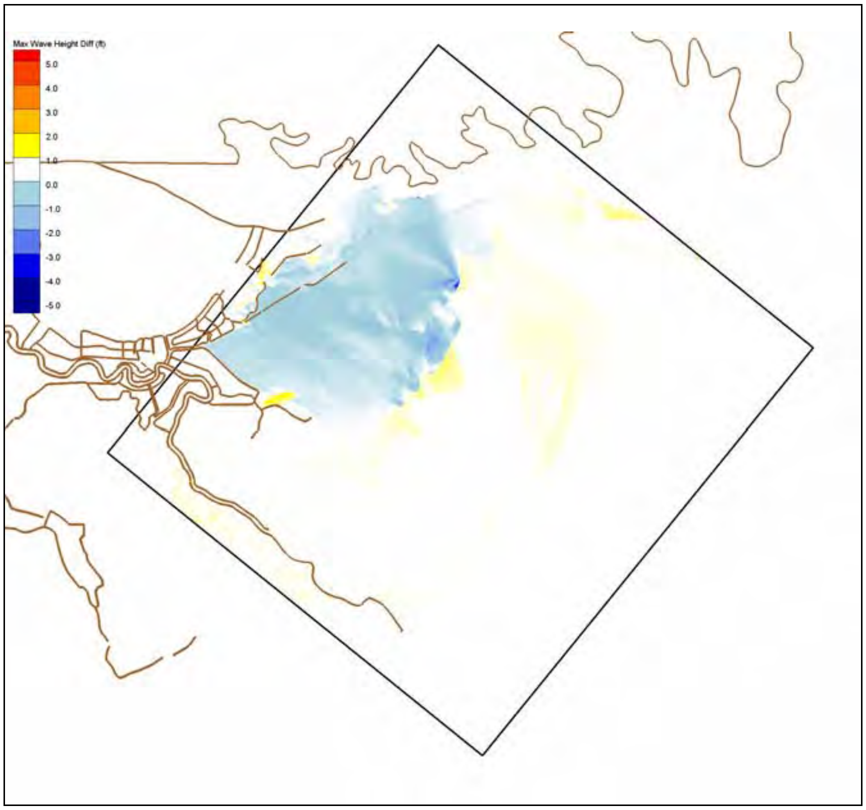


Figure 2.11-5. Difference in maximum wave height for CAT3 simulation: Biloxi Marsh raised to 1.05 ft minus base configuration.

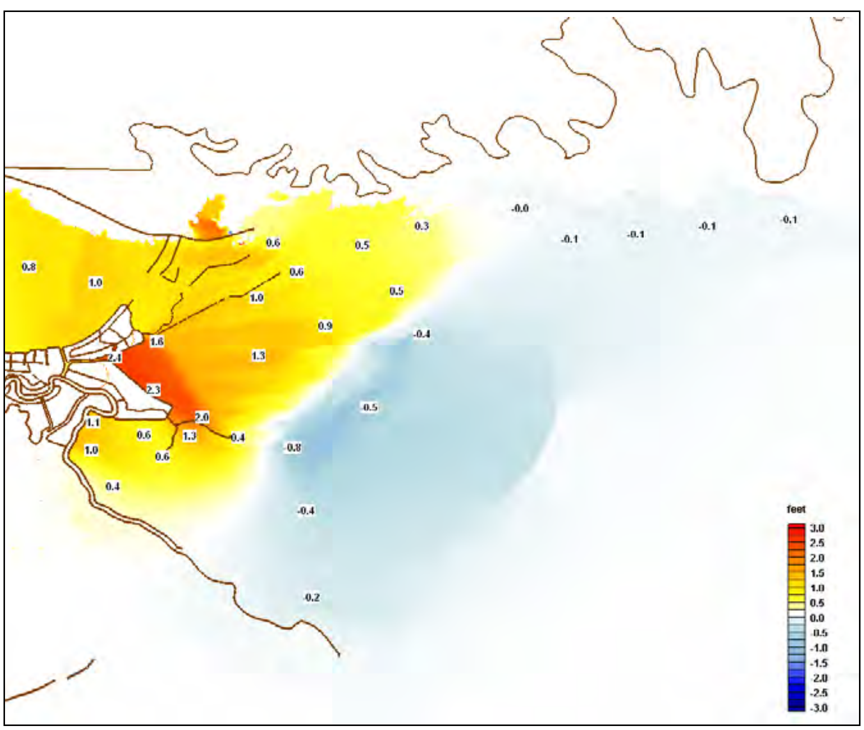


Figure 2.11-6. Difference in peak surge: Biloxi Marsh lowered to -2.0 ft minus base configuration.

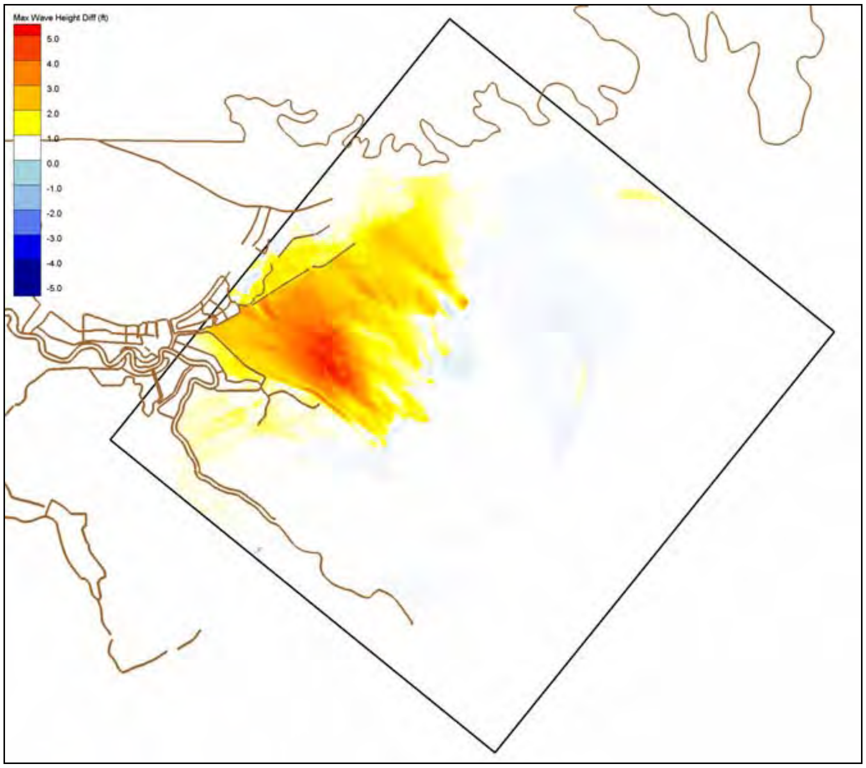


Figure 2.11-7. Difference in maximum wave height for CAT3 simulation: Biloxi Marsh lowered to -2.0 ft minus base configuration.

The CAT3 storm discussed in the previous section was scaled to produce a storm of Category 1 intensity on the Saffir-Simpson scale and thus has less surge potential. Simulations without wave radiation stress forcing were made for the less intense (CAT1) storm. Analysis and comparison of the CAT1 peak water level differences to the CAT3 surge only peak water level differences show similar patterns. Table 2.11-2 lists the peak water level change for the CAT1 and CAT3 storms for comparison. Water level changes on the Mississippi coast were less than 0.5 ft. The most significant difference in peak water level change between the CAT1 and CAT3 storms occurred when the marsh was lowered. The difference in surge for the two storm intensities for the lowered marsh configuration is given in Figure 2.11-8 and Figure 2.11-9. The peak water level is higher for the CAT3 storm, but the high peaks in Biloxi marsh extend over a broader area with the CAT1 storm. In the region further south from the lowered Biloxi marsh area, the CAT3 storm causes greater increases than the CAT1 storm in peak water level. For these cases, the change in peak water level change for a CAT3 storm can be more than double the change in peak water level for a CAT1 storm. A similar trend is observed with the STWAVE results when comparing storm intensity and maximum wave height. An example of the maximum wave height difference for the two storm intensities is given in Figures 2.11-7 and 2.11-10. Note that the maximum wave height change is larger and broader in extent for the CAT3 storm when compared to the CAT1 storm. This trend was especially evident for the lowered marsh configuration simulations performed for this study. For the raised marsh configuration, the area of changes in maximum wave heights is broader in extent, even if the maximum values are comparable for both CAT3 and CAT1 simulations.

Table 2.11-2.
Difference in Surge-only Peak Water Level for the Marsh Change Configuration – Original Marsh Configuration for CAT 1 and CAT3 Storms

Marsh Condition	Maximum Change in Peak Water Level, ft	
	CAT1	CAT3
Restored	-1.2	-1.4
Degraded	1.7	2.7

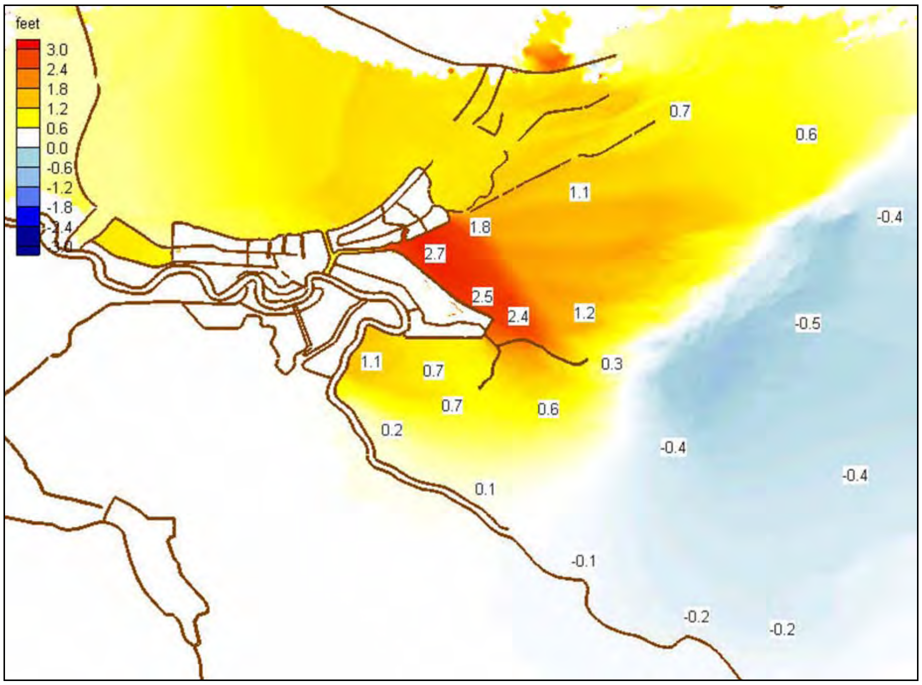


Figure 2.11-8. Difference in peak surge only (no radiations stresses): Biloxi Marsh lowered minus base configuration for a CAT3 storm.

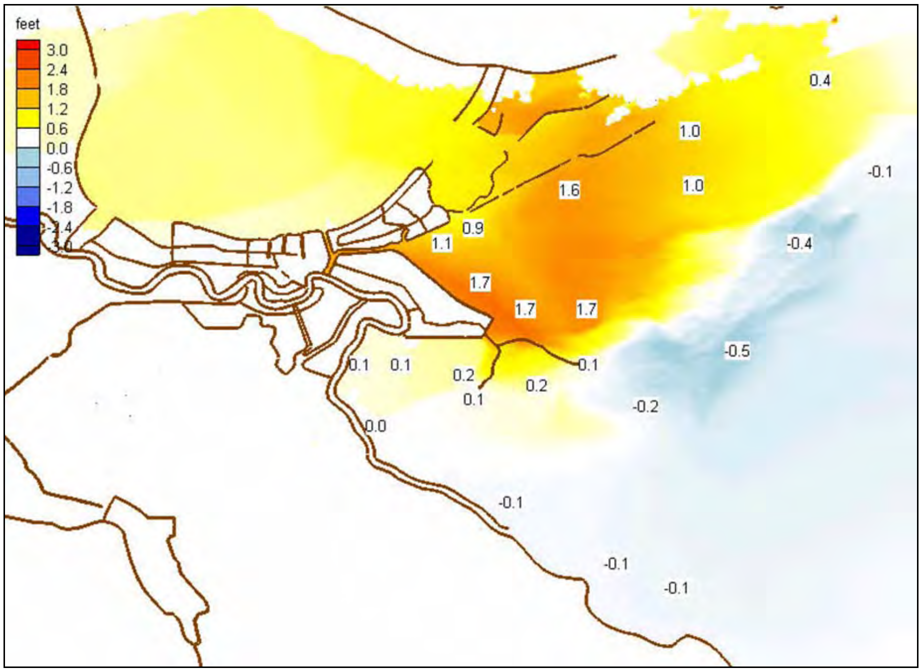


Figure 2.11-9. Difference in peak surge only (no radiation stresses): Biloxi Marsh lowered minus base configuration for a CAT1 storm.

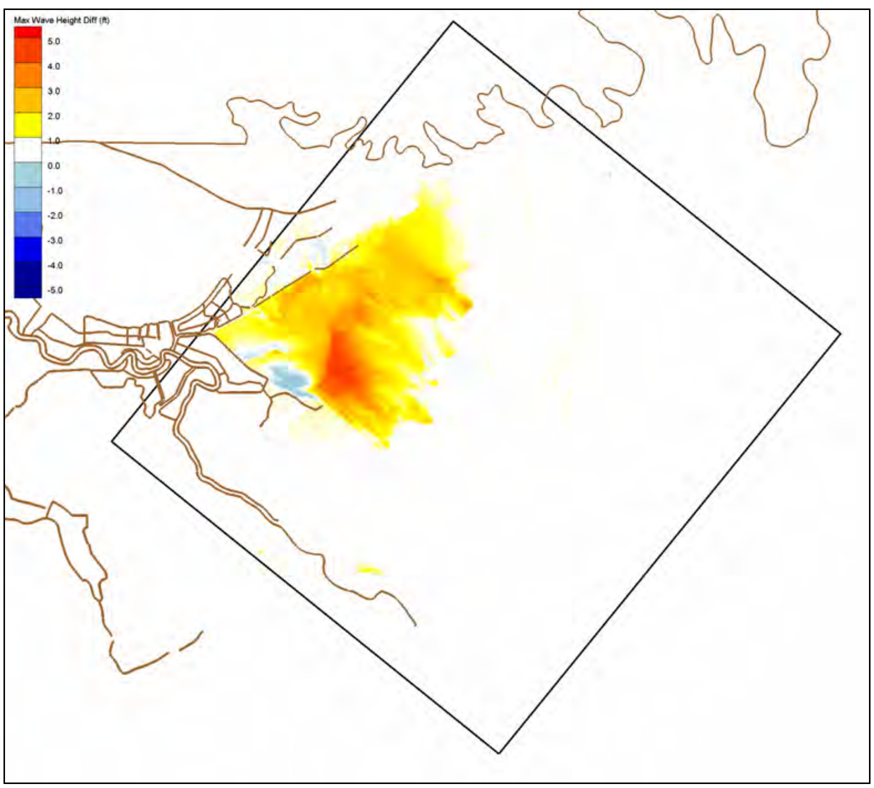


Figure 2.11-10. Difference in maximum wave height: Biloxi Marsh lowered minus base configuration for a CAT1 storm.

2.11.2.3 Summary

The purpose of this sensitivity analysis is to qualitatively assess the potential of coastal landscape features for reducing storm surge and waves for hurricanes with varying intensity. The impact of landscape features on surge propagation is a relatively new application for surge and wave models and an area of active research that suffers from a lack of quality data. The analysis provides valuable information on trends and relative performance but should not be taken as a quantitative assessment of surge and wave reduction. Results indicate that coastal marsh does have surge and wave reduction potential. Restoration and degradation of marsh resulted in decreases (for restoration cases) and increases (for degradation cases) in both surge and waves for both a Category 3 storm on the Saffir-Simpson scale and a less intense Category 1 storm. The magnitude of change was greatest for the more intense storm. The magnitude of change was also correlated with the magnitude of the horizontal extent and elevation changes in the marsh. In general, the wave change patterns are consistent with the water level changes suggesting that the waves are depth limited. Results indicate that the impact of the landscape features is amplified in areas where there are levee pockets, such as at the MRGO and GIWW junction and south of English Turn. Results also indicate that changes in the Biloxi marsh will have little or no impact on water levels and waves for the Mississippi coast.

2.12 Regional Sediment Budget

2.12.1 Purpose

This study evaluated the existing regional sediment transport magnitudes and directions for the Mississippi and Alabama barrier islands fronting Mississippi Sound and the mainland coast, including an analysis of historical long-term barrier island migration. Based on analysis of previous studies, historical bathymetric and shoreline change, and numerical modeling, a suite of sediment budgets was developed. First, a conceptual sediment budget was developed through a review of existing studies; this budget formed the framework for the historical and calculated sediment budgets. Next, a historical sediment budget was developed through analysis of bathymetric and shoreline position change through time. Engineering activities and significant storm events were also documented. A calculated sediment budget was developed based on numerical modeling of regional waves and sediment transport, for the Gulf and Bay shorelines of the barrier islands as well as the mainland coast. The final sediment budget was formulated from all these intermediate budgets, and is presented herein along with a summary of information pertinent to the final budget. Details about the conceptual, historical, and calculated sediment budgets and further discussion of the entire study can be found in Rosati et al. (2007).

2.12.2 Mississippi Coast Physical Setting and Processes

The barrier islands in the project area, Cat, West and East Ship, Horn, Petit Bois, and Dauphin Islands, provide the offshore boundary for Mississippi Sound (Figure 2.12-1). These islands are the first line of defense for the mainland as tropical storms, hurricanes, and cold fronts pass the region. Table 2.12-1 summarizes the tropical storm and hurricane history for locations in and around the study area from 1871 (or 1872) through 2006. Because data were not provided for a city in Hancock County, New Orleans, Louisiana is shown in Table 2.12-1 to provide a western boundary to the study area. Locations in Hancock County are assumed to have storm occurrences similar to those presented for New Orleans and Gulfport.

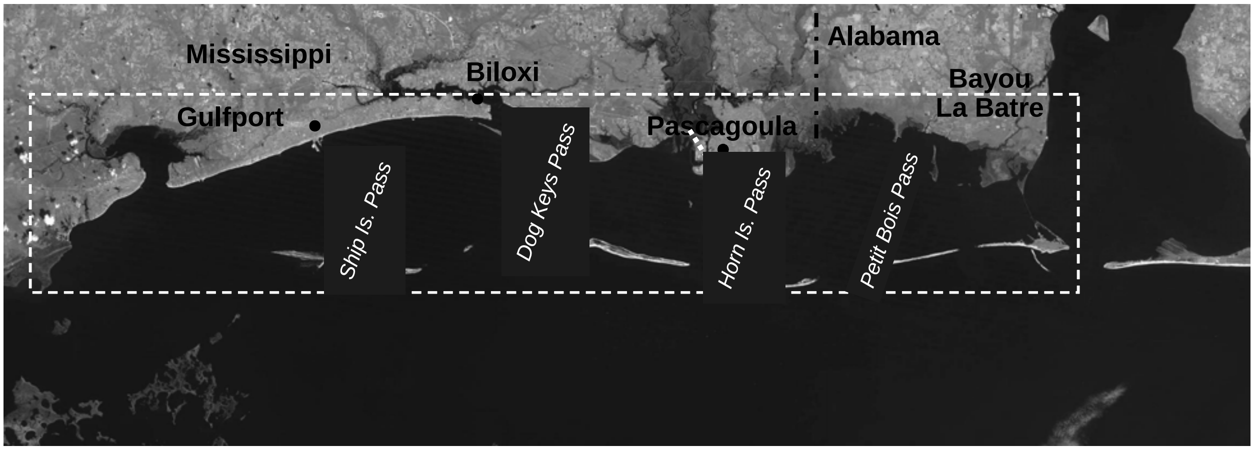


Figure 2.12-1. Mississippi Gulf Coast, showing barrier island system, navigation channels, and the area of study for the regional sediment budget (image courtesy NASA's Earth Observatory, dated 15 Sep 05)

Table 2.12-1.
Storms within 60 miles of Selected Mississippi, Alabama, and Louisiana Cities West of Mobile Bay, 1871/2 through 2006[1]

Location (from west to east)	Year of Storm Occurrence t=tropical storm; b=brush; h=hurricane	Frequency of Occurrence (yr)	
		Brush or Hit	Direct Hit
New Orleans, LA	1879h, 1879t, 1887h, 1888b, 1897b, 1892t, 1893h, 1900tb, 1901h, 1905t, 1907t, 1909h, 1914t, 1915h, 1916b, 1932t, 1934tb, 1936t, 1944tb, 1947h, 1948h, 1949t, 1955t, 1964t, 1965h, 1969b, 1979h, 1985b, 1988t, 1992b, 1998t, 2002t(2), 2004tb, 2005t, 2005h	3.8	12.4
Gulfport, MS	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	3.5	15.1
Biloxi, MS	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	4.4	11.3
Pascagoula, MS	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	3.8	9.7
Dauphin Island, AL	1880b, 1881t, 1882b, 1885, 1887t, 1893h, 1895tb, 1900t, 1901t, 1902t, 1904t, 1906h, 1910h, 1911b, 1912b, 1914tb, 1916b, 1919tb, 1922tb, 1923tb, 1926h, 1932h, 1934t, 1939t, 1944tb, 1947t, 1950h, 1956b, 1959t, 1960tb, 1979h, 1985h, 1985tb, 1995b, 1997h, 1998b, 2002t, 2004h, 2005(2)tb, 2005h	3.3	11.3

[1] http://www hurricanecity.com/. This database does not have any locations in Hancock County, Mississippi; thus, data for New Orleans, Louisiana are included to provide a western boundary for the study area. Locations in Hancock County are assumed to have storm occurrences similar to those provided for New Orleans and Gulfport.

The frequency of direct landfall is approximately equal for Biloxi, Pascagoula, and Dauphin Island, with a direct hit every 10-11 years. The likelihood for a direct hit decreases to approximately once every 15 and 12 years for Gulfport and New Orleans, respectively. However, all locations listed in Table 1 have historically been brushed or hit with a tropical storm or hurricane approximately once every 3-4 years. Cold fronts, although less intense than tropical storms and hurricanes, occur more frequently at approximately 30 to 40 times per year (Stone et al. 1999).

The barrier islands protecting Mississippi Sound experience a low energy wave climate, with average significant wave height at National Data Buoy Center (NDBC) Buoy 42007 (22 nautical miles south-southeast of Biloxi, in 46 ft depth) averaging 2 ft and 1.3 ft in the winter and summer months, with associated average peak wave periods of 4 to 3.5 sec, respectively. Wave transformation modeling by Cipriani and Stone (2001) indicated that breaking wave heights on the barrier islands range from 1 to 2 ft. Waves in Mississippi Sound are fetch- and depth-limited. The Coastal Studies Institute's Wave-Current Surge Information System (WAVCIS[3]) gage CSI-13 located at Ship Island Pass (23 ft depth) from June 1998 through July 2005 measured an average significant wave height of 0.3 ft and associated average wave period of 2.5 sec.

Tides in Mississippi Sound are diurnal, with a tidal range of 1.5 ft and 1.8 ft for the mean and spring tides at Biloxi, Mississippi[4], respectively. However, the relatively shallow and large area of the Sound create strong currents in the tidal passes between the barrier islands, ranging from 1.63 to 3.3 ft/sec and 5.9 to 11.5 ft/sec on flood and ebb tides, respectively (Foxworth et al. 1962). In the winter months, winds from the same direction and of a sufficient magnitude are capable of lowering water surface elevations in the bays and nearshore from 1-2 ft (U.S. Army Corps of Engineers Mobile District 1984).

For the Gulf barrier island beaches, net longshore sediment transport is from east to west, although local reversals in the net transport occur adjacent to the tidal passes. The primary sources of sediment are longshore sediment transport from east to west, and, potentially, the offshore shelf (Otvos 1979, Cipriani and Stone 2001). Cipriani and Stone (2001) discussed that a well-defined cellular structure exists for each barrier island in which, over historic times, little sand transfer exists between islands. However, dredging records at Horn Island and Ship Island Passes (also called Pascagoula Bar Channel and Gulfport Bar Channel, respectively) suggest that infilling of sand from adjacent barrier islands occurs, indicating the potential for transport of sand between islands. Eastern Dauphin Island, with a Pleistocene core, is more stable than the other barriers although eastern Dauphin Island has been eroding in response to the dominant westerly-directed transport. Based on grain size analysis, Cipriani and Stone (2001) inferred that offshore sources may provide some sediment to central Petit Bois Island. The Mississippi Sound barrier islands range from very well vegetated, with maritime forests on east Dauphin Island, to low elevation barriers that are overwashed and breached during hurricanes. Long-term relative sea level rise for Dauphin Island, Alabama from 1966 to 1997 was 0.12 +/- 0.023 in/year[5].

On the mainland coast, beach change in Harrison County has been dominated by harbor construction, beach restoration and replenishment since 1951 (Byrnes et al. 1993a, 1993b). Cross-shore sediment transport processes dominate beach change, with wave-induced sediment transport processes of secondary importance, typically from east-to-west (Byrnes et al. 1993a, 1993b). Hancock County had beach nourishment in 1993-1994 between Waveland and Bay St Louis and again in 1996 for the Bay St Louis Downtown beach (Schmid 2002). Net longshore transport in Hancock County is generally from northeast to southwest. The bays, distributaries, and bayous of

[3] http://www.wavcis.lsu.edu/, dated 11 December 2006, accessed 11 December 2006.
[4] http://tidesandcurrents.noaa.gov/tides05/tab2ec4.html#107, dated 25 March 2005, accessed 11 December 2006.
[5] http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8735180, dated 10 February 2006, accessed 29 July 2006.

the remaining coast are typically bordered with marsh populated by *Spartina-Juncus* succession (Christmas 1973).

2.12.3 Review of Existing Studies and Dredging Database

Existing studies were reviewed for the project area to provide information about sediment transport processes of the barrier island and mainland coast. This knowledge gained was incorporated into the sediment budget as appropriate. For a full summary of each study that was reviewed, please see Rosati et al. (2007).

Dredging rates for navigation channels within Mississippi Sound were also evaluated in the study. As shown in Figure 2.13-1, the study area is traversed by many navigation channels: two "bar" channels that extend through Horn Island Pass (also called Pascagoula Bar Channel) and Ship Island Pass (also called Gulfport Bar Channel); the Gulf Intercoastal Waterway (GIWW) that runs east-west through Mississippi Sound; and five Sound navigation channels that extend from Gulfport, Biloxi, Pascagoula, Bayou Cassotte, and Bayou La Batre. The SAM dredges these channels on a regular basis. The U.S. Army Corps of Engineers' Navigation Data Center[6] (NDC) has documented all Corps contract and non-contract dredging for all Districts for Fiscal Year (FY) 1990 through 2005. NDC's database for SAM's entire District dredging program is provided in Rosati et al. (2007).

Byrnes and Griffee (2007) culled historical dredging and placement information from published Corps reports and databases to develop annual dredging and placement rates for each of the bar channels. Sediment dredged from the GIWW and other channels extending through Mississippi Sound was side-cast or placed in disposal areas to either side of the channels, and is assumed to shoal primarily from fine sediment that is mobilized in the bay. Thus, these dredging and placement activities in the Sound do not change the sediment budget for the mainland and barrier islands. However, dredging and placement adjacent to the barrier islands (Ship Island Pass/Gulfport Bar Channel and Horn Island Pass/Pascagoula Bar Channel) must be considered in the sediment budget.

Dredging data provided by Byrnes and Griffee (2007) have been analyzed to provide estimated maintenance shoaling rates for each of the Bar Channels as a function of channel depth, width, and length (Table 2.12-2). Of particular interest is the maintenance dredging rate as a function of channel depth, as shown in Figure 2.12-2.

Table 2.12-2.
Summary of Dredging Rates for Navigation Channels Adjacent to Barrier Islands (modified from Byrnes and Griffee 2007)

Date	Description	New Work (cy)	Maintenance (cy)
Ship Island Pass/Gulfport Bar Channel (Data from 1881-2003)			
Mar 1899–Mar 1948	26-ft deep, 300-ft width, 0.76-mile long channel (1.9-mile length dredged in 1922)	163,401	2,115,576 (43,175 cy/yr) (33,028 cu m/yr)
Mar 1948–Jul 1992	32-ft deep, 300-ft wide, 8 miles long	3,679,044	21,111,495 (476,200 cy/yr) (364,292 cu m/yr)
Nov 1993–Apr 2003	38-ft, 300-ft wide, 8 miles long	9,695,988	5,456,817 (579,485 cy/yr) (443,306 cu m/yr)

[6] http://www.iwr.usace.army.mil/NDC/data/datadrg.htm , updated 25 July 2006, accessed 13 December 2006.

Table 2.12-2.
Summary of Dredging Rates for Navigation Channels Adjacent to Barrier Islands (modified from Byrnes and Griffee 2007) (continued)

Date	Description	New Work (cy)	Maintenance (cy)
1899 to 2003	**Total Dredging**	**13,538,433**	**28,683,888 (275,807 cy/yr) (210,992 cu m/yr)**
Horn Island Pass/Pascagoula Bar Channel (Data from 1881-2005)			
Feb 1897–Mar1948	25-ft deep, 300-ft wide channel	896,748	1,735,817 (34,000 cy/yr) (26,010 cu m/yr)
Mar 1948–Jan 1965	38-ft deep, 325-ft wide, 2.8 mile length	2,910,835	2,711,925 (161,104 cy/yr (123,245 cu m/yr)
Jan 1965–Sep 1993	40-ft deep, 350-ft wide; Impoundment area along the western end of Petit Bois Island	1,305,589	14,772,517 (515,320 cy/yr) (394,220 cu m/yr)
Sep 1993–Nov 2005	44-ft deep, 550-ft wide; Impoundment area along the western end of Petit Bois Island	3,117,658	2,986,712 (245,483 cy/yr) (187,690 cu m/yr)
1897 to 2005	**Total Dredging**	**8,230,830**	**22,206,971 (205,600 cy/yr) (157,284 cu m/yr)**

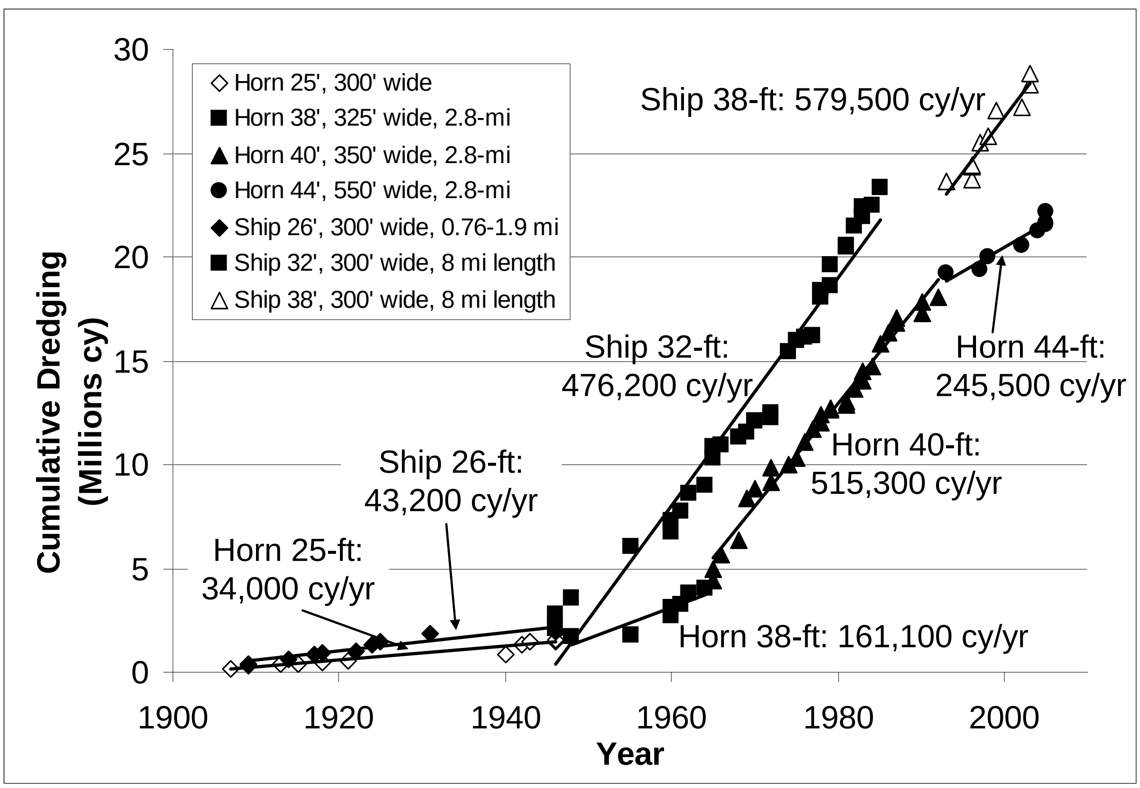


Figure 2.12-2. Cumulative maintenance dredging volumes and associated dredging rates for Horn Island Pass (Pascagoula Bar Channel) and Ship Island Pass (Gulfport Bar Channel)

These data indicate that deepening Ship Island Pass in 1948 by 23% (from 26 to 32 ft depth) and lengthening the channel (from 0.76 and 1.9 miles to 8 miles) increased the maintenance dredging rate by more than an order of magnitude (from 43,200 to 476,200 cy/yr). Dredging rates also increased more than an order of magnitude at Horn Island Pass through several depth increases from 25 to 40 ft, an increase in width from 325 to 350 ft, and length to 2.8 miles (dredging increased from 34,000 to 515,300 cy/yr). However, the dredging rate at Horn Island Pass decreased most recently when the channel was deepened to 44 ft and widened to 550 ft. This decrease in shoaling is opposite to what would be expected and possibly indicates a change in dredging or placement practices at Horn Island Pass. As these channels were deepened, they were also lengthened to provide safe navigation from a similar depth contour offshore. Thus, the deeper channels not only provided a better trap for sand moving alongshore but also resulted in longer channels which captured more of sand that is being transported in the offshore zone.

As mentioned previously, dredging for channels in the Sound do not modify the sediment budget for the barrier islands and mainland coast. The NDC's dredging database has been evaluated to provide a complete regional sediment budget as shown in Table 2.12-3.

Table 2.12-3.
Dredging Rates for Navigation Channels in Mississippi Sound (from SAM and NDC Database)

Location	Dates	Duration (years)	Shoaling Rate (cu yd/yr)	Notes
Gulfport Harbor Channel[1]	Jul 1991 – Sep 2004	8.3	1,151,000	Assume includes GIWW dredging
Biloxi Harbor Channel	Dec 1991 – Aug 2003	12.5	43,600	
Pascagoula Harbor Channel	Aug 1992 – Jan 2005	13.5	3,074,600	Assume includes GIWW dredging in vicinity of Pascagoula
Bayou Cassotte	Sep 1992 – Sep 2000	8	248,500	
Bayou La Batre	May 1996 – Sep 2004	8.3	732,400	Assume includes GIWW dredging

[1] Omitted Gulfport deepening in 1992.

2.12.4 Historical Data Analysis

A second phase of this study developed a historical sediment budget for the barrier islands and adjacent passes based on bathymetric change, shoreline position change, and dredging and placement data. The historical sediment budget is utilized to develop the present-day sediment budget. In this chapter, historical volumetric change, shoreline position change, and dredging data are reviewed. This portion of the study was conducted by Byrnes and Griffee (2007).

Shoreline and bathymetric data were compiled within a Geographic Information System (GIS) for the Mississippi Sound region. This database has associated metadata specifying the coordinate system, vertical datum, measurement units, and timing of data collection for each data set. Data are available for 1846/57, 1916/21, and 1960/71 periods, with coverage of the eastern portion of the study area available for 1984/89.

The primary goal of bathymetric change analysis is to identify regional sediment transport pathways and quantify net sediment volume changes associated with the historical evolution of nearshore morphology and adjacent beaches. Table 2.12-4 provides a summary of bathymetric data available for the Mississippi Sound area. Initial bathymetric surveys of the area were completed for the period 1847/56. All data have been compiled within a GIS framework, so metadata regarding coordinate system, vertical datum, measurement units, and timing of data collection are provided in the attribute table for each data set. These data, in addition to recorded shoreline changes, have been used to quantify regional sediment dynamics throughout the study area and evaluate the historical sediment budget for the period 1917/21 to 1960/71. Limited coverage offshore of Horn, Petit Bois, and Dauphin Islands for the 1960/71 period limits volumetric change calculations and, ultimately, the historical sediment budget.

Table 2.12-4.
Bathymetry Source Data Characteristics (from Byrnes and Griffee 2007)

Date	Data Source	Comments and Map Numbers
1847/56	USC&GS Hydrographic Sheets 1:20,000	First regional bathymetric survey within the study area. 1847 - H-00191; 1847/48 – H-00192; 1848 – H-00193, H-00194; 1851 – H-00256, H-00261; 1852 – H-00329; 1853 – H-00328, H-00365; 1854 – H-00430; 1855 – H-00485, H-00488, H-00489; 1856 – H-00546.
1916/20	USC&GS Hydrographic Sheets 1:40,000 (all others) 1:80,000 (H-4171)	Second regional bathymetric survey in the study area. 1916/17 – H-03960; 1917 – H-04000; 1917/18 – H-04020, H-04021, H-04023; 1920 - H-04171.
1960/71	USC&GS Hydrographic Sheets 1:10,000 (H-08524, H-08525, H-08560, H-08561, H-08562, H-08642, H-08643, H-08644, H-08645, H-08646, H-08649 to 08652, H-08922, H-08923, H-08925, H-08970, H-09156, H-09177) 1:20,000 (all others)	Third regional bathymetric survey in the study area. 1960 – H-08524, H-08525, H-08562, H-08563); 1960/61 - H-08560, H-08561; 1961 – H-08642; 1961/62 - H-8643 to 08648; 1962 – H-08649 to 08652; 1966/68 – H-08922, H08923; 1967/68 – H-08924, H-08925; 1968 – 08970, H-08971; 1968/69 – H-09004; 1970 – 09103, H-09109; H-09028, H-09156, H-09177; 1971 – H-09200.
1984/89	USC&GS Hydrographic Sheets 1:20,000 (D-00079, F-00324, H-10179, H-10208, H-10226, H-10247, H-10261) 1:40,000 (D-00078, H-10206) 1:80,000 (D-00065)	Survey covering eastern portion of the study area; 1984/87 - D-00065, D-00078; 1985/87 - H-10179; 1985 - H-10206, H-10208; 1986/88 – H-10226; 1987 - H-10247, H-10261; 1988 - D-00079; 1989 - F-00324.

Several insights into forcing processes and engineering activities were observed from the bathymetric change data.

(1) Overall, the barrier islands have eroded on the eastern regions and accreted to the west, indicating the dominant direction of longshore sand transport from east-to-west. Similarly, the Passes between barrier islands have also migrated to the west, as noted by the ebb shoal that erodes to the east and reforms to the west. Thus, the migrating barrier islands naturally “push” the Passes to the west.

(2) Dredging of the ship channels in Mississippi Sound is readily observed in the bathymetric change maps that include the 1960/71 surface, with side-casting and placement of the dredged material shown on either side of the channels. This side-cast sediment does not appear to move within Mississippi Sound.

(3) As the barrier islands have eroded, portions of the barriers have rolled over towards the Sound. For example, East Ship Island and western Dauphin Island have eroded on the Gulf side and reformed in a more northerly location further into the Sound. The processes transporting sand into the Sound is a combination of overwash during storms and inlet formation and possible subsequent closure.

(4) Portions of the barrier islands are relatively stable and maintain position through time (this is observed in Byrnes and Griffee's (2007) shoreline position data). Examples of these locations are the widest portions of Horn, Petit Bois, and Dauphin Islands. These areas are likely more stable ancient Pleistocene formations along which the sand spits which comprise the rest of the barrier island morphology form.

(5) Cat Island is not part of the sand-sharing system that comprises Dauphin, Petit Bois, Horn, and Ship Islands and the Passes that separate these barrier islands. Cat Island is a separate entity and the bathymetric change maps do not indicate that sand from Ship Island naturally bypasses or transports to Cat Island. If there were connectivity between Ship and Cat Island, it would be evidenced by erosion or accretion of morphologic features between the islands.

(6) From the historical shoreline position data (Byrnes and Griffee 2007), it is evident that the barrier islands have experienced cycles of breaching and mending throughout history. For example, Dauphin Island breached in 1917 in response to the 1915 hurricane, and reformed by 1957 slightly further northward (into the Sound) at the location of the washover deposit. Dauphin Island again shows a breach in the 2006 shoreline position data. Similarly, Ship Island breached in response to the 1947 hurricane and the barrier had reformed by 1950. Ship Island has been divided into East and West Ship Islands since another breach formed in the 1960s. These cycles of breaching and reformation indicate that breaches will naturally mend through the dominant longshore sand transport direction to the west, if a sufficient source of sediment is available. The historical data analysis is further discussed in Byrnes and Griffee (2007) and Rosati et al. (2007).

2.12.5 Numerical Modeling

Two numerical models were applied to develop estimates of sediment transport magnitudes and pathways. First, GENESIS shoreline change modeling that was conducted as a part of a larger regional study was incorporated to provide potential longshore sand transport rates for the Gulfside of the barrier islands for representative yearly waves. This model used pre-Katrina shoreline positions. Next, regional wave transformation modeling was conducted with STWAVE to estimate breaking wave height and direction magnitudes for the Gulfside and mainland coast beaches. These wave parameters and the shoreline orientation for sections of the Gulf barrier beaches and mainland coast were used to calculate potential longshore sand transport rates. Potential longshore sand transport rates are those estimated to occur if a sufficient quantity of sand were available for transport. Thus, these calculations do not apply to muddy coastlines or wetland regions of the study area. Finally, STWAVE was also applied to estimate wind-induced wave parameters for the Sound side of the barrier islands and subsequent sand transport on the Sound barrier coast. The methodology and results for this numerical modeling are discussed in Rosati et al. (2007).

2.12.6 Sediment Budget

Using the calculated and historical sediment budgets, and dredging and placement practices from 1993-2005 as presented by Rosati et al. (2007), a present-day (post-Katrina shoreline position) sediment budget has been hypothesized. In formulating this budget, several assumptions were made as follows:

(1) The historical sediment budget (1917/20-1960/71) was weighted more heavily than the calculated sediment budget, because the historical budget is based on actual measured changes in the region. However, for portions of the barrier islands that have changed morphology since the 1917/20 to 1960/71 period, or would be modified by a change in dredging or placement practices, the calculated sediment budget was given preference. The calculated sediment budget was adopted

for eastern Dauphin Island because volume change data have not yet been released, pending acceptance of the Dauphin Island mitigation study.

(2) In the absence of historical data, the calculated sediment budget and observed morphologic response were adopted for the mainland coast.

(3) Dredging and placement practices from 1993 to 2003/2005 were adopted for Ship Island Pass and Horn Island Pass, and the barrier island response to these activities was hypothesized. Dredging rates for Gulfport, Biloxi, and Pascagoula Harbor Channels, and Bayou Cassotte and Bayou La Batre were adopted as shown in Table 2.12-3. The source of sediment for these channels in Mississippi Sound was assumed to be fine-grained sediment that is mobilized during storms and wind events.

The hypothetical present-day sediment budget is shown in Figures 2.12-3 through 2.12-10, in which P=placement of dredged material, R=dredging or removal of sand, and sand fluxes are shown in thousands of cubic yards per year. It is emphasized that this sediment budget is only one of many possible solutions that could represent typical present-day conditions.

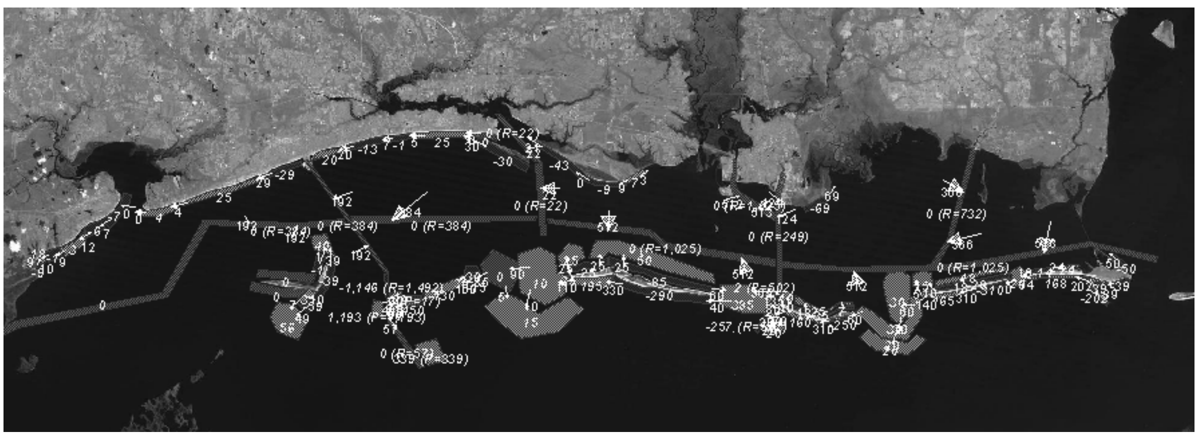


Figure 2.12-3. Overview of hypothetical present-day sediment budget (thousands of cy/yr)

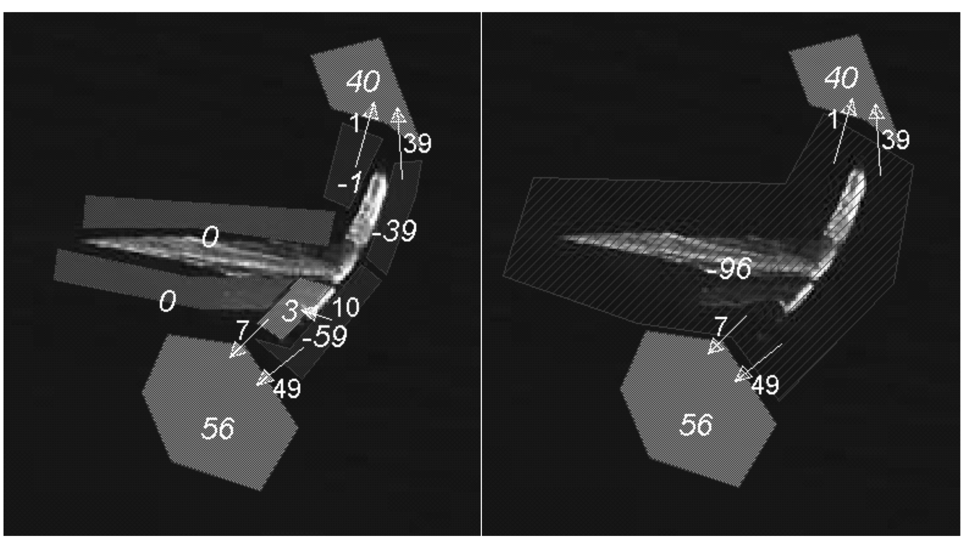


Figure 2.12-4. Hypothetical present-day sediment budget and macrobudget: Cat Island thousands of cy/yr).

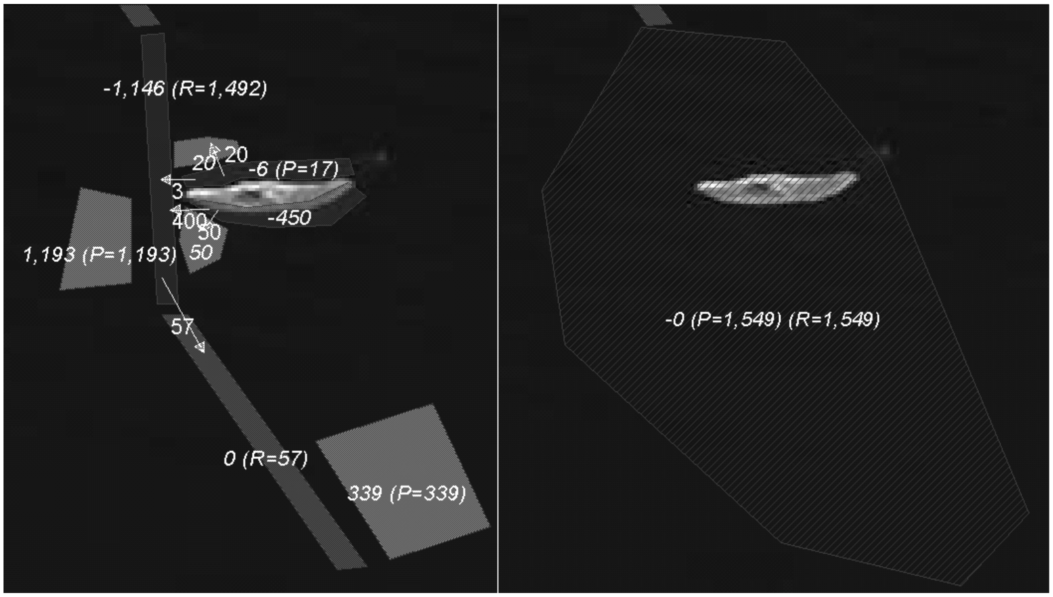


Figure 2.12-5. Hypothetical present-day sediment budget and macrobudget: West Ship Island and Ship Island Pass (thousands of cy/yr).

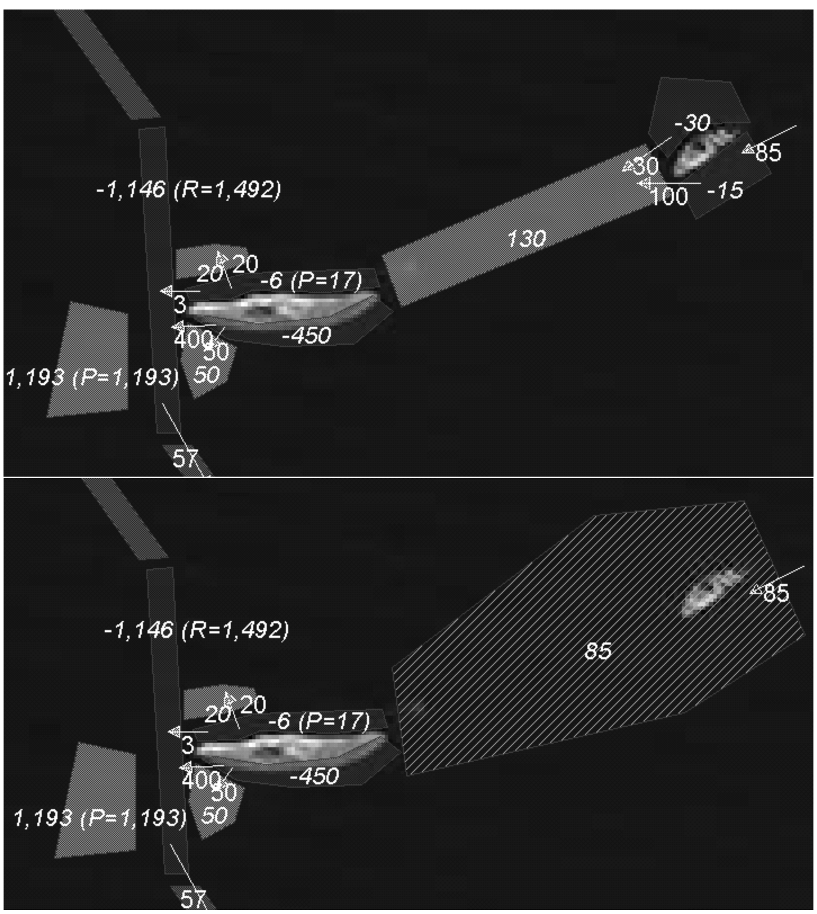


Figure 2.12-6. Hypothetical present-day sediment budget and macrobudget: East Ship Island and Camille Cut (thousands of cy/yr).

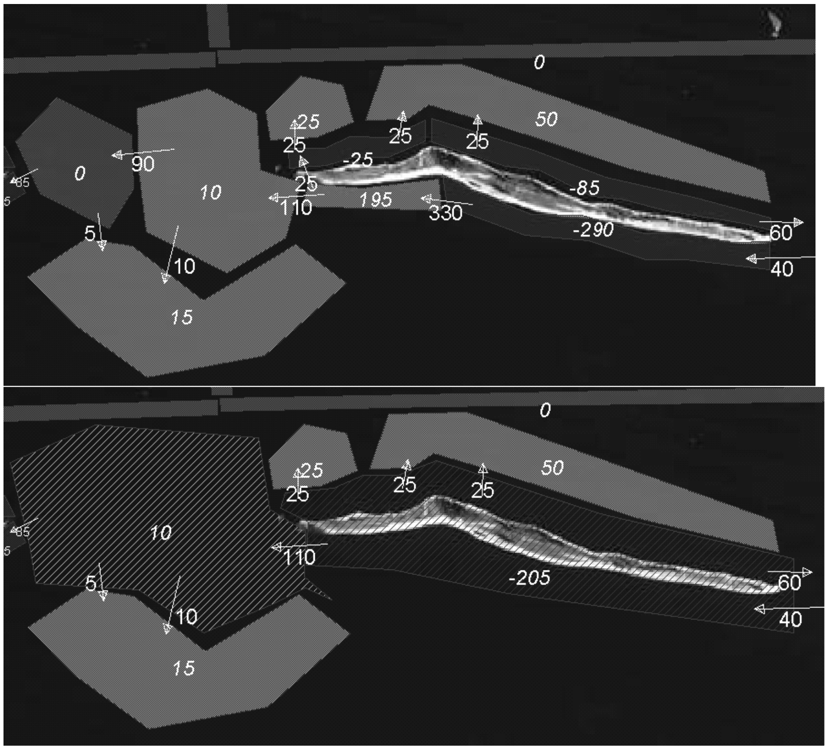


Figure 2.12-7. Hypothetical present-day sediment budget and macrobudget: Horn Island and Dog Keys Pass (thousands of cy/yr).

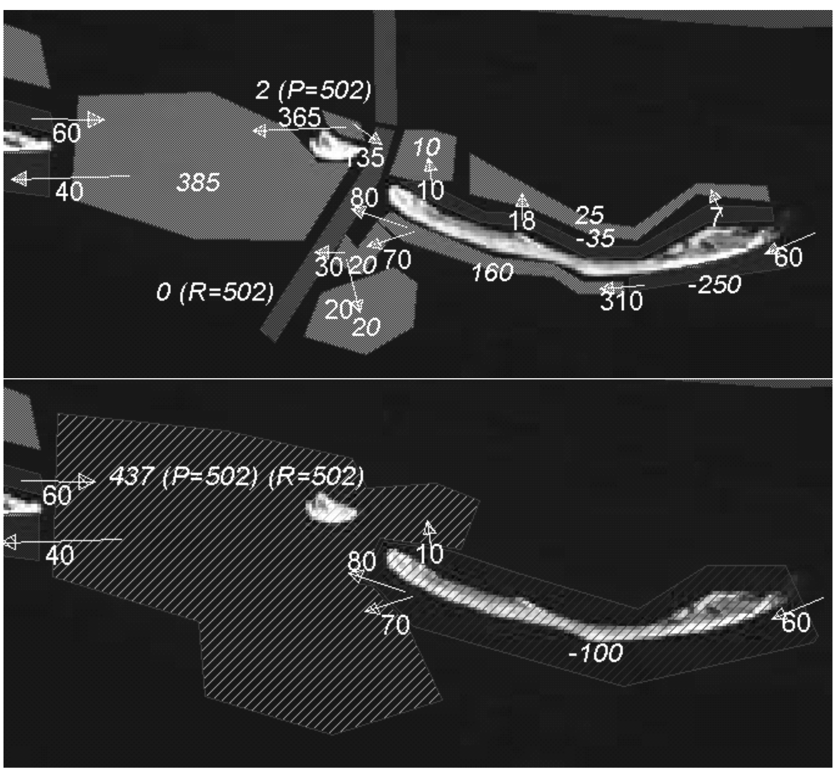


Figure 2.12-8. Hypothetical present-day sediment budget and acrobudget: Petit Bois Island and Horn Island Pass (thousands of cy/yr).

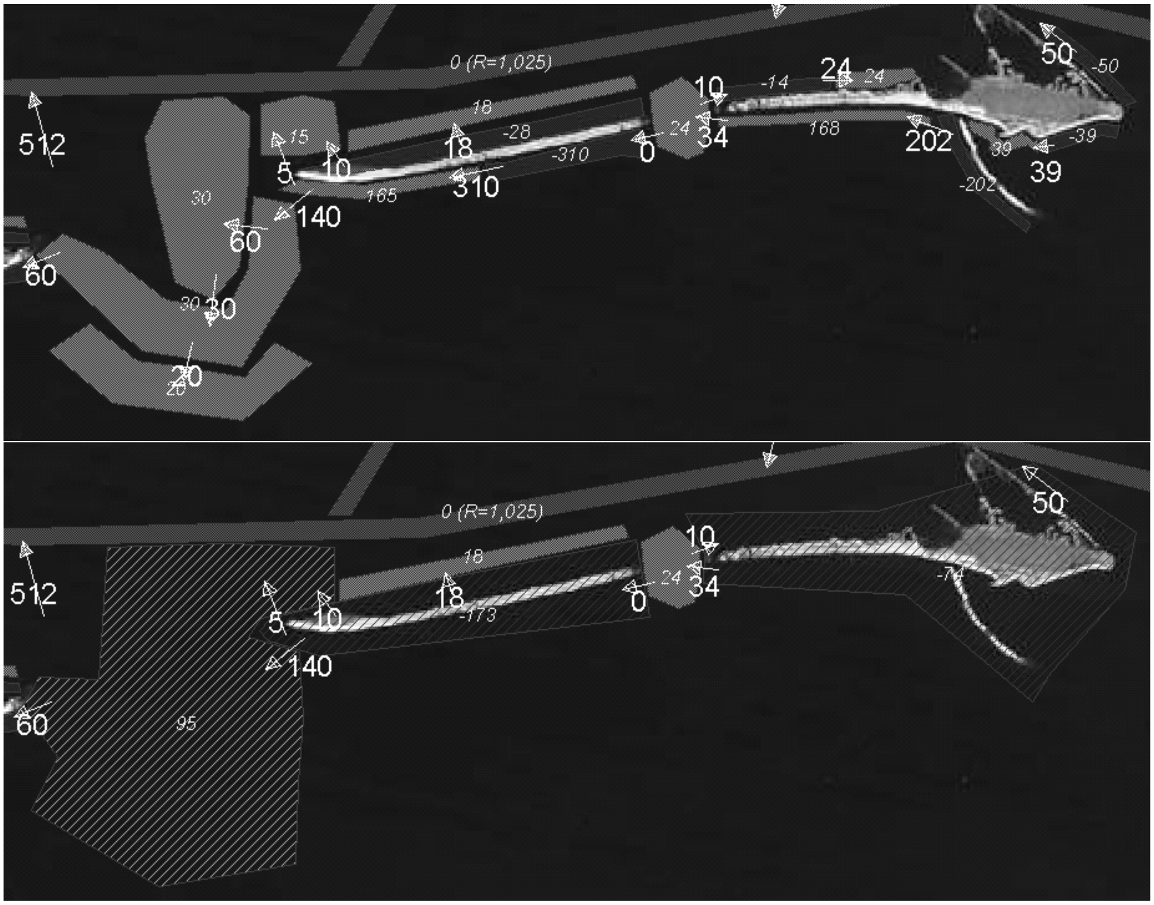


Figure 2.12-9. Hypothetical present-day sediment budget and macrobudget: Dauphin Island and Petit Bois Pass (thousands of cy/yr).

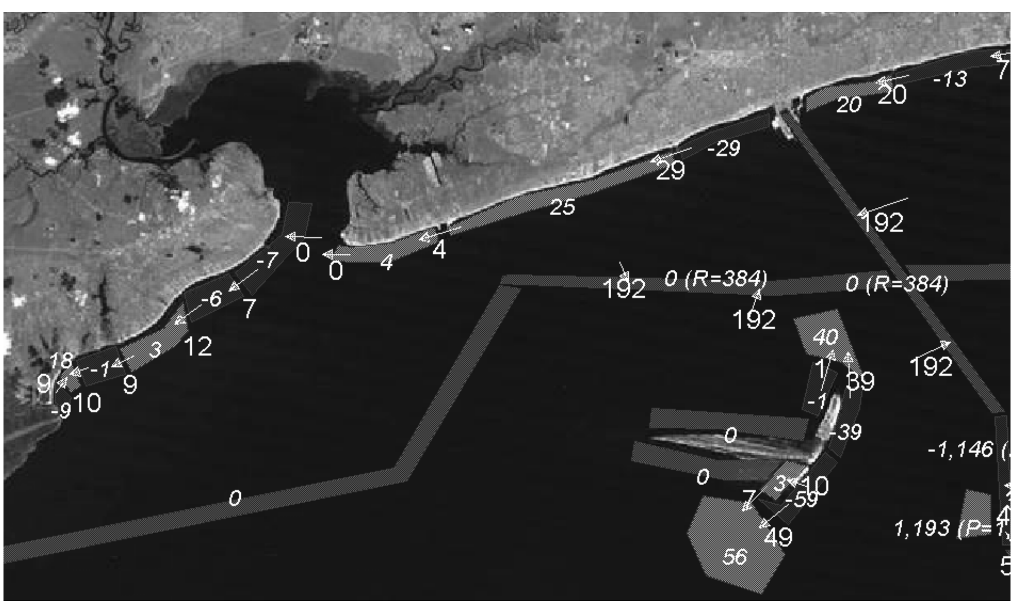


Figure 2.12-10. Hypothetical present-day sediment budget: Hancock County, Gulfport Harbor Channel, and a portion of the Gulf Intercoastal Waterway (thousands of cy/yr).

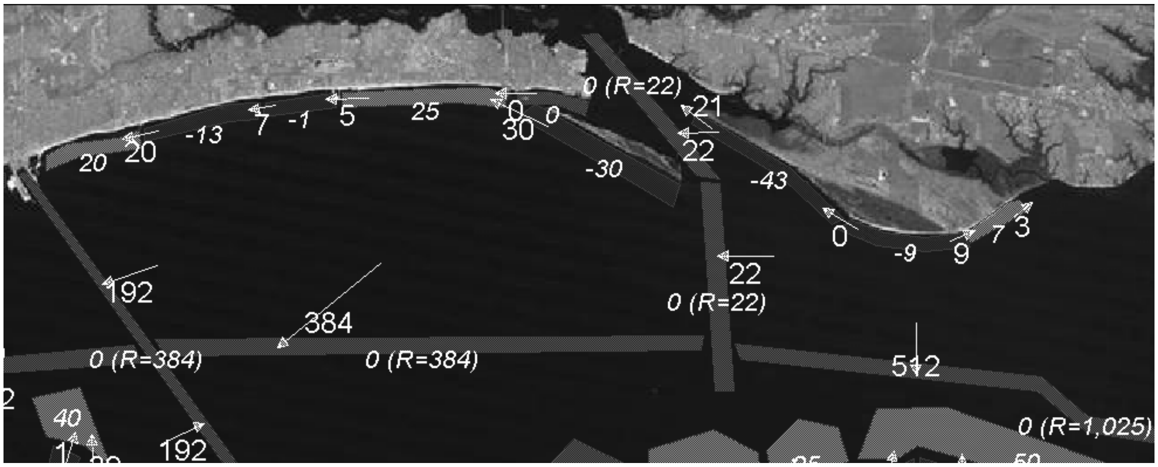


Figure 2.12-11. Hypothetical present-day sediment budget: Harrison County, Pascagoula Harbor Channel, and a portion of the Gulf Intercoastal Waterway (thousands of cy/yr).

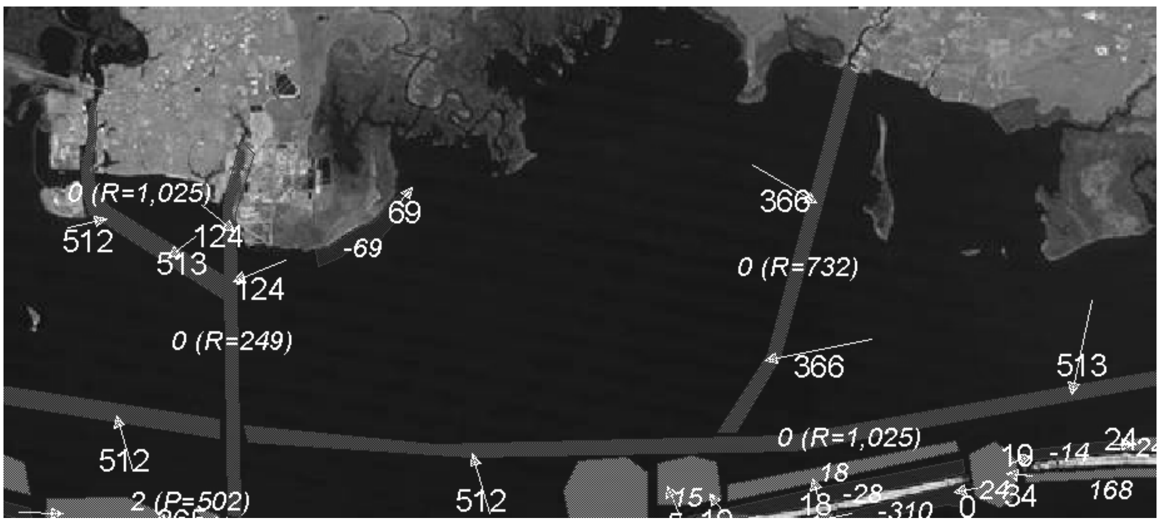


Figure 2.12-12. Hypothetical present-day sediment budget: Jackson County, Bayou La Batre, and a portion of the Gulf Intercoastal Waterway (thousands of cy/yr).

Knowledge gained through this study and recommendations that follow include the following:

(1) Cat Island is not a part of the barrier island littoral system represented by Dauphin, Petit Bois, Horn, and East and West Ship Islands. Cat Island is a separate morphologic feature that is naturally eroding due to waves, storm surge, and relative sea level rise in the region. Dredged sand that is placed in the littoral zone to the west of Ship Island Pass most likely will not be transported to Cat Island. Even in the absence of any engineering activities in Mississippi Sound, there is no evidence that sand from Ship Island would ever reach Cat Island.

(2) The net longshore sand transport rate for the barrier islands is from east-to-west. The barrier islands are migrating towards the west and, as they move west, also move the Passes between islands in a westerly direction. The source of sand for this region is the Mobile Pass ebb tidal shoal and the sandy shelf and shoreline to the east of Mobile Pass. Ship Island is the terminus of the longshore sand transport system in this region. Thus, the regional shortage of littoral sand will be most profoundly observed at Ship Island. We do observe disintegration of this barrier island, especially since Hurricane Katrina in 2005. We recommend that restoration of any barrier islands in

Mississippi Sound begin with Ship Island. In addition, we recommend back-passing sand dredged from Ship Island Pass, placing this sand either in Camille Cut, near East Ship Island, or in Dog Keys Pass. Sand can be placed in the surf zone (3 to 6-ft depths) and the natural longshore sand transport process will rebuild the island and begin to mend breaches.

(3) The historical sediment budget from 1917/20 to 1960/71 includes bathymetry change, shoreline position change, and dredging and placement practices representative of this period. However, data for the 1960/71 period are very sparse offshore of the barrier islands. This lends some uncertainty to the historical budget. In addition, Ship Island Pass and Horn Island Pass were deepened (and Horn Island was widened) in 1992/1993. Since that time, dredging rates have increased from those that occurred during the 1917/20 to 1960/71 period. Thus, the historical sediment budget is not representative of present-day dredging and placement activities, and has uncertainty with respect to bathymetric change offshore of the barrier islands. We recommend measurement of modern bathymetry (to 30 or 40-ft depths) and formulation of a sediment budget characterizing the period from 1917/20 (which has sufficient bathymetric coverage) to present-day.

(4) The historical analysis indicated that Horn Island has not experienced washover deposition across the entire island and has only been breached on a part of terminal spit during Hurricane Katrina (personal communication, Ms. Linda Lillycrop, May 2005). This cross-shore stability implies that the elevation and width of this barrier island might be a good template to evaluate for possible future restoration of the Mississippi Sound barrier islands.

(5) Wave modeling indicated that the mainland coast experiences a greatly reduced wave climate due to sheltering by the barrier islands fronting Mississippi Sound, as well as the Chandeleur Islands, and the Mississippi River's Bird's Foot delta. Restoration of the barrier islands could also consider lengthening the islands to recreate a previous historical footprint to provide additional wave protection for the mainland coast.

2.13 Flood Damage Analysis Model HEC-FDA

The Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) program uses risk-based analysis methods for evaluating flood damage and flood damage reduction alternatives. The program relies on hydrologic, hydraulic, and economic data input. Uncertainties in these data are input and used by the model for computing annual damages. The program's risk-based analysis methods conform to Corps of Engineers policy regulations (Ref. 1, 2) and technical procedures (Ref. 3).

Version 1.2.3b dated August 2007 was used. This is a customized version of the current official release version 1.2 dated March 2000. The official version computes uncertainty using the method of order statistics as described in ETL 1110-2-537. Because uncertainty distributions for the synthetic portion of the stage-frequency curves used for these evaluations were developed using methods different than order statistics, customization by the HEC was required in order to permit user-specified stage-frequency uncertainty as discussed in Section 2.13.2. Detailed model information is contained in the HEC-FDA User's Manual (Ref. 4).

This section of the Engineering Appendix deals primarily with the model's hydrologic and hydraulic input. The Economic appendix describes the economic input and results. The MsCIP Main Report describes how the model output was examined and used in the plan formulation process.

2.13.1 Model Overview

Generally, HEC-FDA computes flood damages for a given area by integrating the flooding source's annual stage-frequency curve with that area's structure inventory's stage-damage curve, resulting in a damage-frequency curve. The stage-frequency curve reflects the annual probability of a stage, or water surface elevation, being equaled or exceeded and the resulting damage-frequency curve represents the annual probability that a given dollar amount of damage will be equaled or exceeded. Uncertainty is accounted for by sampling the stage-frequency and stage-damage curves throughout their respective uncertainty ranges using an iterative numerical process called Monte Carlo simulation and the expected annual damage (otherwise known as the probability weighted average annual damage) with uncertainty is determined. Expected annual damage is the mean estimate of annual damage obtained from the resultant annual damage probability distribution. There are numerous permutations of economic output according to the plan (including the no-action plan and alternative plan(s)), year, and subject area. MsCIP economic input and output is discussed in detail within the economic appendix.

2.13.2 Methodology

HEC-FDA models were developed and model simulations were carefully constructed and executed to systematically evaluate flood damage risk and the effectiveness of various storm damage reduction measures, individually and combined, for reducing flood damage risk. All plans and measures were evaluated against a range of sea level rise scenarios (no rise, ‘expected’ rise, and ‘high’ rise).

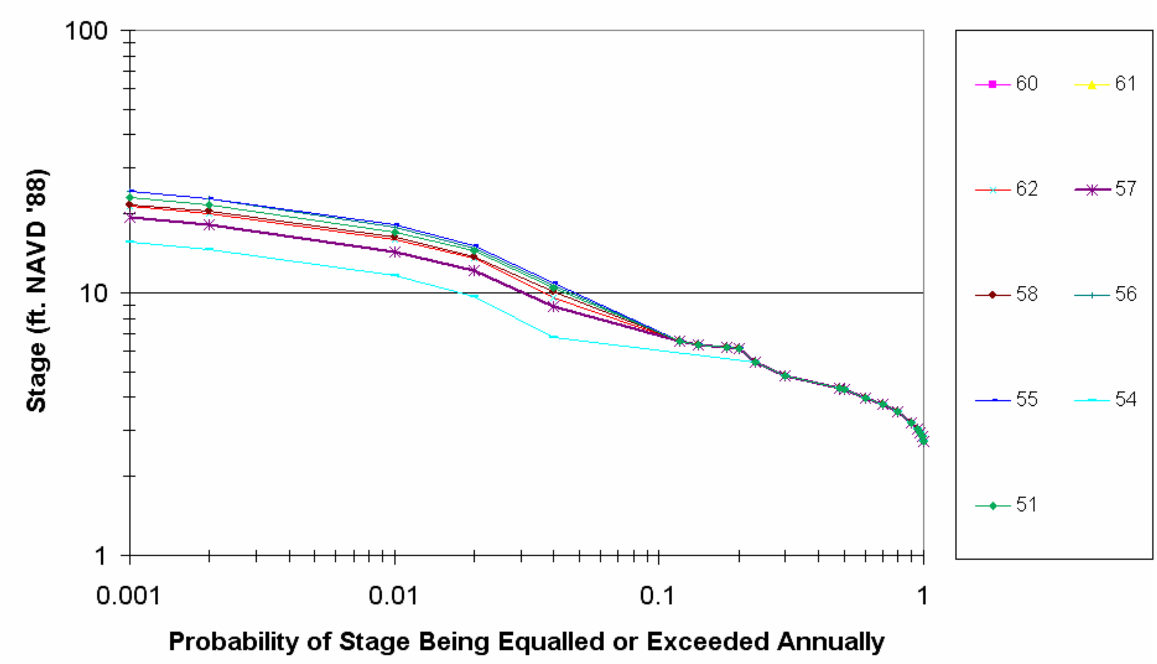
2.13.2.1 Planning Sub-units

FDA models were developed for each coastal Mississippi county: Hancock, Harrison, and Jackson counties). Each county represents a planning unit, and each was further delineated into planning sub-units (PSU). Each planning sub-unit is an HEC-FDA ‘damage reach.’ The planning subunits are shown in Figures 2.13-1 through 2.13-3. The planning units were numbered only for bookkeeping purposes. The PSU’s were extended inland to early (Fall 2006) estimates of the inland limit of surge-induced flooding and do not cover the entirety of each county. Political boundaries, source of flooding, topography, development density, potential surge inundation limits, and preliminary Lines of Defense (LOD) alignments were also considered when delineating the sub units. Where these factors did not clearly dictate where PSU boundaries might exist, or where the planning units were so large as to bring into question the whether the flooding threat for that planning unit might be reasonably approximated by a single stage-frequency curve, circa 2001 hurricane surge atlases (Ref. 5) were used to interpret where they might be located such that that PSU’s representative stage-frequency curve would be representative of the surge still water elevation to +/- 1 foot. There are ten PSU’s in Hancock County, 19 in Harrison County, and 26 in Jackson County.

2.13.2.2 Stage-Frequency Curve Overview

The source of flooding is Mississippi Sound with the primary cause being severe tropical system disturbance of that water body. Stage datum is the North American Vertical Datum of 1988 (NAVD ‘88). Stage-frequency curves were developed for each PSU from (a) long-term Mobile District tide gage data for Biloxi no. 02480350, Gulfport no. 02481341, and Pascagoula no. 02480301 through the 2005 calendar year; and (b) hydrodynamic simulation modeling conducted at the Engineer Research and Development Center (ERDC) in Vicksburg, MS. USACE tide gage records are discussed in Chapter 1.3, and the hydrodynamic modeling effort is described in Chapter 2. The USACE Gulfport, Biloxi, and Pascagoula tide locations correspond to hydrodynamic model ‘save points’ (a location for which detailed hydrodynamic output was obtained) 47, 15, and 14 respectively. The observed data were plotted at the median plotting position based on the period of record of each gage and graphically fit, while hydrodynamic plotting positions were determined by statistical analysis of the computed inundation surface as described by the save points; the resultant frequency curves were joined graphically resulting in a composite stage-frequency curve for each save point location. The observed data generally constitute that portion of each stage-frequency curve between the 99.9 percent and 25 percent annual chance exceedance stages, while the ERDC results composing the balance of the curves out to the 1 in 1000 annual chance event. Each stage-frequency curve then is composed partly of one of the three gage data sets, and partly from one ‘save point’ output file.

Figure 2.13-1. PSU’s and Save Points, Vicinity of Hancock County



Note: Save points referenced to USACE Gulfport gage.

Figure 2.13-2. Composite Stage-Frequency Curves, Hancock Co. Save Points

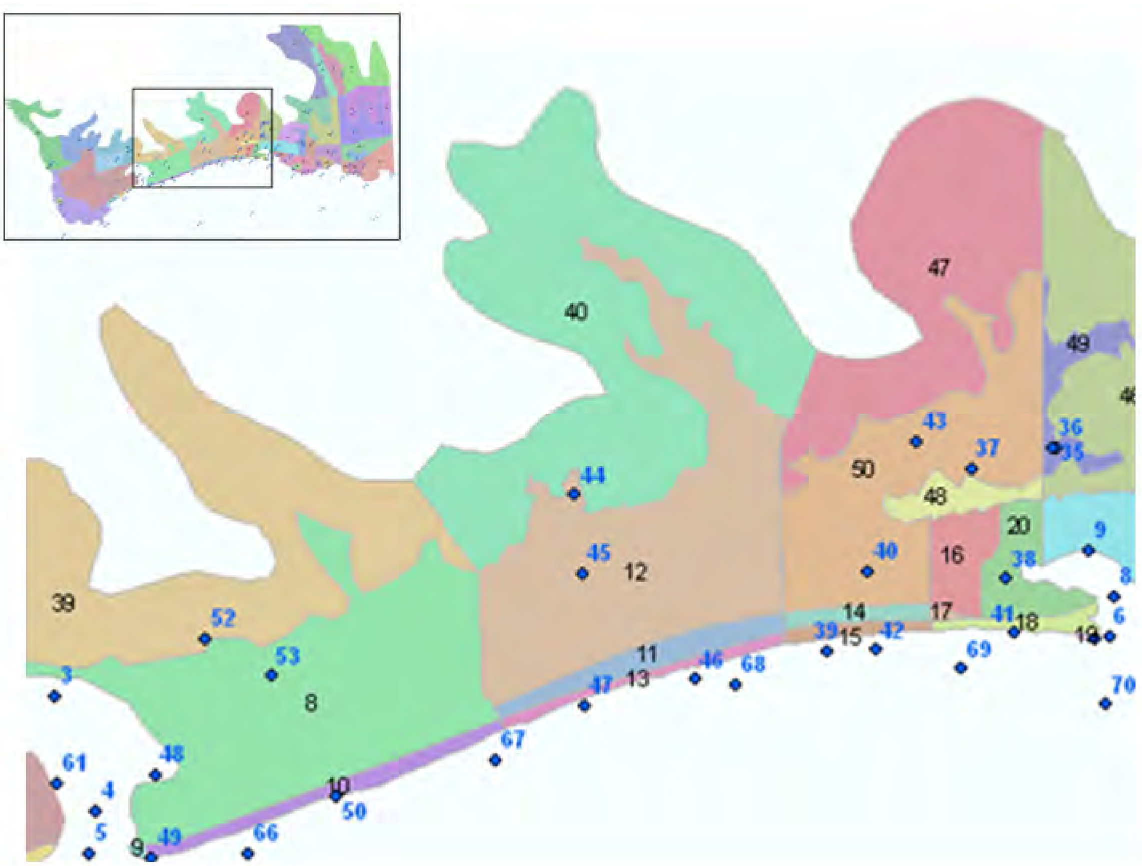


Figure 2.13-3. PSU's and Save Points, Vicinity of Harrison County

USACE technical guidance (Ref. 6) recommends the method of order statistics for computing uncertainty for graphical, synthetic, or composite frequency curves. Stage uncertainty at +/- 1 and +/- 2 standard deviations was computed by the method of order statistics using HEC-FDA Version 1.2 dated March 2000. Uncertainty for that portion of the curve described by hydrodynamic modeling results were estimated at one standard deviation by recently developed statistical methods described by Resio et. al. (Ref. 7). Uncertainties obtained by both methods were merged graphically. HEC-FDA Version 1.2 was modified by the HEC in order to allow direct input of these composite stage-frequency uncertainties.

The stage-frequency curves represent an estimate of the probability of a 'still water elevation' being equaled or exceeded in a calendar year. Waves are not explicitly accounted for in the stage-frequency curves; in other words, the observed or computed portions of each stage frequency curve have not been adjusted upward for surge-coincident wave amplitude[7]. This simplifying assumption is necessary for a variety of reasons, the most compelling being that (1) while participatory scientists agreed that existing shoreline and inundated area wave prediction methods are archaic and in need of revision, there is presently no regionally unified agreement as to how to treat them; (2) surge modeling evaluations do not provide for coincident unsteady freshwater inflow and (3) the level of effort required to develop representative wave information, particularly over inundated inhabited/developed areas where wave behavior is complex and highly variable, exceeded the scope and time available for this effort. While these issues may be overcome to various degrees by careful examination, such effort exceeds the scope and overall level of detail of this investigative phase. The impact of this assumption is that damages and benefits may be somewhat understated. The adopted method used does allow for a consistent evaluation between plans and is consistent with the overall level of study detail.

Each PSU's stage frequency curve was adjusted for future relative sea level rise as required by adding the computed sea level rise to the present stage for a given frequency. Adopted relative sea level rise predictions were derived from IPCC circa 2001 sea level rise predictions and are shown in Table 1.6-9.

The contribution of riverine or rainfall-runoff flood phenomena is not explicitly reflected in the FDA stage-frequency curves with the exception of tributary runoff and its contribution to Bay St. Louis and Biloxi Bay stage as a result of the surge barriers. This exception only applies for certain 'with-project' scenarios. That runoff should be neglected for FDA purposes at this stage of analysis is necessary one, given the relatively flat terrain and large number of sub-basins for which no previous hydrologic studies exist, and should not obscure coastal storm damage problems or opportunities. Should certain storm damage reduction measures be selected for further consideration, additional consideration will be given to quantifying the coincident nature of riverine flooding and coastal surge. Note that runoff has not been neglected in the conceptual design of the lines of defense, which provide dedicated water conveyances for the 24-hour, 25-year event rain event (no hurricane), and pumping for runoff coincident with hurricane storm events, as discussed in the Interior Drainage sections of Chapter 3.

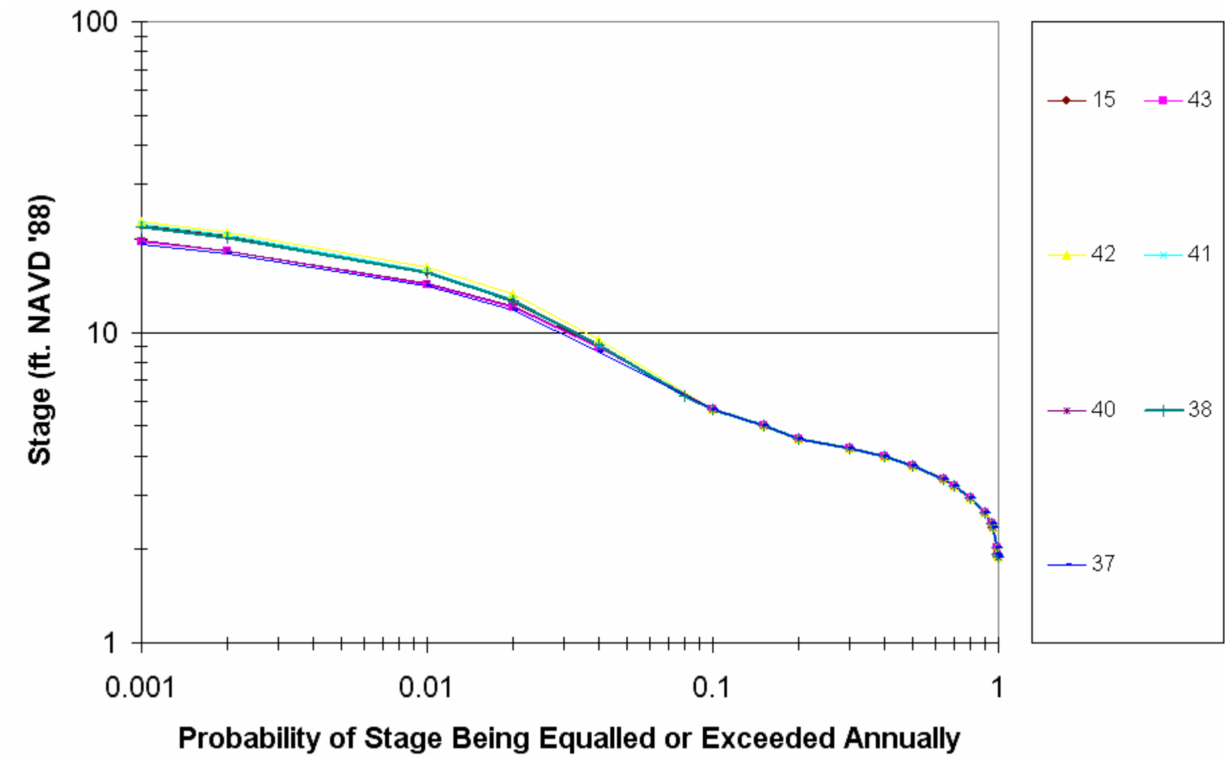
2.13.2.3 Assignment of Stage-Frequency Curves to Planning Sub Units

One stage-frequency curve is assigned to each Planning Sub Unit (PSU). Stage frequency curve components and assignments are displayed by Planning Unit (i.e. county) and PSU in Table 2.13-1. PSU (black numbering) and save point locations (blue numbering) are shown for Hancock, Harrison, and Jackson counties in figures 2.13-1, 2.13-3, and 2.13-6 respectively. Composite without-project stage-frequency curves are shown for Hancock County in Figure 2.13-2; for Harrison County in figures 2.13-4 and 2.13-5, and for Jackson County in figures 2.13-7, 2.13-8.

[7] The contribution to surge due to wave radiation stresses is accounted for as discussed in Chapter 2.

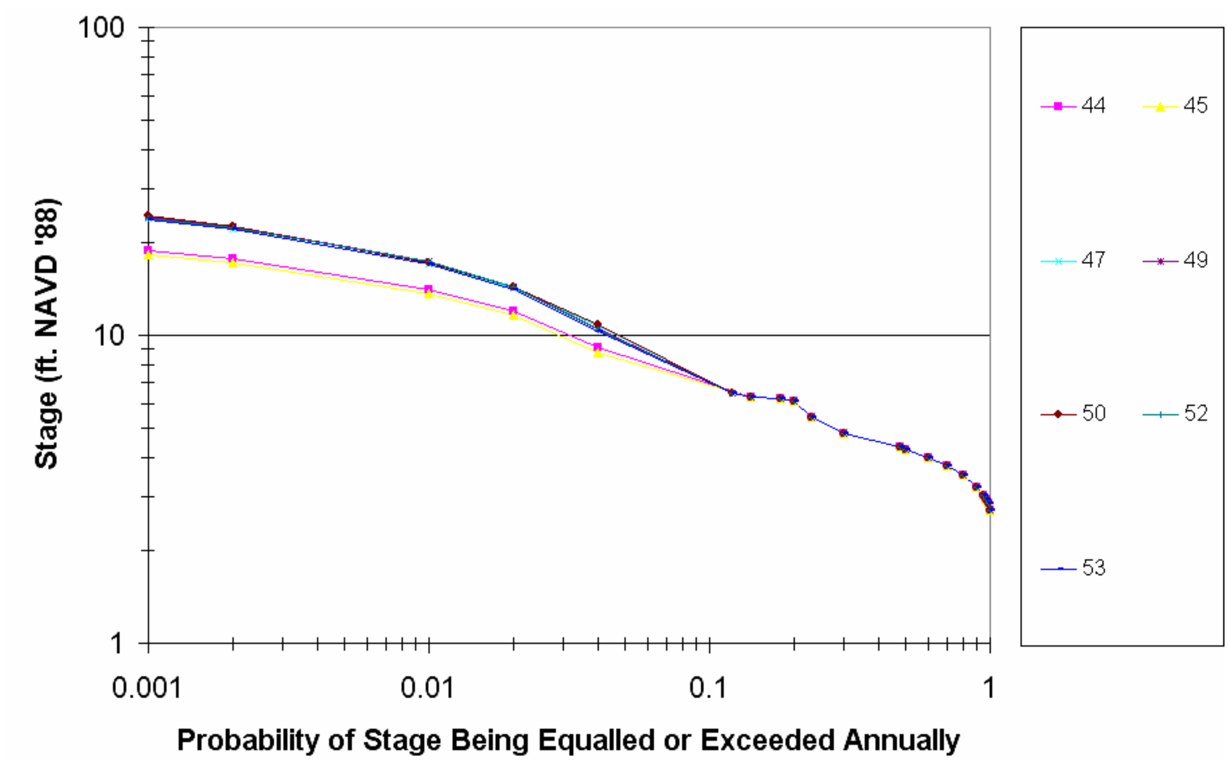
Table 2.13-1.
Stage-Frequency Curve Components and Assignments

County	PSU Number	Assigned Tide Gage	Assigned Save Point Number	County	PSU Number	Assigned Tide Gage	Assigned Save Point Number
Hancock County	5	Gulfport	62	**Jackson County**	21	Biloxi	9
	2	Gulfport	61		22	Biloxi	33
	1	Gulfport	60		23	Biloxi	29
	36	Gulfport	54		24	Biloxi	32
	6	Gulfport	62		25	Biloxi	25
	3	Gulfport	56		26	Biloxi	1
	4	Gulfport	55		27	Biloxi	30
	37	Gulfport	58		28	Biloxi	30
	7	Gulfport	57		29	Pascagoula	30
	38	Gulfport	51		30	Pascagoula	27
Harrison County	8	Gulfport	53		31	Pascagoula	26
	9	Gulfport	49		32	Pascagoula	23
	10	Gulfport	50		33	Pascagoula	17
	11	Gulfport	47		34	Pascagoula	18
	12	Gulfport	45		35	Pascagoula	20
	13	Gulfport	47		41	Biloxi	28
	14	Biloxi	42		42	Pascagoula	17
	15	Biloxi	42		43	Pascagoula	24
	16	Biloxi	38		44	Pascagoula	19
	17	Biloxi	42		45	Pascagoula	18
	18	Biloxi	41		46	Biloxi	35
	19	Biloxi	15		49	Biloxi	35
	20	Biloxi	38		51	Pascagoula	21
	39	Gulfport	52		52	Pascagoula	22
	40	Gulfport	44		53	Pascagoula	11
	47	Biloxi	43		54	Pascagoula	21
	48	Biloxi	37				
	50	Biloxi	40				



Note: Save points referenced to USACE Biloxi gage.

Figure 2.13-4. Composite Stage-Frequency Curves, Harrison Co. Save Points



Note: Save points referenced to USACE Gulfport gage.

Figure 2.13-5. Composite Stage-Frequency Curves, Harrison Co. Save Points (cont.)

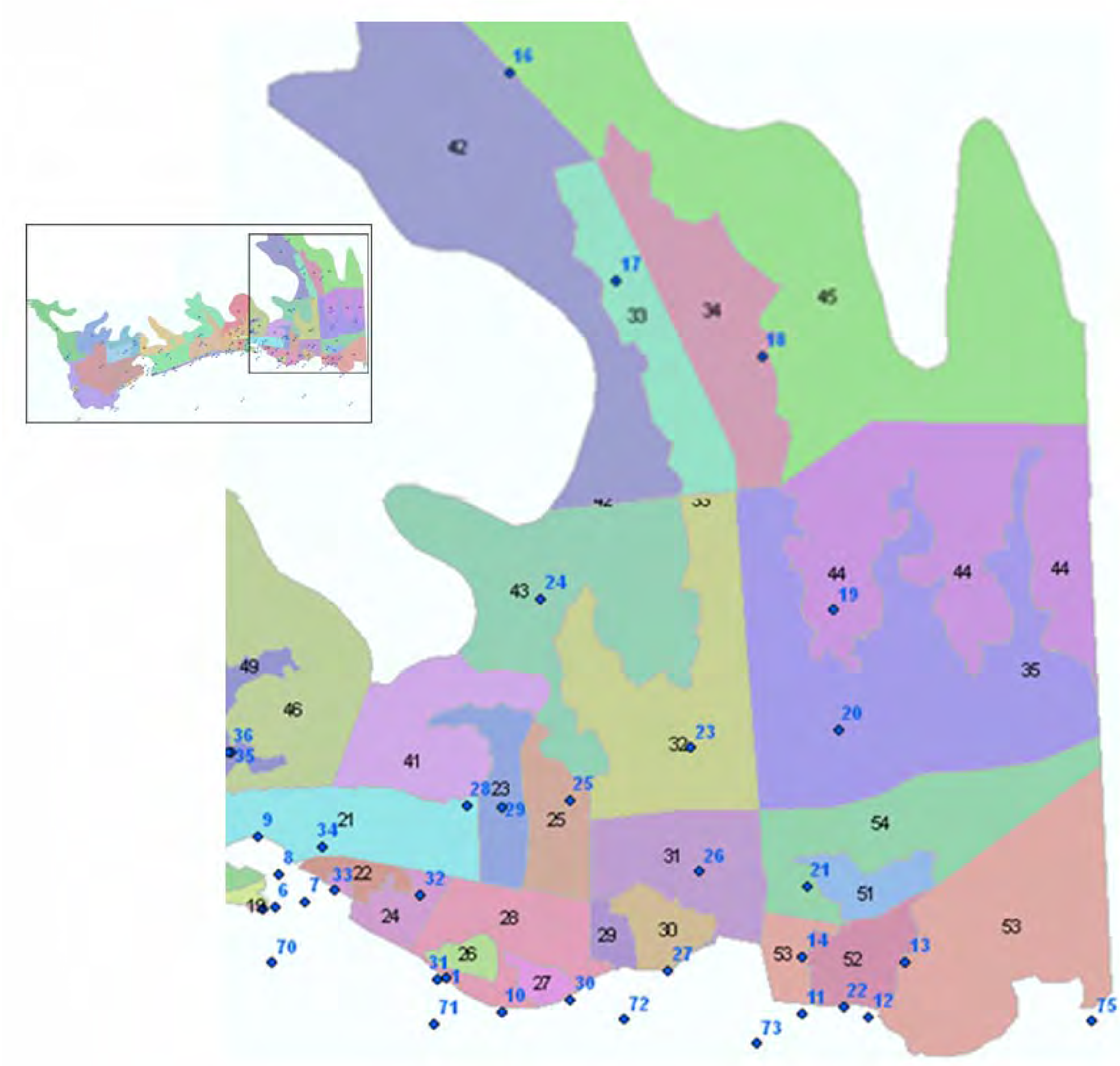
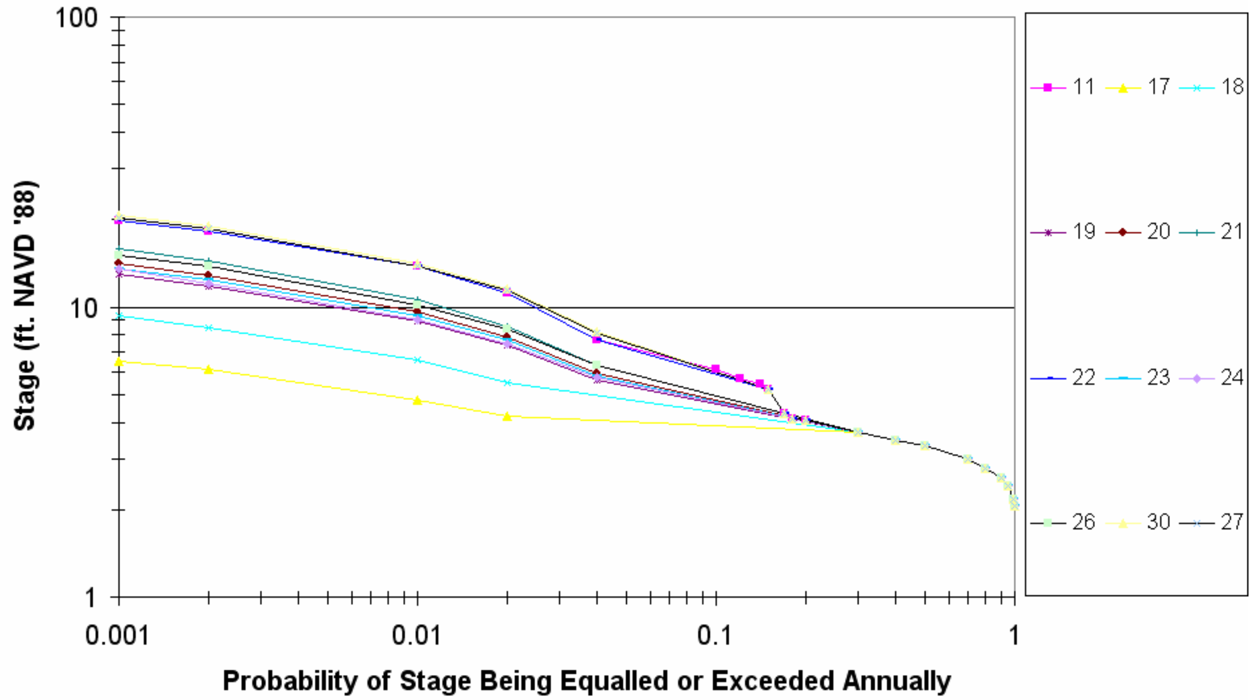
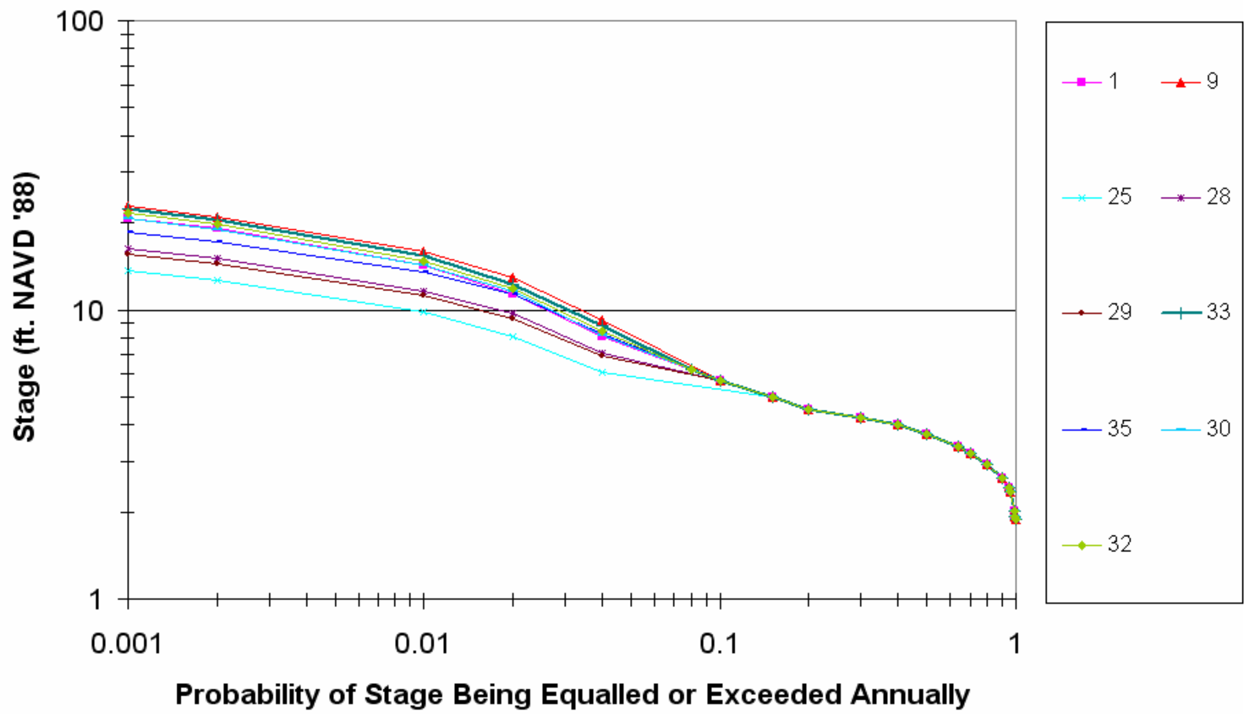


Figure 2.13-6. PSU's and Save Points, Vicinity of Jackson County



Note: Save points referenced to USACE Pascagoula gage.

Figure 2.13-7. Composite Stage-Frequency Curves, Jackson Co. Save Points



Note: Save points referenced to USACE Biloxi gage.

Figure 2.13-8. Composite Stage-Frequency Curves, Jackson Co. Save Points (cont.)

2.13.2.4 Scenarios

A number of scenarios, or plans, were evaluated. Scenarios include the existing condition; the future without-project; and the future with-project. The existing condition is the assumed condition for the base year 2012. The future without-project is an 'average' of future conditions over the project lifetime, which in this case is presumed to be 100 years. Existing condition and without-project evaluations differ only in their assumed structure inventories, which were varied to evaluate the sensitivity of computed damages to reconstruction patterns. Without-project conditions were also evaluated against 'expected' and 'high' sea level rise scenarios to test damage sensitivity to sea level rise uncertainty. Existing condition and without-project scenarios are shown in Table 2.13-2.

Table 2.13-2.
Existing Condition and Without-Project Scenario HEC-FDA Simulations

Run	County	Structure Inventory	MLFY (2111) Sea Level Scenario
1	Hancock	EC	Existing Sea Level
2	Harrison	EC	Existing Sea Level
3	Jackson	EC	Existing Sea Level
4	Hancock	Residential	Existing Sea Level
5	Hancock	Commercial/Condo	Existing Sea Level
6	Hancock	Residential	'Expected'
7	Hancock	Commercial/Condo	'Expected'
8	Hancock	Residential	High Sea Level Rise
9	Hancock	Commercial/Condo	High Sea Level Rise
10	Hancock	Residential	Existing Sea Level
11	Harrison	Commercial/Condo	Existing Sea Level
12	Harrison	Residential	'Expected'
13	Harrison	Commercial/Condo	'Expected'
14	Harrison	Residential	High Sea Level Rise
15	Harrison	Commercial/Condo	High Sea Level Rise
16	Harrison	Residential	Existing Sea Level
17	Jackson	Residential	'Expected'
18	Jackson	Residential	High Sea Level Rise

The future with-project condition represent an average of future conditions over the project lifetime with the incorporation of storm damage reduction measures, individually or in combination, in place. A number of model evaluations were structured in order to test the effectiveness of any one measure (e.g. Line of Defense 4) in the absence of all other measures for reducing without-project expected annual damages. The evaluations also provide for a cursory evaluation of measure performance with respect to uncertainty as to the rate of future sea level rise. The intent of structuring the model runs in this manner was to help identify measures that may warrant further consideration. Generally, the primary modeling differences between with-project conditions for a given planning unit rest (a) in the structural inventories attributed to the base year and most likely future year (MLFY); (b) the treatment of stage-frequency curves to reflect the presence of storm damage reduction measures; and (c) stage-frequency curve adjustments for sea level rise. Structure inventory assumptions are described in the Economics appendix.

With-project HEC-FDA evaluations are shown in Table 2.13-3. The simulations are grouped by county. Each measure is tested against a sea level rise scenario. The measures are assumed to remain 'as-built' (i.e. they are not significantly changed over their lifetime, and are not raised according to the sea level rise that is assumed). Additional measures (e.g. Menge Avenue, Hancock Co. ring levee, etc.) have since been added under this program but have not been fully developed for economic evaluations at press time for this document; future versions of this document will be revised to reflect evaluation of the additional measures.

Table 2.13-3.
HEC-FDA Individual Measures Scenario Simulations

Run	County	Measure	MLFY (2111) Sea Level Scenario
1	Hancock	LOD4-20'	Existing Sea Level
2	Hancock	LOD4-30'	Existing Sea Level
3	Hancock	LOD4-40'	Existing Sea Level
4	Hancock	Seawall	Existing Sea Level
5	Hancock	Pearlington 20'	Existing Sea Level
6	Hancock	Pearlington 30'	Existing Sea Level
7	Hancock	Nonstruct 1	Existing Sea Level
8	Hancock	Nonstruct 2	Existing Sea Level
9	Hancock	SHORELINE	Existing Sea Level
10	Hancock	LOD4-20'	‘Expected’ sea level
11	Hancock	LOD4-30'	‘Expected’ sea level
12	Hancock	LOD4-40'	‘Expected’ sea level
13	Hancock	Seawall	‘Expected’ sea level
14	Hancock	Pearington 20'	‘Expected’ sea level
15	Hancock	Pearlington 30'	‘Expected’ sea level
16	Hancock	Nonstruct 1	‘Expected’ sea level
17	Hancock	Nonstruct 2	‘Expected’ sea level
18	Hancock	Shoreline	‘Expected’ sea level
19	Hancock	LOD4-20'	‘High’ sea level
20	Hancock	LOD4-30'	‘High’ sea level
21	Hancock	LOD4-40'	‘High’ sea level
22	Hancock	Seawall	‘High’ sea level
23	Hancock	Pearington 20'	‘High’ sea level
24	Hancock	Pearlington 30'	‘High’ sea level
25	Hancock	Nonstruct 1	‘High’ sea level
26	Hancock	Nonstruct 2	‘High’ sea level
27	Hancock	Shoreline	‘High’ sea level
28	Harrison	LOD4-20'	Existing Sea Level
29	Harrison	LOD4-30'	Existing Sea Level
30	Harrison	LOD4-40'	Existing Sea Level
31	Harrison	Seawall	Existing Sea Level
32	Harrison	Nonstruct 1	Existing Sea Level
33	Harrison	Nonstruct 2	Existing Sea Level
34	Harrison	Turkey Creek	Existing Sea Level
35	Harrison	LOD4-20'	‘Expected’ sea level
36	Harrison	LOD4-30'	‘Expected’ sea level
37	Harrison	LOD4-40'	‘Expected’ sea level
38	Harrison	Seawall	‘Expected’ sea level
39	Harrison	Nonstruct 1	‘Expected’ sea level
40	Harrison	Nonstruct 2	‘Expected’ sea level
41	Harrison	Turkey Ck.	‘Expected’ sea level
42	Harrison	LOD4-20'	‘High’ sea level
43	Harrison	LOD4-30'	‘High’ sea level
44	Harrison	LOD4-40'	‘High’ sea level
45	Harrison	Seawall	‘High’ sea level
46	Harrison	Nonstruct 1	‘High’ sea level
47	Harrison	Nonstruct 2	‘High’ sea level
48	Harrison	Turkey Ck.	‘High’ sea level
49	Jackson	LOD4-20'	Existing Sea Level
50	Jackson	LOD4-30'	Existing Sea Level
51	Jackson	LOD4-40'	Existing Sea Level
52	Jackson	Seawall	Existing Sea Level
53	Jackson	Ring Dike 20'	Existing Sea Level
54	Jackson	Ring Dike 30'	Existing Sea Level
55	Jackson	Nonstruct 1	Existing Sea Level
56	Jackson	Nonstruct 2	Existing Sea Level
57	Jackson	Bayou Cumbest	Existing Sea Level
58	Jackson	LOD4-20'	‘Expected’ sea level
59	Jackson	LOD4-30'	‘Expected’ sea level
60	Jackson	LOD4-40'	‘Expected’ sea level
61	Jackson	Seawall	‘Expected’ sea level
62	Jackson	Ring Dike 20'	‘Expected’ sea level
63	Jackson	Ring Dike 30'	‘Expected’ sea level
64	Jackson	Nonstruct 1	‘Expected’ sea level
65	Jackson	Nonstruct 2	‘Expected’ sea level
66	Jackson	Bayou Cumbest	Existing Sea Level
67	Jackson	LOD4-20'	‘High’ sea level
68	Jackson	LOD4-30'	‘High’ sea level
69	Jackson	LOD4-40'	‘High’ sea level
70	Jackson	Seawall	‘High’ sea level
71	Jackson	Ring Dike 20'	‘High’ sea level
72	Jackson	Ring Dike 30'	‘High’ sea level
73	Jackson	Nonstruct 1	‘High’ sea level
74	Jackson	Nonstruct 2	‘High’ sea level
75	Jackson	Bayou Cumbest	Existing Sea Level

2.13.2.5 Scenario Stage-Frequency Curves

As discussed previously, the stage-frequency curves used in HEC-FDA analyses were composed from observed tide gage data and from hydrodynamic modeling results. In support of the HEC-FDA effort, the hydrodynamic modeling evaluated the existing condition; Line of Defense (LOD) 4 alone; and LOD 3 plus ring levees. Additional simulations could not be accomplished for this program. The LOD's and ring dikes were coded into the hydrodynamic terrain database of their respective hydrodynamic models as 'infinitely high walls', thus obtaining an estimate of the height of the LOD required to contain a given annual chance event from model output nodes located seaward of each LOD. LOD's 3 and 4 are discussed in detail in Chapter 3.

Existing condition stage-frequency curves (adjusted for sea level rise as required) were used for all damage reaches in many of the measure evaluations listed in Table 2.13-3.

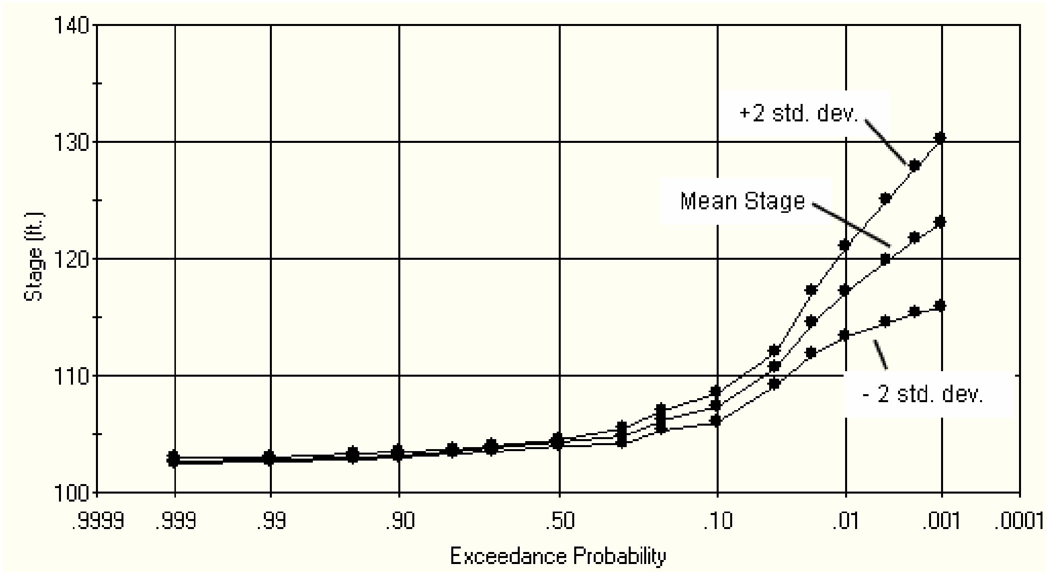
For future with-project HEC-FDA evaluations involving individual measures, stage-frequency curve development required use of either existing condition hydrodynamic model output, or LOD hydrodynamic model output, or both, depending upon the location of both the PSU and that PSU's hydrodynamic model save point with respect to the line of defense. Additional consideration was required if the PSU was subject to induced runoff storage due to Bay St. Louis or Biloxi Back Bay closure structures. The following paragraphs describe the methodology used to develop stage frequency curves for the with-project scenarios involving individual measures.

- *Save Point and PSU behind levee or ring-dike.* For example, consider PSU 7 and its representative save point (57) in Hancock County as shown in Figure 2.13-1. This PSU is located behind both of the conceptual LOD3 and 4 alignments. This case is typical of many inland PSU's that do not border Biloxi Bay or Bay St. Louis. For this circumstance, the existing condition stage-frequency curve is used and PSU is coded into HEC-FDA as being protected to the prescribed LOD crest elevation. The existing condition stage-frequency curve is used because the with-LOD save point is essentially 'dry' due to the infinite wall approach in the hydrodynamic modeling. The interior water surface elevation is assumed to equal the stage-frequency curve stage for all events exceeding the levee crest elevation.
- *Save Point in front of levee and PSU behind levee or ring dike.* Consider PSU 11 and its representative save point (47) in Harrison County as shown in Figure 2.13-3. This PSU is also located behind the prospective LOD 3 and 4 alignments, but it's representative save point is outside (seaward) of the LOD's. This case is typical of nearshore and ring-levee PSU's. The stage-frequency curve was developed from observed gage data and the appropriate ERDC LOD model output and coded into HEC-FDA as being protected by a levee to the prescribed LOD crest elevation. As with the previous case, the interior water surface elevation is assumed to equal the stage-frequency curve stage for all events exceeding the levee crest elevation.
- *Save Point and PSU behind levee and PSU is subject to induced stage due to closure structures across bays.* As with the '*Save Point and PSU behind LOD'* situation, this situation also utilizes the existing conditions stage-frequency curve but the HEC-FDA levee routine cannot be invoked because streamflow into Bay St. Louis and Biloxi Bay will cause their water surface elevations to rise when the surge barriers are closed to prevent hurricane surge inundation. Existing condition frequency curves (e.g. Figure 2.13-9) were transformed (e.g. Figure 2.13-10 for the LOD4 at 20' crest elevation) graphically using the following assumptions:
    - o The surge barriers would be operated for extreme hurricanes only, on average once every twenty years (e.g. hurricanes resembling H. Katrina, H. Betsy, H. Camille, or the 1947 or 1915 hurricanes etc.)

- o Closure of the barriers would result in Bay St. Louis and Biloxi Bay water surface elevations rising to elevation 6.8 ft. and 8.4 ft NAVD ’88, respectively based on preliminary hydrologic analyses (see Chapter 3.4).
- o Should the crest elevation of the barrier be such that it overtopped, the representative existing condition stage would rapidly be attained.

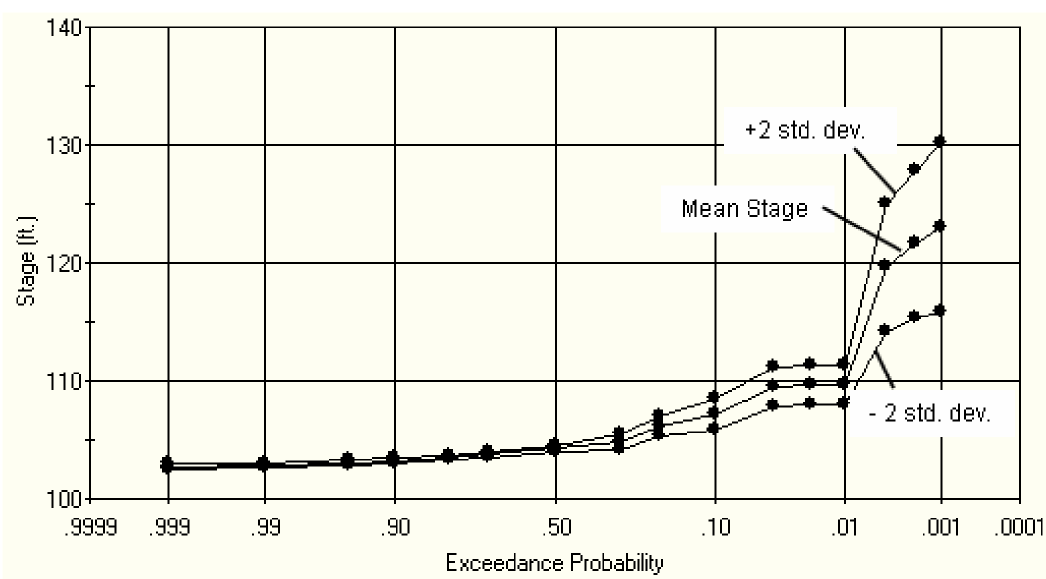
As mentioned previously, Most Likely Future Year (MLFY) stage frequency curves were adjusted for future sea level rise by adding the predicted rise to stage for a given frequency.

Flood damage evaluation results are reported in the Economics Appendix.



Note: 100 ft. added to stage for HEC-FDA computational purposes.

Figure 2.13-9. Existing Condition Frequency Curve, PSU 1 Save Point 60, Hancock Co.



Note: 100 ft. added to stage for HEC-FDA computational purposes.

Figure 2.13-10. Transformed Frequency Curve, PSU 1 Save Point 60, Hancock Co.

2.13.3 References

USACE (2000). Planning Guidance Notebook. Engineer Regulation ER 1105-2-100. Department of the Army, US Army Corps of Engineers, Washington, D.C. 22 April 2000.

USACE (2006). Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

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USACE (1997). Uncertainty Estimates for Nonanalytic Frequency Curves. Engineering Technical Letter ETL 1110-2-537. US Army Corps of Engineers. Washington, DC. 31 October 1997.

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2.14 Lines of Defense Crest Elevation Analyses

Evaluations were conducted to estimate the required levee crest elevation in order to provide protection to the one in one hundred (1%), one in five hundred (0.2%), and one in one thousand (0.1%) annual chance exceedance surge events.

Typical levee and elevated roadway sections are shown in Figures 2.14-1 and 2.14-2. The crest elevations shown on the typical levee section were used to generate levee cost-height curves, and should not be interpreted to be design elevations. Design elevations follow from performance and cost effectiveness objectives as discussed in the main report.

Resultant crest elevations are a function of the surge still water elevation, wave height, wave period, levee slope, levee surface roughness, nearshore depth, nearshore slope, and the seaward levee side slope. Still water elevations for the given events were obtained for the nearest representative save points using the same ERDC results comprising the synthetic portions of the composite stage frequency curves. ERDC also provided mean estimates of the 1%, 0.2%, and 0.1% significant wave height (Hs) and peak period (Tp) estimates based on statistical analyses of with-project (either LOD3 or LOD4) STWAVE model results. Wave characteristics were computed independently of surge characteristics. Surge elevation, Hs, and Tp determinations are given in Chapter 2.9 of this appendix. Nearshore depth and geometry was estimated from a limited number of beach profiles (i.e. sections normal to the shoreline) obtained by Mobile District; and/or from interpretation of existing USGS topographic maps; and/or from interpretation of post-Katrina, LIDAR-derived topography. Levee side-slopes were assumed to be 3 to 1 (horizontal to vertical) with a mowed grass face except as noted in following paragraphs.

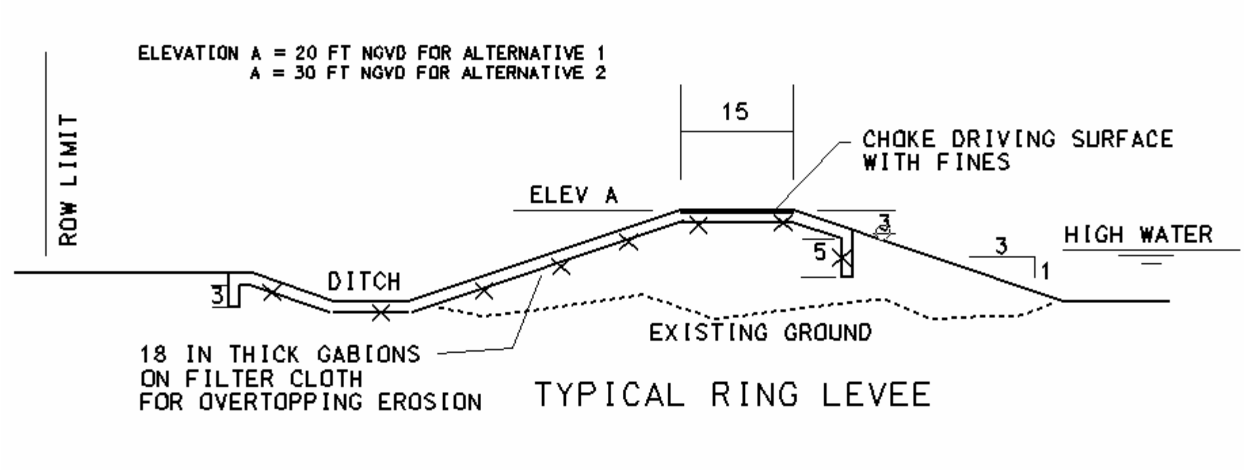
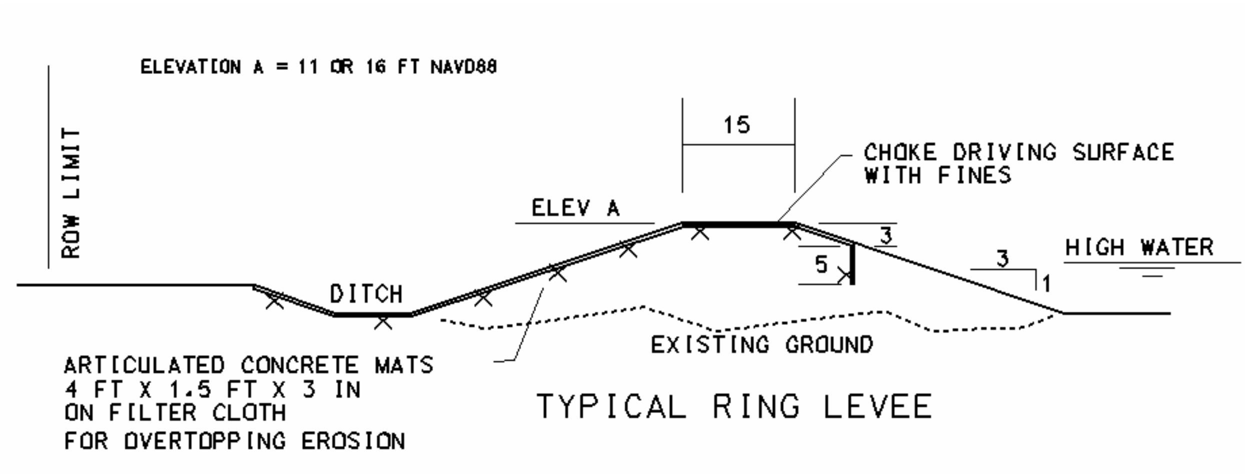


Figure 2.14-1. Typical Section, Levees



Source: ERDC, Steven Hughes

Figure 2.14-2. Typical Section, Elevated Roadway

Computations generally followed those prescribed in a recent, though draft, USACE Technical Letter (Ref. 1). Crest elevations for a given event assumed coincident occurrence of percent chance event surge and wave characteristics; in other words, the crest elevation computations for the 1% event assumed coincident occurrence of the mean 1% Hs, 1%Tp, and 1% surge elevations. Because Tp and Hs percentiles were computed independently of surge elevations, this assumption is thought to yield somewhat conservative results (Ref. 1). Computations were performed using Table VI-5-11 (van der Meer and Janssen's equation) of the Coastal Engineering Manual (CEM) Professional Edition software, version 2.0.1.1. All gamma factors in the underlying equation were assigned a value of unity. The presence of other potentially complementary project features, such as sand dunes and berms, was neglected.

Adequate protection was defined for these preliminary purposes as the crest elevation for which the computed average overtopping rate for each event was on the order of 0.01 cubic feet per second per foot (cfs/ft), which is equivalent to an average overtopping discharge rate of 10 cfs per 1,000 feet of levee. This rate is less than the 0.1 cfs/ft which is currently being considered as an appropriate threshold for well-designed and constructed coastal levee defenses, but this higher rate is best applied in conjunction with conditional probability methods; the lower rate assumed herein was judged appropriate for these preliminary purposes given the limited spatial extents of model output and the deterministic methods otherwise employed.

The information available for these computations allows for just a few spot estimates of requisite levee elevations. Those elevations presented in following paragraphs should be understood to be applicable to a discreet location and not interpreted to describe the crest elevation of any particular line of defense throughout its entire length. Should a levee or other structural line of defense measure be selected for additional investigation, a much higher density of well positioned surge, wave, and geometric information would be needed in order to adequately define the required levee profile along its length for the desired level of performance. These computations are deterministic thought in most instances the results are expected to agree reasonably with results determined through a probabilistic conditional probability simulation.

2.14.1 Line of Defense 3

Computed crest elevations for locations along LOD 3 are given in Table 2.14-1. Crest elevations given are reported in feet NAVD ’88 datum. Locations at which the elevations were computed are shown in Figures 2.14-3 through 2.14-11. Computed crest elevations range from elevation 13 feet to 53 feet over the range events. For the one in 100 chance events, computed crest elevations range from 13 to 37 feet, with most locations yielding elevations in the high teens to mid twenties. In some instance, such as the Harrison County elevated roadway, the computed elevation is on the order of 36 feet. Locations that yield these types of results would be inherently difficult to defend due to large surge depths, severe storm wave climates, and the absence of a shallow foreshore. The typical levee section is inappropriate where results such as these arise, which would probably be better defended perhaps by a levee with some or all of the following features: (a) a frontal berm seaward of the primary levee prism; (b) a flatter seaward slope (6 to 1 or greater); and (c) a roughened, hardened slope in lieu of grass. Such features would reduce the required height at a given location for a given event overtopping rate. Where possible, the levee height may also be reduced by removing the structure landward from the shoreline to an upslope location beyond the wave breaking zone. For example, the results in Table 2.14-1 suggest that the levee crest elevation at Pascagoula might be reduced from 37 feet to 19 feet by removing the levee from the shoreline to an alignment in the vicinity of Washington Avenue. Similar findings and recommendations apply when interpreting LOD 4 levee performance and attributes.

Table 2.14-1.
Computed Structure Crest Elevations, LOD 3

Feature and Location	Annual Event Chance		
LOD-3	**1 in 100**	**1 in 500**	**1 in 1000**
Hancock County			
Pearlington Levee	20	30	34
Bay St Louis Levee	22	38	42
Hancock Elevated Road, 11.0	N/A[1/]	N/A[1/]	N/A[1/]
Harrison County			
Harrison Elevated Road, 16.0	36[2/]	50[2/]	N/A[3/]
Jackson County			
Ocean Springs Elevated Road, 11.0			
Ocean Springs Levee	20	27	29
Gulf Park Estates	24	35	40
Gulf Park Estates Alter.	20	25	29
Belle Fontaine	29	41	45
Bell Fontaine Alter.	20	29	33
Gautier	32	41	43
Pascagoula			
South Shore	37	39	53
Bayou Cassotte	18	23	26
River near tide gage	16	24	27
Moss Point	13	19	21
Pascagoula – Washington			
At Washington Ave.	19	28	31
Bayou Cassotte	18	23	26
River near tide gage	16	24	27
Moss Point	14	19	21
Pascagoula Moss Pt. Alter			
Moss Point Alt.	14	19	21
Bayou Cassotte	18	23	26
River near tide gage	16	24	27
South Shore	37	39	53
Pascagoula – Washington/MP			
Moss Point Alt.	14	19	21
Bayou Cassotte	18	23	26
River near tide gage	16	24	27
At Washington Ave.	19	28	31

Notes: 1/ This feature is given a discrete elevation. El. 11 ft. is approximately the 1 in 25 annual chance still water elevation. Feature was dropped from consideration. 2/ This feature is also given a discrete elevation. Crest el. of 16 ft. is between the 2% (1 in 50 chance) and 1% (1 in 100) still water elevation. This feature was also dropped from consideration. 3/ Not computed due to excessive crest elevations required at this location for lesser events.

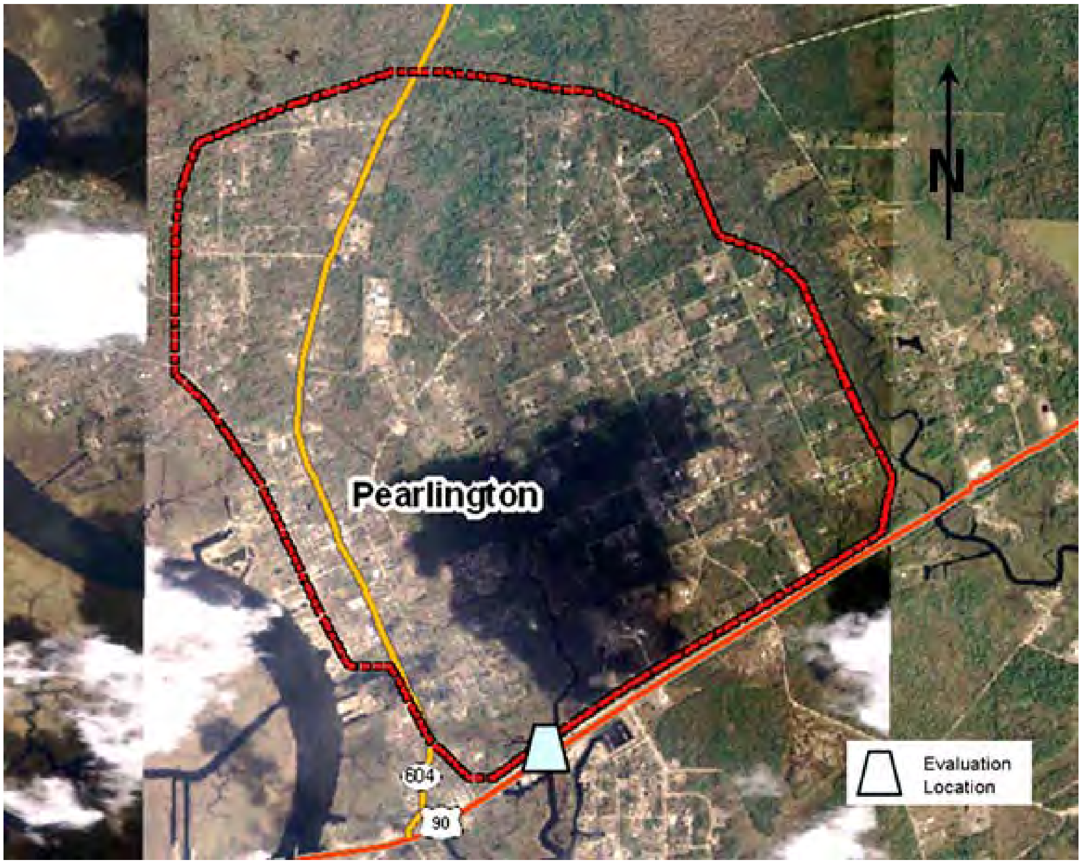
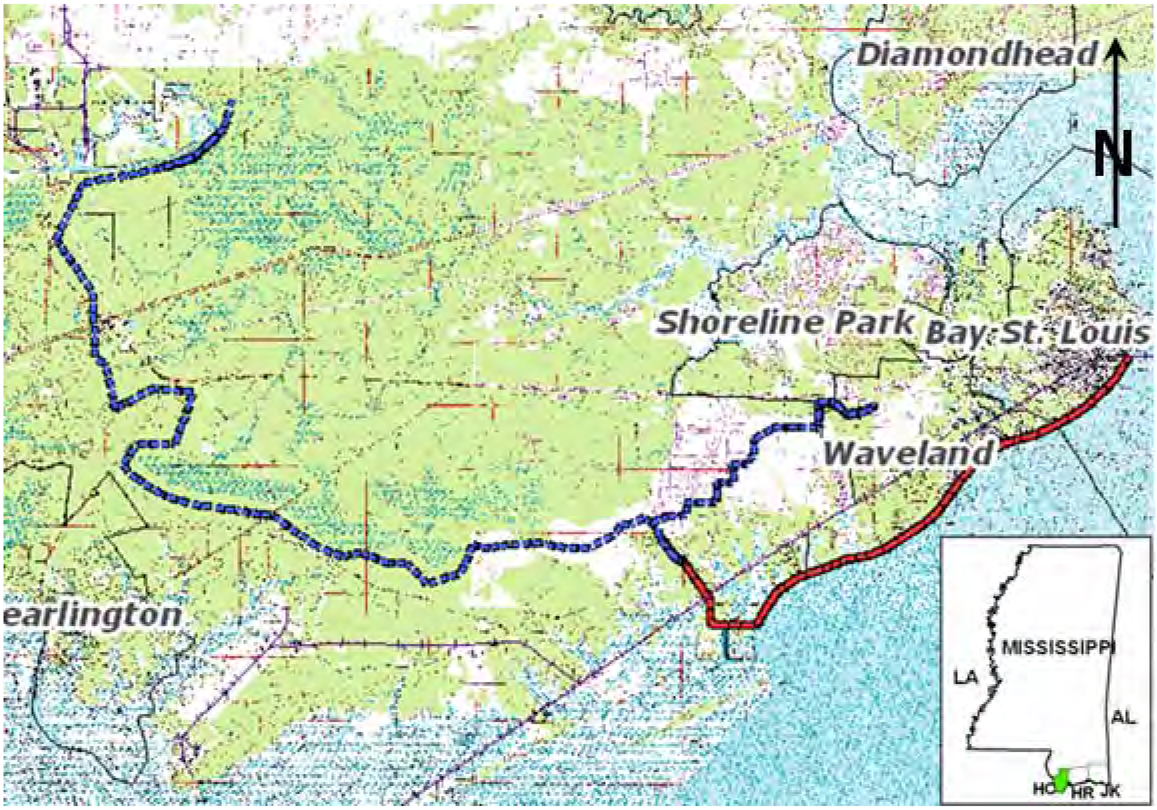


Figure 2.14-3. Hancock County, Pearlington Ring Levee

Figure 2.14-4. Hancock County, Bay St. Louis Ring Levee



NOTE: LOD3 shown linked to inland LOD4 feature.

Figure 2.14-5. Hancock County, Elevated Roadway and LOD4

Figure 2.14-6. Harrison County, LOD3 at Pass Christian

Figure 2.14-7. Jackson County, Ocean Springs Ring Levee

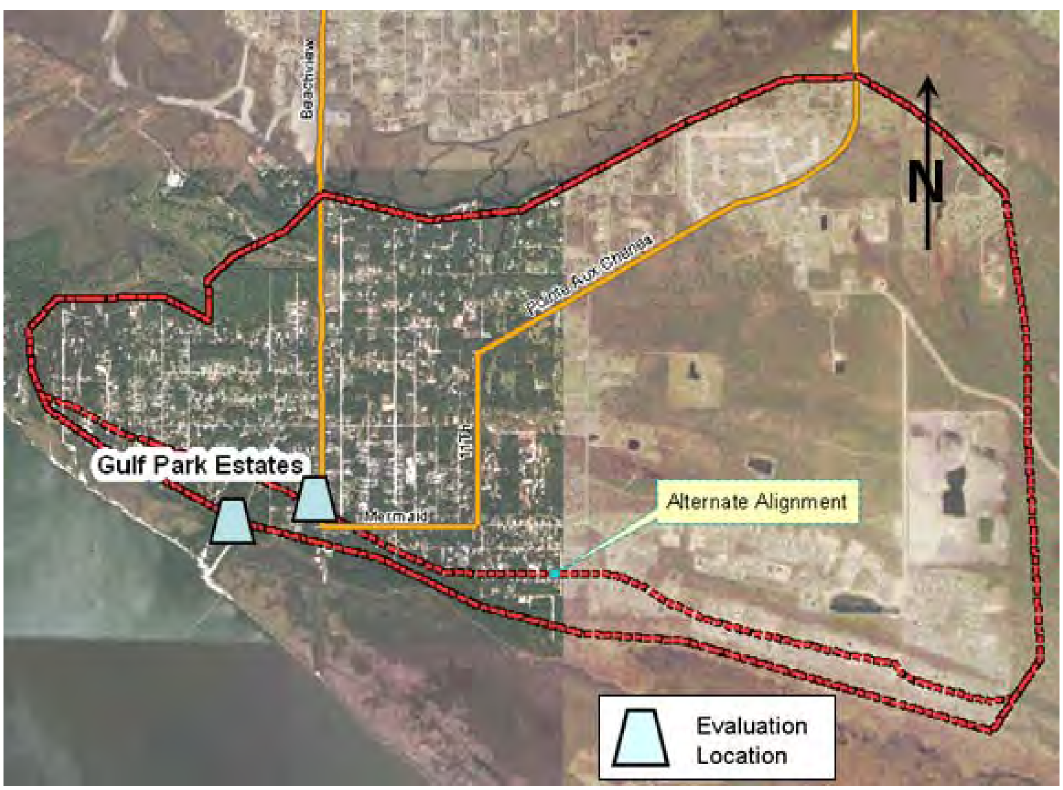


Figure 2.14-8. Jackson County, Gulf Park Estates Ring Levee

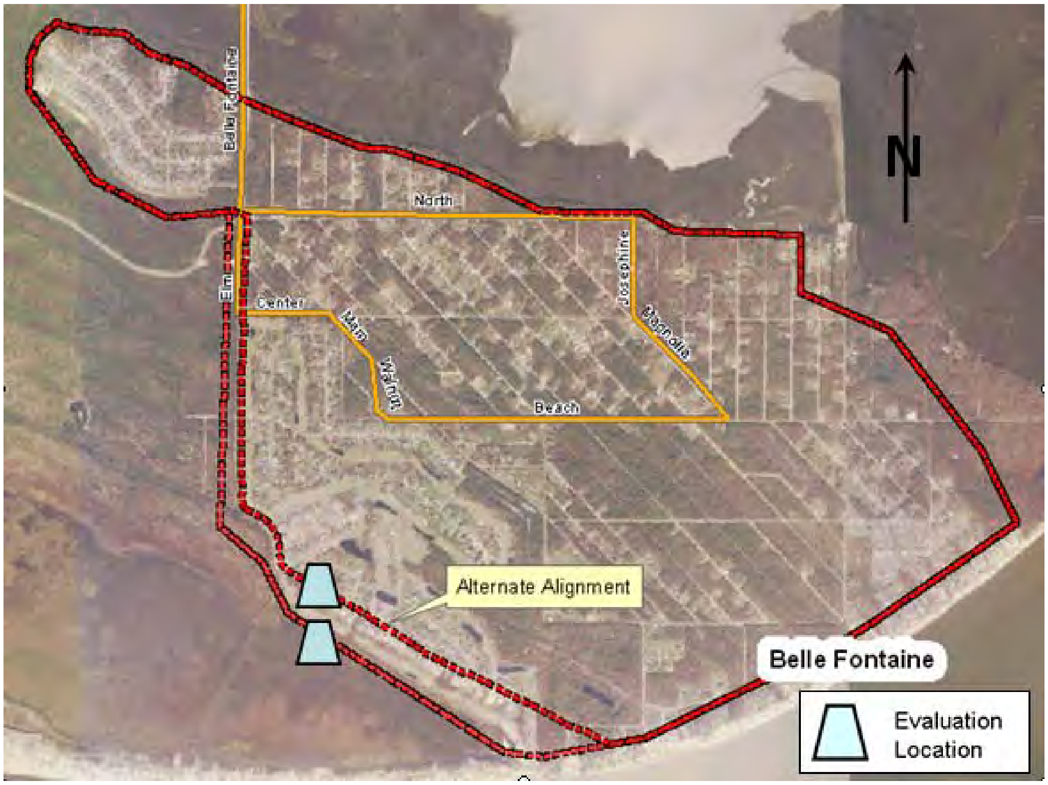


Figure 2.14-9. Jackson County, Bellefontaine Ring Levee

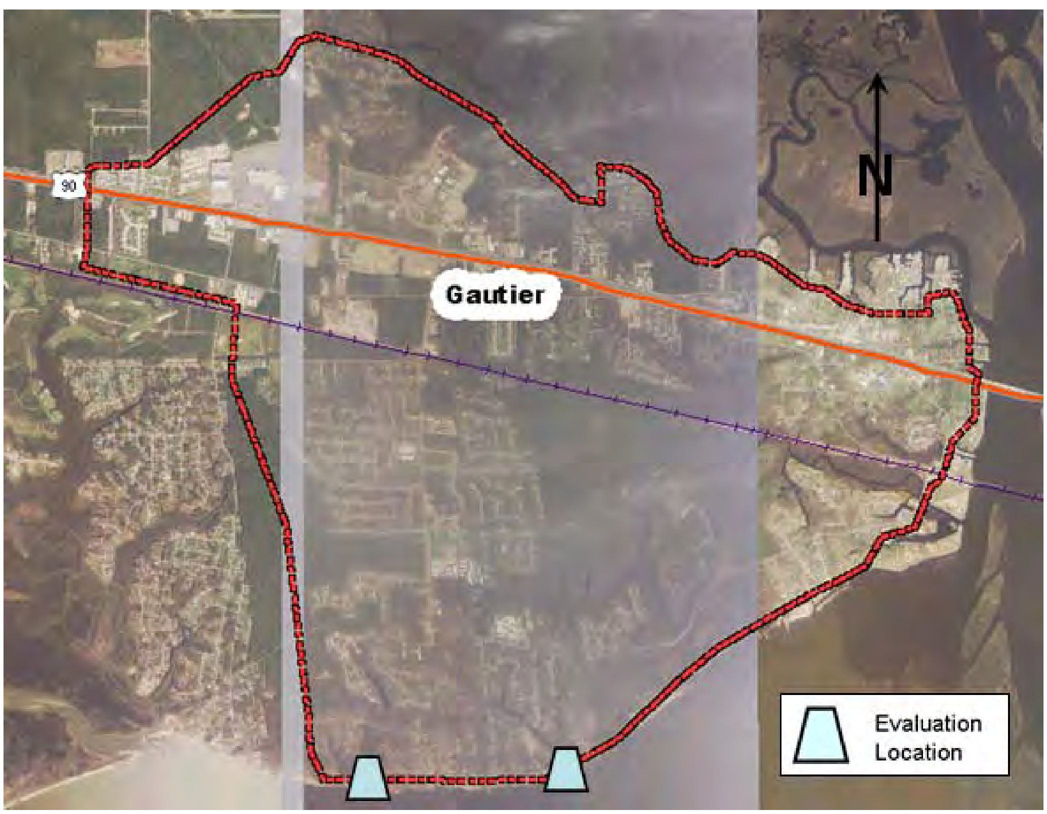


Figure 2.14-10. Jackson County, Gautier Ring Levee

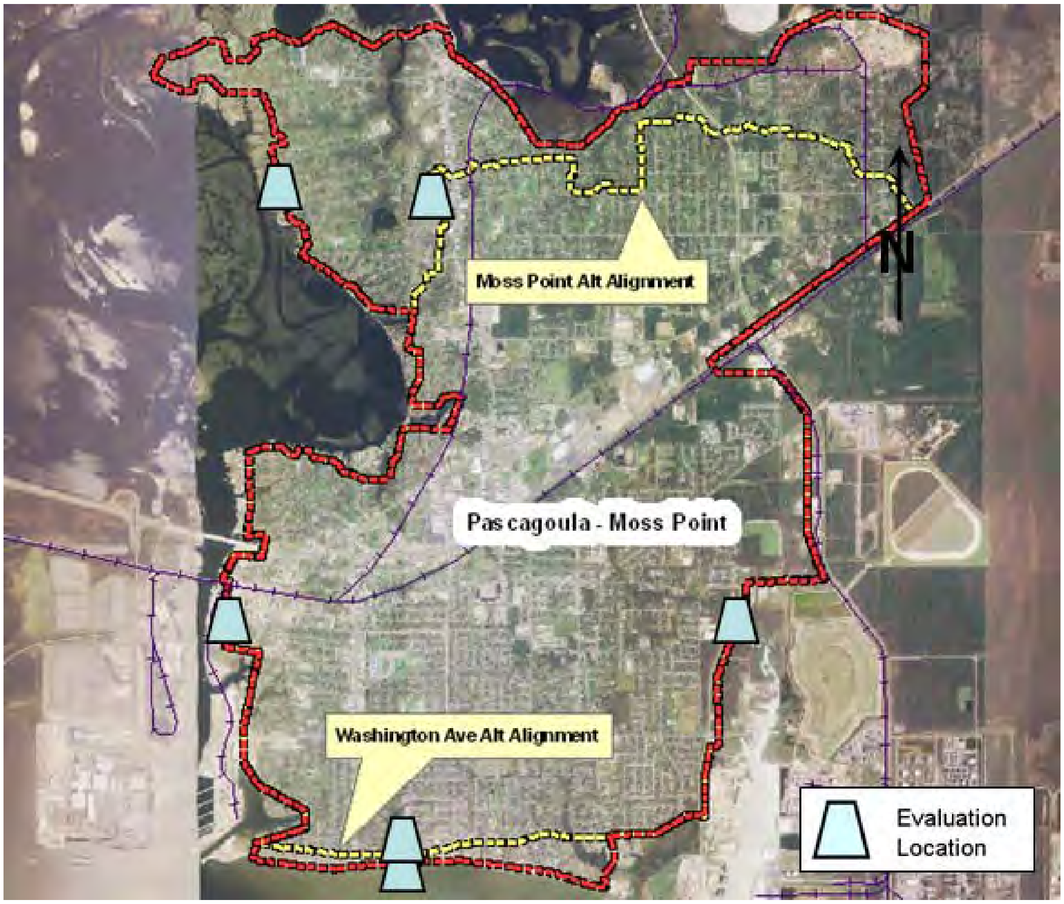


Figure 2.14-11. Jackson County, Pensacola/Moss Point Ring Levee

2.14.2 Line of Defense 4

Computed crest elevations for locations along LOD 4 are given in Table 2.14-2. Crest elevations given are reported in feet NAVD '88 datum. Locations at which the elevations were computed are shown in Figures 2.14-12 through 2.14-16. Computed levee crest elevations range from elevation 18 feet to 50 feet over the range events. For the one in 100 chance events, computed crest elevations range from 13 to 35 feet, with most locations yielding elevations in the high teens to mid twenties. Here, as also with LOD3, the given crest elevation for the 1 in 100 annual chance event is on the order of 30 to 35 feet; such results suggest that the typical levee cross-section geometry is not practicable at the given location and would benefit from modification and/or from a change in alignment to a more quiescent location.

The surge barriers are of novel geometry in their at-rest and deployed condition and empirical overtopping rate relations do not apply to them. Crest elevations for the surge barriers were computed using Table VI-5-13 (Franco and Franco's equation) of the Coastal Engineering Manual (CEM) Professional Edition software, version 2.0.1.1. This equation is most applicable to vertical wall structures. As with the levees, the elevations given assume an acceptable overtopping rate of 0.01 cfs/ft without consideration to interior (i.e. landward of the barrier) flooding attributes. It is possible that a larger overtopping rate might be structurally and operationally acceptable for these features, which would result in a lower crest elevation and lower construction costs. Design refinement awaits further study as desired.

Table 2.14-2.
Computed Structure Crest Elevations, LOD 4

Feature and Location	Annual Event Chance		
LOD-4	**1 in 100**	**1 in 500**	**1 in 1000**
Hancock County			
Clermont	35	47	50
Bay St. Louis Levee	22	29	32
Harrison County			
Pass Christian Harbor	21	29	31
Biloxi West	20	26	29
Biloxi Casino Row	20	29	34
Menge Ave	20	28	30
Jackson County			
Jackson County - Ocean Springs	18	25	28
Surge Barriers			
Bay St. Louis Closure Structure	30	44	49
Biloxi Bay Closure Structure	31	43	47

Figure 2.14-12. Hancock County Inland Barrier

Figure 2.14-13. Harrison County Inland Barrier

Figure 2.14-14. Jackson County Inland Barrier

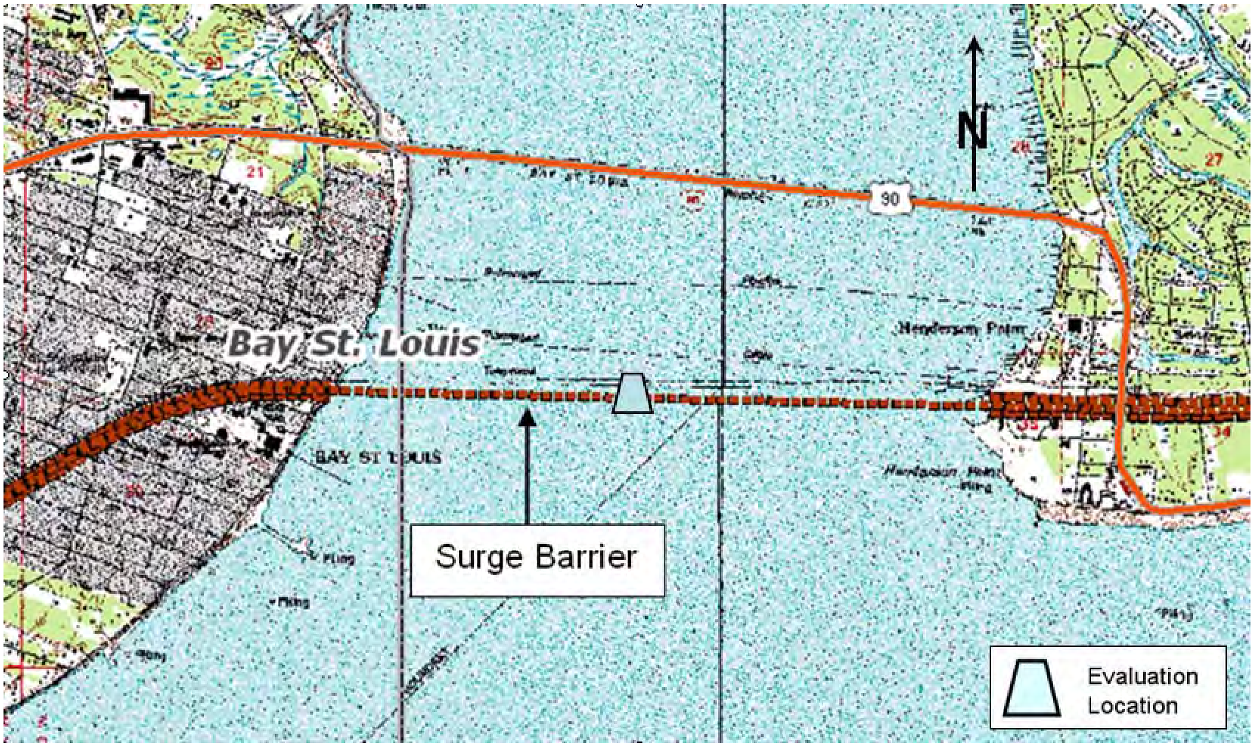


Figure 2.14-15. Bay St. Louis Surge Barrier

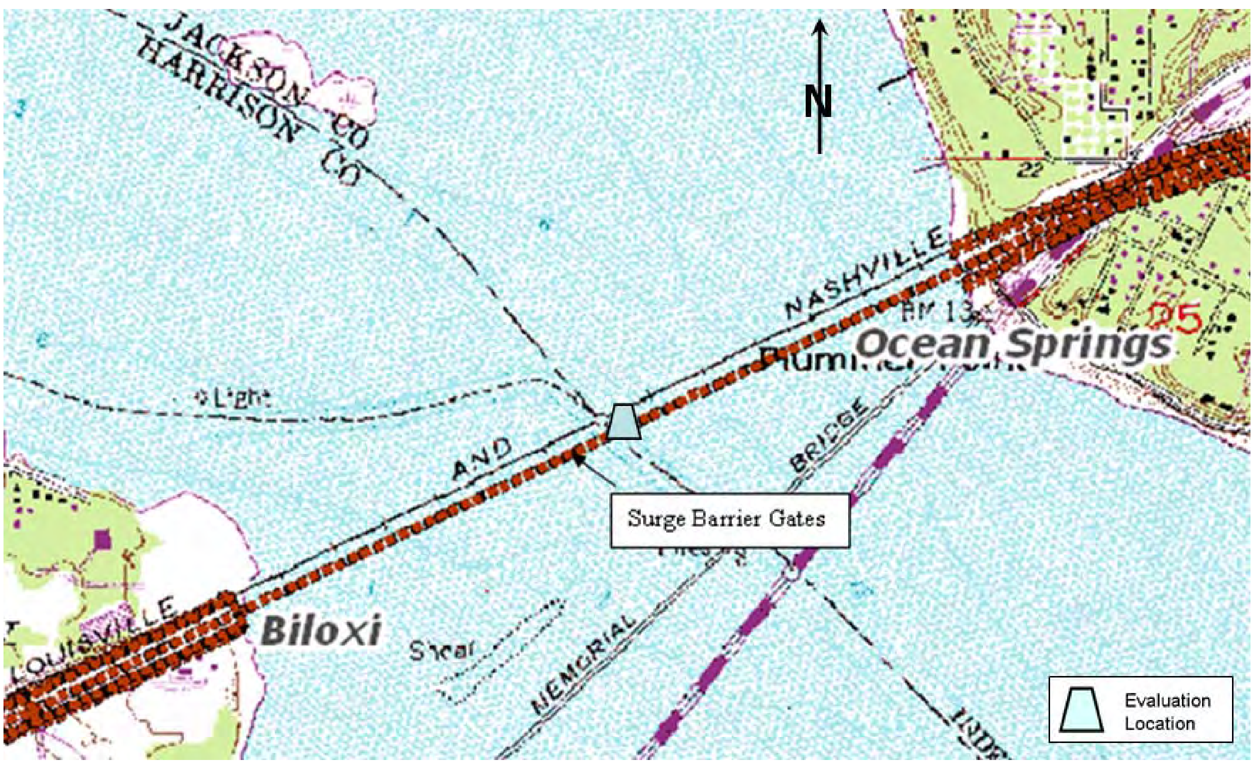


Figure 2.14-16. Biloxi Bay Surge Barrier

2.14.3 References

USACE (2007). Certification of Levee Systems for the National Flood Insurance Program (NFIP). Technical Letter No. 1110-2-570. US Army Corps of Engineers, Washington, DC. 7 August 2007.

PART 3. LINES OF DEFENSE

3.1 Line of Defense 1 – Offshore Barrier Islands

3.1.1 General

The coastline of mainland Mississippi is bordered on the south by the Mississippi Sound, a shallow body of water that separates the coast from four barrier islands that lie several miles to the south as shown in Figure 3.1.1-1. These barrier islands are located along a littoral drift zone that moves sand westward creating three elongated islands and then westward toward Cat Island, where littoral currents are not as well defined. The birds-foot delta system from the Mississippi River has extended through the historic littoral system, cutting off the sediment transport. Cat Island had the same origin than the other islands, but now being re-shaped by wave action and lack of new sediments moving into the system. Wave action has created a beach on the eastern side of the island forming a distinctive T-shape. From west to east, the islands are Cat, Ship (now actually two islands, West and East Ship Island), Horn and Petit Bois. As noted above, Ship Island has been breached by prior hurricanes and now is actually two small islands, West Ship Island and East Ship Island, with a shallow sand bar between the two. Since Hurricane Camille in 1969, this breach has existed with varying amounts of natural rebuilding between later storms and is now known as Camille Cut. The western ends of both Petit Bois and West Ship Islands have migrated westward and are now against maintained deep-water navigation channels and the continuing littoral drift of the sand into the channels is causing an artificial termination of the migration. A small, new island has emerged on the west side of the channel from Petit Bois Island, created from the dredged sand coming from the island that is disposed of on the west side of the channel.

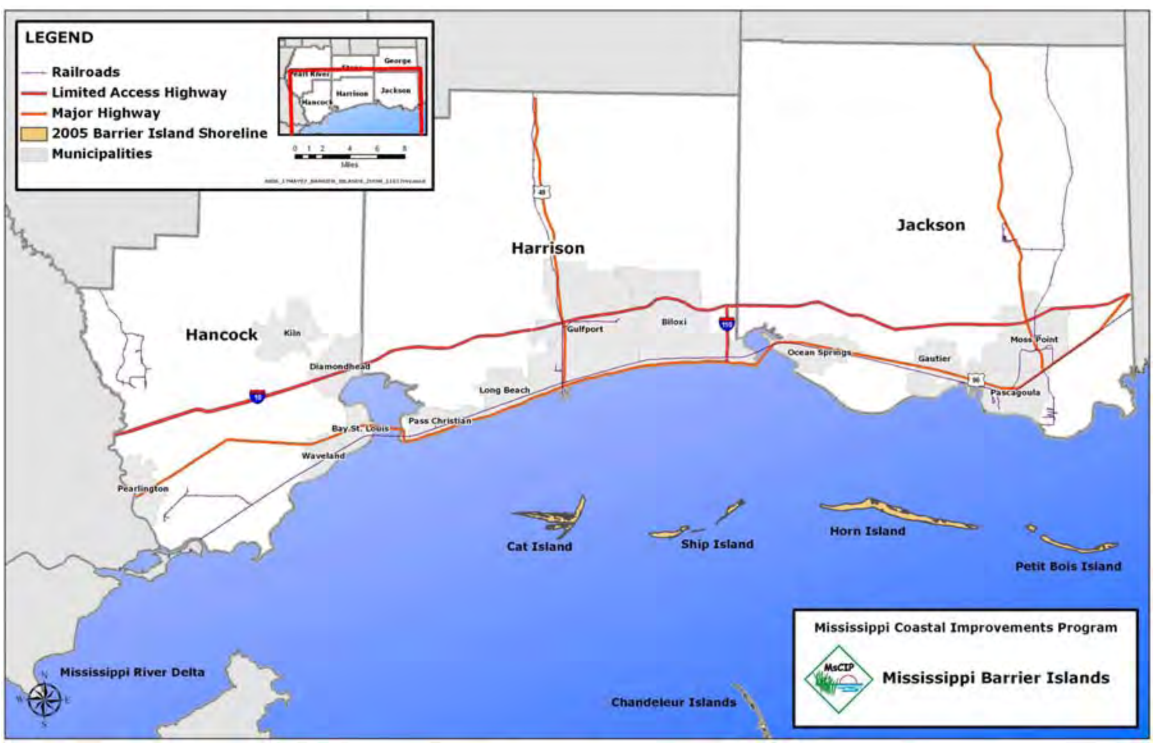


Figure 3.1.1-1. Location of the Mississippi Barrier Islands

Immediately following Hurricane Katrina, most of the effort was spent protecting human life and securing structures throughout the impacted areas on the mainland; therefore, few assessments of the vegetation impacts exist, especially on the barrier islands. For the barrier island system, most all of the marsh vegetation recovered several months following Hurricane Katrina. The predominant vegetation that has long-term impacts consists of those pines found in the maritime forests. It is estimated that about 75% of these pine species were killed following the hurricane season of 2005, with most of that attributable to Hurricane Katrina. Figure 3.1.1-2 is a photograph taken on Horn Island after Hurricane Katrina that shows the loss to the pine trees. The emergent marsh habitat is thriving so well it actually looks as though hurricanes never passed through the barrier island system. The sea oats are still found in small patches due to the reduced dune system. Any option that includes the planting of marsh vegetation will have to consider the current population of nutria that inhabits the islands. These exotic animals from South America can destroy attempts to establish marsh planting and any program should include the control of these rodents.

Figure 3.1.1-2. Photo of interior of Horn Island. Note the mature pine trees that were killed from the effects of salt water that covered the island during Hurricane Katrina.

In 1998, Hurricane George played a role in destroying many of the sand dunes on the islands. Although a relatively small storm, the constant pounding of the waves along the beaches eroded most of the dunes on the southern shores which were the higher elevations on the islands. Along with the destruction of the dunes was the loss of the associated vegetation and habitat. Figure 3.1.1-3 is a photo of the south beach of Horn Island where hurricanes have destroyed the dune system.

Prior to Hurricane Katrina, the State of Mississippi was working on a coastal storm protection plan that included restoring the barrier islands to the condition that existed prior to Hurricane Camille. The general assumption was that there would have been less damage along the coast from Hurricane Katrina if the islands had been in this improved condition. This was also included in the Mississippi Governor's Hurricane Katrina Recovery Plan which called for restoring the islands to a pre-Camille footprint. This concept was included in the hurricane protection study as LOD-1.

Figure 3.1.1-3. Photo of the south beach at Horn Island. Pre-existing dunes have been destroyed by numerous hurricanes over the last several years.

To determine the effects of the islands in reducing the surge damage to the mainland, a number of storms were selected to model against the chain of islands in a pre-Camille and a post-Katrina configuration. The post-Katrina condition can be considered a baseline condition for the modeling and the pre-Camille condition would be an improved condition. The pre-Camille footprint of the islands was obtained from historical records and an assumption was made as to a top of dune elevation and a typical island width. During the modeling process, the island sizes were held constant and not allowed to be destroyed. It should be noted that some of the islands have migrated and any reconstruction would be to increase their footprint at their present location and not move them back to historical locations. In general, the islands were modeled with a 2000-foot width and with an elevation 20.0 dunes, but may be in a slightly different position. Modeling efforts have concluded that over a wide range of storms, there would be some protection provided to the eastern coast of Mississippi along the Jackson County shoreline if the islands are in the pre-Camille condition. This area is the most protected from the restored islands and this protection may result in only up to a 10% reduction in storm surge. The effect of this protection diminishes rapidly to the west from Jackson County. An important aspect of the islands shown by the modeling is the reduction of the large sea waves as they advance towards the mainland. Reduction in wave height up to several feet is realized by the presence of the islands. Loss of Ship Island would leave a portion of the heavily developed Harrison County shoreline subject to these larger waves.

All of Petit Bois, Horn, and Ship Islands and part of Cat Island are within the boundaries of the Gulf Islands National Seashore under the jurisdiction of the National Park Service. The park boundaries are shown in Figure 3.1.1-4. In most cases, the boundary extends one mile from the shore of the island. Petit Bois and Horn Islands have also been designated as Wilderness Areas by the U.S. Department of the Interior and have a higher degree of protection than the other islands.

The formation of Camille Cut has created problems for the National Park Service due to the location of two historically important sites. Fort Massachusetts is located on the northern shore of West Ship and the French Warehouse is located on the northern shore of East Ship Island. Both of these sites are endangered by on-going erosion of the shoreline with Mississippi Sound. Another site known as the Quarantine Station has already been lost to erosion. These sites are shown in Figure 3.1.1.-5. This photo was taken after Hurricane Katrina, but, would be similar to conditions after Hurricane Camille.

Figure 3.1.1-4. Boundaries of the Gulf Islands National Seashore

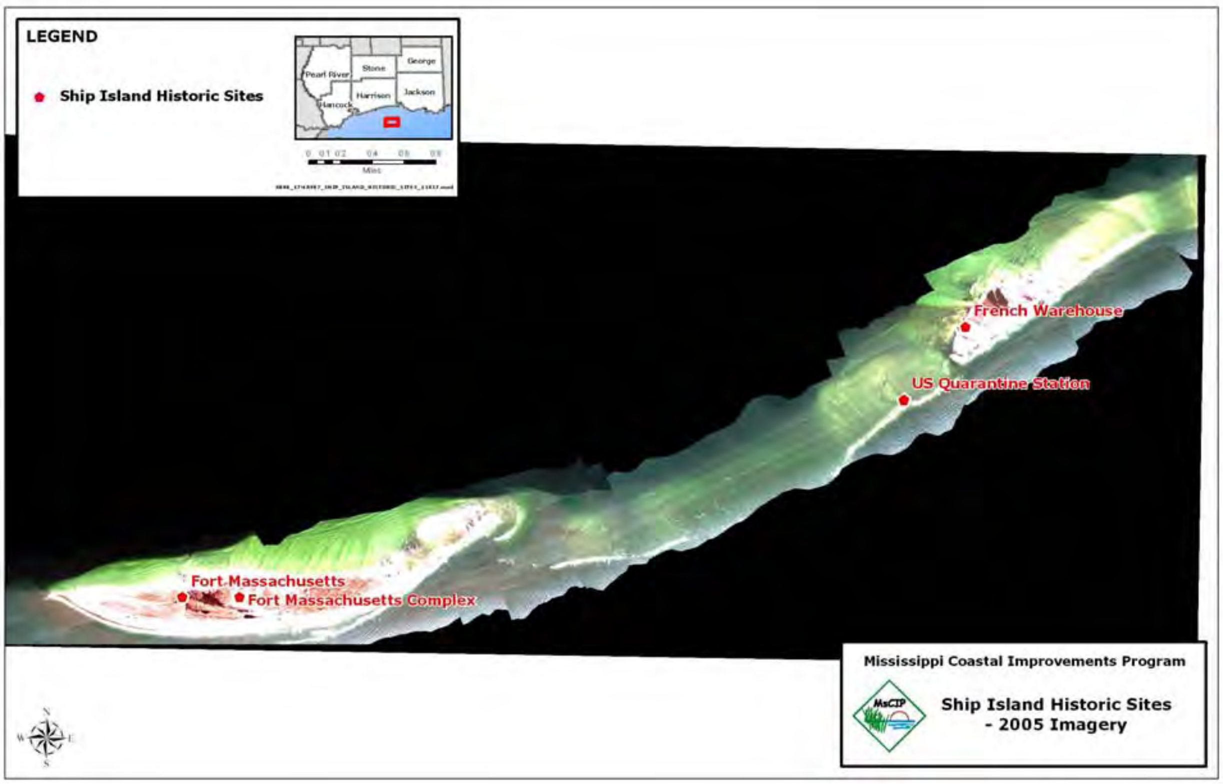


Figure 3.1.1-5. Aerial photo of West and East Ship Island taken in 2005 after Hurricane Katrina showing the locations of listed historical sites separated by Camille Cut.

Fort Massachusetts was originally built on the western tip of Ship Island. The westward migration of sand along the southern shore and erosion of the northern shore now has put the fort almost a mile from the western tip of the island, but dangerously close to being in the Sound. Several emergency beach re-nourishments have taken place over the last 35 years to protect the fort from wave action during winter storms. At present, the NPS is again requesting that the Corps place sand along the shore near the fort in conjunction with dredging operations at the Gulfport navigation channel. This emergency placement of sand is being repeated about every five to six years.

The French Warehouse site has not had any sand placement on its shoreline in the past. The erosive process is slower at that location, but now there are concerns from the NPS about the integrity of the site. Unlike the location of the fort, the warehouse site is covered by maritime forest which may be slowing the erosion of the shore.

The Corps was asked to visit Fort Massachusetts with the NPS during July, 2007 to look at the present erosion problem and to discuss any possible long-term solutions to the loss of sand along the shoreline. The immediate erosion problem will require re-nourishment of the beach adjacent to the fort similar to the past protection projects. Any type of hardened structural feature as protection for the fort was not desired by the NPS nor was this recommended by the Corps. There was a breakwater placed north of the fort in the past (prior to the barrier islands becoming a National Seashore under the NPS) and seems to be compounding the erosion problems. The problem of a long-term fix may be tied to closing the three mile wide breach known as Camille Cut between West and East Ship Island. Review of historical footprints of the islands indicates that after the breach caused by Hurricane Camille, the westward migration of sand was continuing, but that the sand supply was being depleted before it reached West Ship Island. Aerial photos show the formation of a sand spit that extends westward from East Ship Island. The volume of sand that is creating this spit is being depleted from reaching West Ship Island. The photos also show that a deeper channel has formed a pass between the eastern end of West Ship Island and the western end of the spit. It appears that an ebb tidal delta at this pass moves the sand southward where it is removed from any migration along the northern shore of West Ship Island. The sand continues to supply the south beach and extends the western tip of the island in its migration. The loss of the sand from the littoral drift along the northern shore of West ship Island has resulted in erosion of that shoreline. Figure 3.1.1-6 shows an excellent aerial view of this process. Note the boat on the northern side of the pass.

Figure 3.1.1-6. Aerial photo of West and East Ship Island taken in 2001. Note the sand spit extending westward from East Ship Island and the pass between the two islands.

A positive by-product of filling of the Camille Pass would be to provide a longer term solution to the erosion on the northern shores of West Ship Island. This will require modeling to better understand the benefits that are believed to be associated with this plan. The costs will be substantial due to the large quantities of high quality sand that will be required to fill the breach. Initial estimates for sand requirements are approximately 8 million cubic yards. The fill would be expected to prevent the continuing loss of sand to West Ship Island, but it is also understood that the islands are a dynamic system, ever changing to nature's forces. Different types of dune vegetation planting would also be included to restore habitat on the newly created land.

3.1.2 Restoration of the Offshore Barrier Islands

3.1.2.1 General

Soon after Hurricane Katrina, it was reported that many residents in Mississippi were of the opinion that if the islands had been in the condition that existed prior to Hurricane Camille, there would have been less damage along the coast from Hurricane Katrina. This initial concept was also included in the Mississippi Governor's Restoration Plan which called for restoring the islands to a pre-Camille footprint. Changes in the footprints are shown in Figures 3.1.2.1-1 through 3.1.2.1-4.

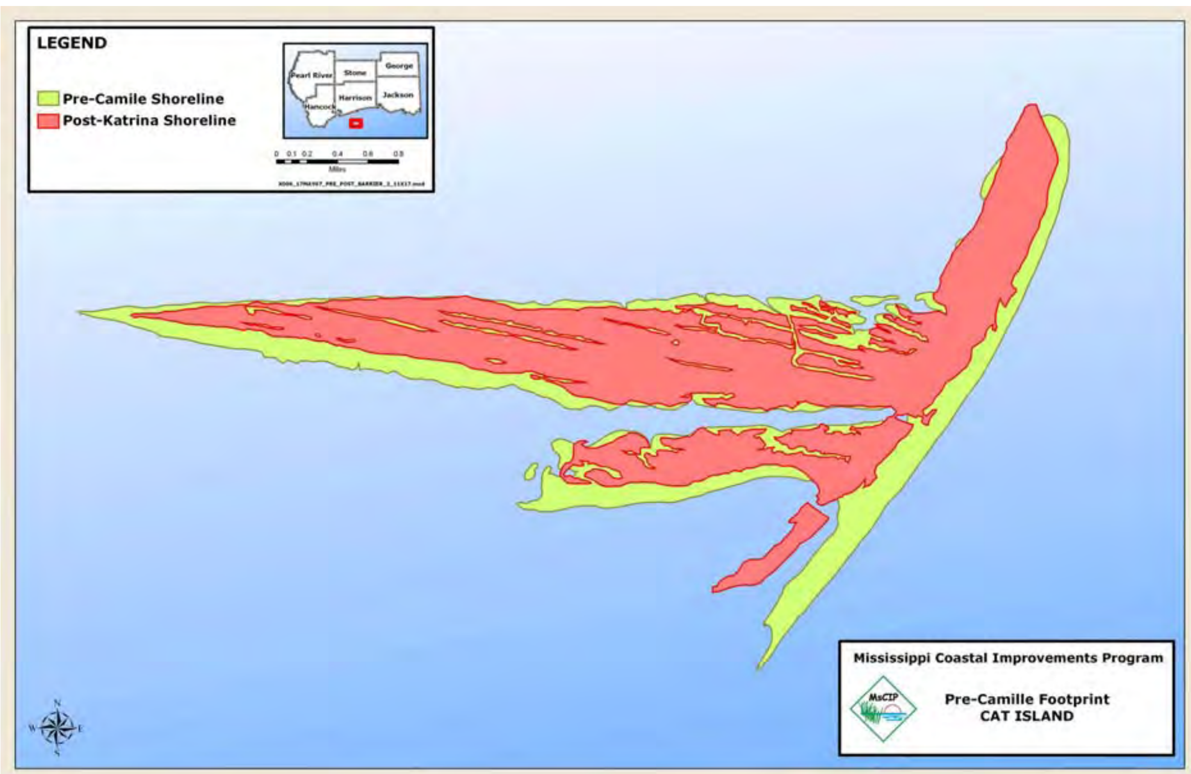


Figure 3.1.2.1-1. Changes in footprint of Cat Island from pre-Camille to post-Katrina

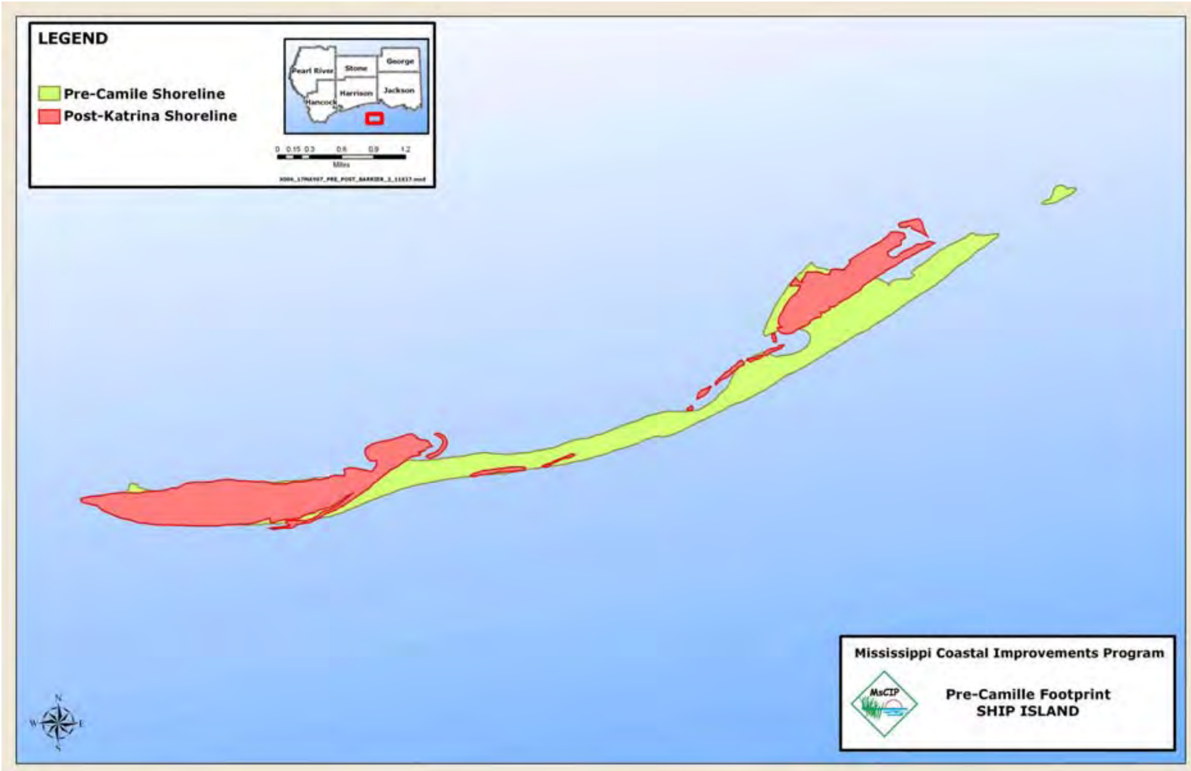


Figure 3.1.2.1-2. Changes in footprint of Ship Island from pre-Camille to post-Katrina

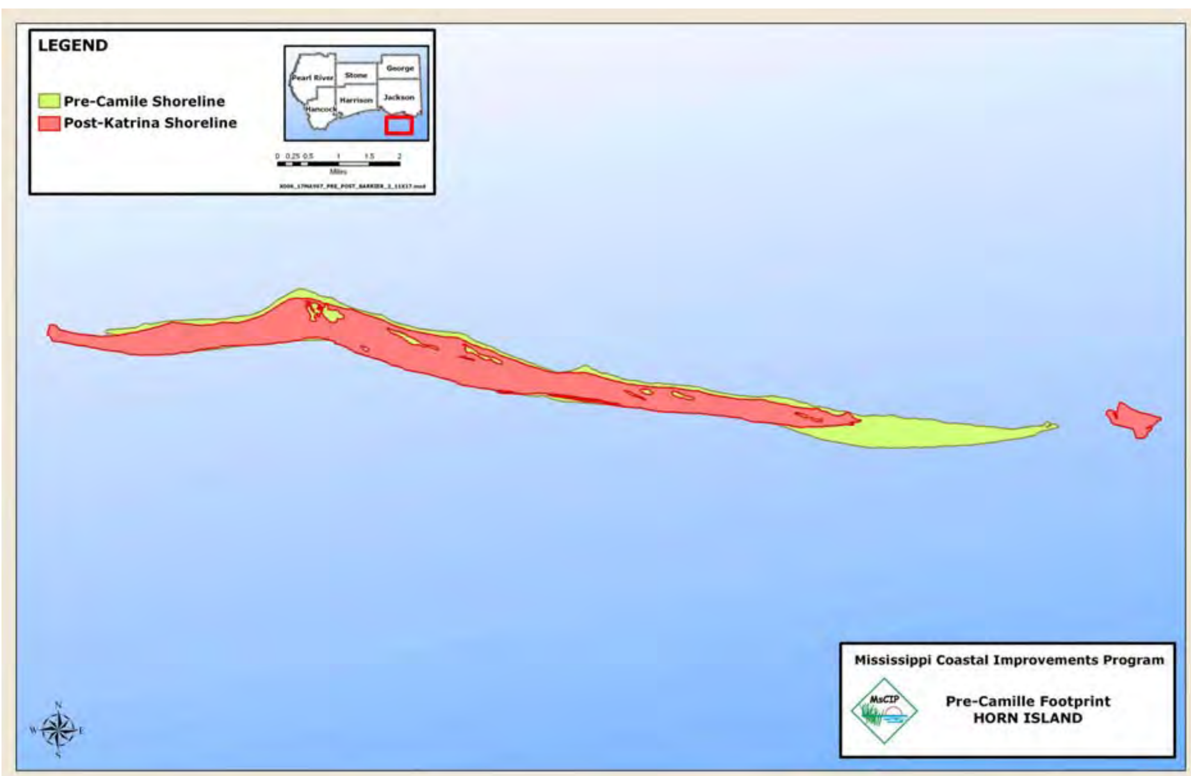


Figure 3.1.2.1-3. Changes in footprint of Horn Island from pre-Camille to post-Katrina

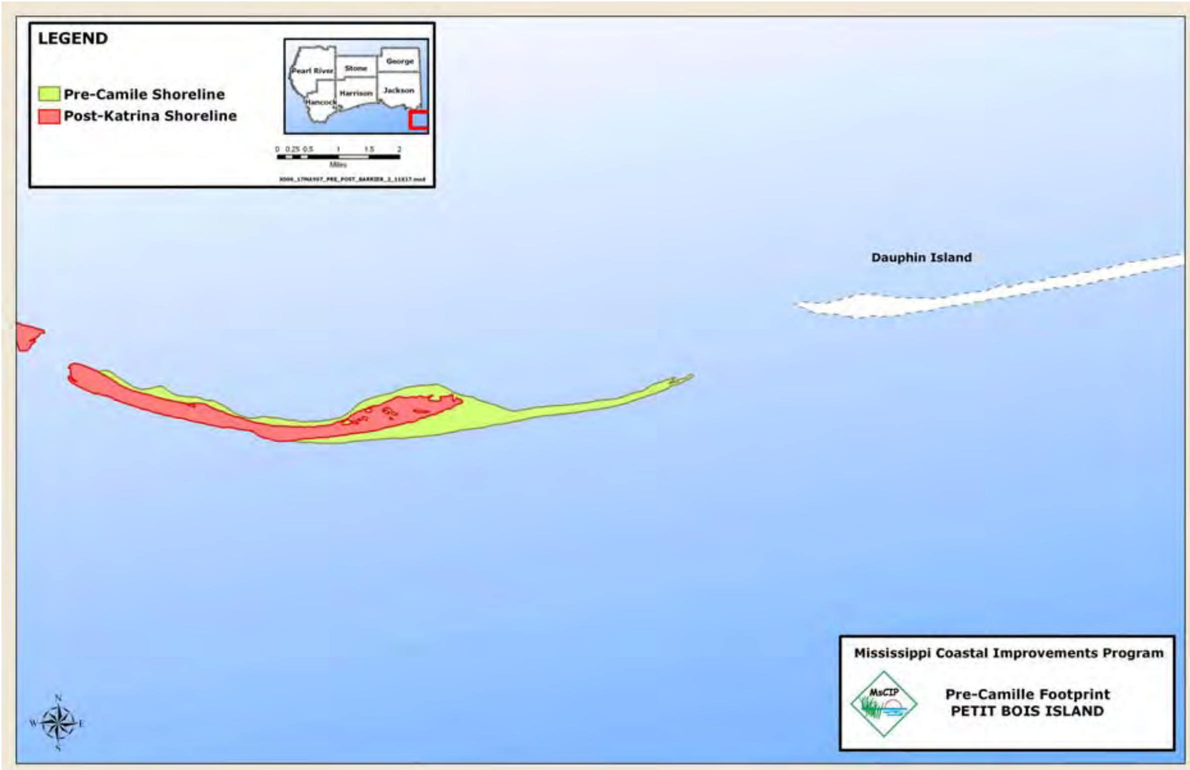


Figure 3.1.2.1-4. Changes in footprint of Petit Bois Island from pre-Camille to post-Katrina

As discussed in Section 3.1.1, a number of storms were selected to model against the chain of islands in a pre-Camille and a post-Katrina configuration. The post-Katrina condition can be considered a baseline condition for the modeling and the pre-Camille condition would be an improved condition. The pre-Camille footprint of the islands (USGS, 2007) was obtained from historical records and an assumption was made as to a top of dune elevation of 20 feet. It should be noted that some of the islands have migrated and any reconstruction would be to increase their footprint at their present location and not move them back to historical locations. This increase in size generally increased their length and maintained their typical width.

Modeling efforts have concluded that over a wide range of storms, there would be some protection provided to the eastern coast of Mississippi along the Jackson County shoreline if the islands are in the pre-Camille condition. This area is the most protected from the restored islands and this protection may result in only up to a 10% reduction in storm surge. As was shown in Figure 2.1-6, the effect of this protection diminishes rapidly to the west from Jackson County. With the consideration that these islands are within the National Park Service and that Petit Bois and Horn Islands are designated Wilderness Areas, any improvements to these islands may be politically difficult based on the limited benefits.

Another consideration to help restore the islands is to supplement the sand into the littoral system. This could be accomplished by adding sand in specific locations based on sediment transport modeling. This sand would not be put on the islands, but in areas between the islands where the currents that make up the littoral drift zone could transport the sand to the islands where the natural process of island building could take place. There, waves and wind could cause accretion on the islands. This may mitigate the loss of land mass at the islands that has been occurring since Hurricane Camille. The source of these sands may be from inland sources or from offshore borrow areas. This would not directly affect the present-day islands and would help mitigate any effects of dredging the ship channels that pass through the chain of islands where sand may have been lost from the system.

A positive affect that the islands have is to provide a natural off-shore breakwater for the large sea waves that are generated from hurricanes. For this to occur, the islands only need to be a low stretch of sand or even a shallow sandbar. The presence of the islands and the relatively shallow water of the Mississippi Sound between the islands and the mainland prevent the sea waves from maintaining their considerable size as they move towards the mainland. Sea waves, often reported at heights of 40 feet and higher in large storms, would break as they approach the chain of islands. The open water between the islands and the mainland, generally ten miles or more, would have enough fetch for waves to regenerate, but at a much lower height due to the shallower water. The generally accepted relationship between water depth and wave height is that the wave can sustain itself at a height that is one half the depth of the water.

An environmental impact of the islands continuing to diminish in size is to allow salinity increases in the Mississippi Sound. Mississippi Sound would be classified as a 'bar-built' estuary as opposed to a 'drowned river valley' (like Mobile Bay). The physics of bar-built estuaries is very different from others and you would expect to see broad zones of 'salinities' with the estuary which respond greatly to both river flow and wind conditions. Should the 'bars' go away, then the estuary is totally lost because in general an estuary is considered part of the coast as opposed to forming the coast. Under current conditions, the islands provide a natural boundary between the water’s salinity [~33 parts per thousand (ppt)] of the open Gulf of Mexico and the brackish water found in Mississippi Sound. Salinity in the Sound during low flow periods range from 10 to 30 ppt. Highest salinities occur just south of Pascagoula and Gulfport and the lowest salinities in the Lake Borgne-Pearl River area. Several studies have investigated the impacts of diverting freshwater to promote reversing a historic increase in salinity in the Mississippi Sound/Biloxi marshes area in order to support fresher marshes and oyster reef health and productivity thus enhancing both their economic value and the ecological services they provide. Oysters are sensitive to specific ranges of salinity. Additional modeling and study would be required to determine impacts to salinity from the loss of the barrier islands.

One restoration option for the barrier islands would be to re-establish the vegetation that was destroyed by Hurricane Katrina. This option could involve environmental restoration of the existing islands through adding sand dunes on the beaches along with planted vegetation, planting of marshes and maritime forests, and planting sea grasses in the near-shore areas of the islands. This plan would not involve adding any land mass to the islands other than the possibility of adding to the dune system. The addition of vegetation from sea oats up to trees would aid in reducing erosion of the sand from wind thus helping in maintaining the stability of the islands. The vegetation would also aid in preventing erosion by water in the event that the islands get overtopped by storm surge in a large hurricane. Sources of this sand could be from the beach area behind the dunes or from sources off the island. Historically, large areas of sea grass existed north of the islands. Much of this sea grass is now gone and the loss of these areas have been mapped. Replanting the grasses and other vegetation will aid in establishing valuable habitat that was lost from the ecological system. Figure 3.1.2.1-5 shows the extent of vegetation on Horn Island prior to Hurricane Katrina.

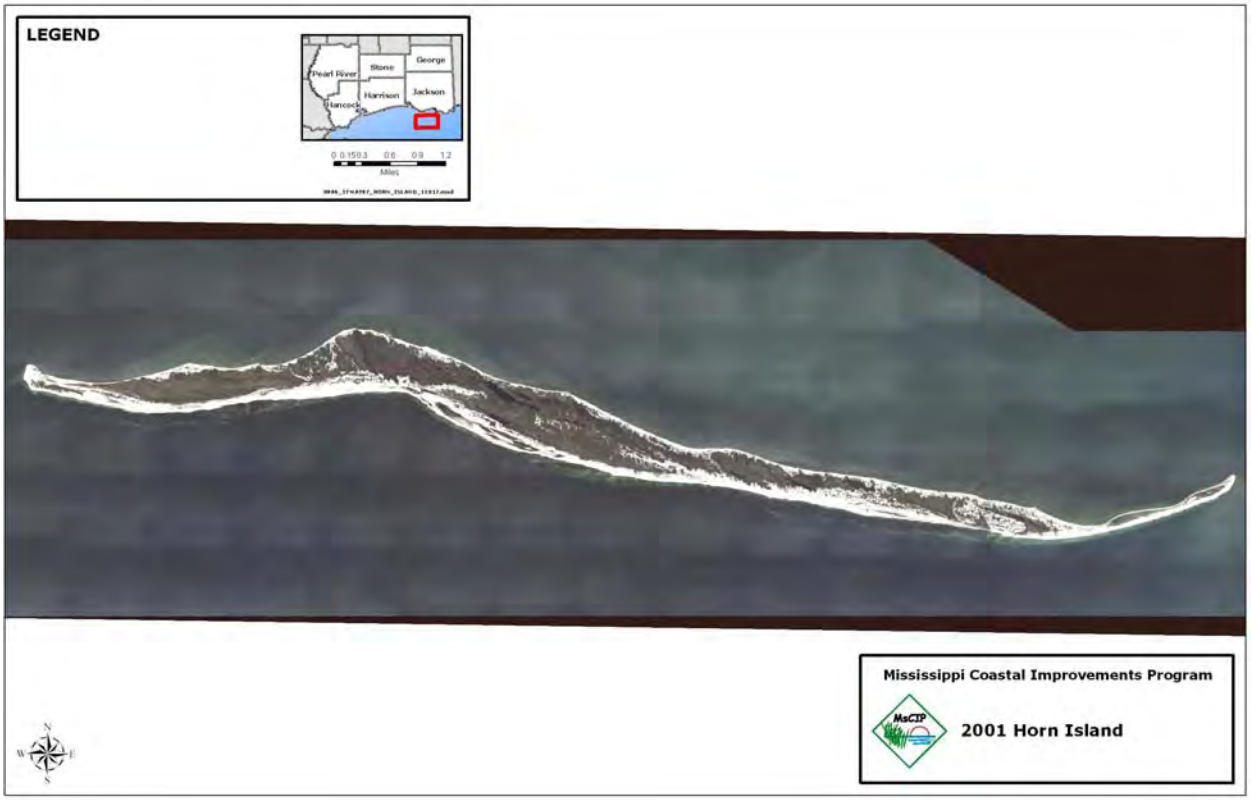


Figure 3.1.2.1-5. Aerial photo of Horn Island. The darker areas are vegetation consisting of maritime forest and marsh grasses.

As discussed in Section 3.1.1, an additional restoration option has been added that will fill and close Camille Cut between West and East Ship Island. In addition to providing some storm damage reduction, this option will provide some protection to two historical sites on West and East Ship Island, respectively. This option will require additional study to model the desired results of slowing erosion near the two sites. During coordination with the NPS, agreements have been reached that will provide positive affects to the barrier islands. These proposals have been incorporated into an alternative based on LOD-1 Options C and G. This alternative consists of adding sand into the littoral zone and closing the breach between West and East Ship island. This alternative is fully described in a separate appendix based on this combination of options titled the Comprehebsive Barrier Island Restoration Plan. A working paper that documents the NPS position on the barrier islands (NPS, Sept. 2007) along with other cooperating agencies is included in the Barrier Island Appendix. An important result of the NPS agreement was that any work that involved direct placement of any sand into Camille Cut would be a one-time event without additional O&M sand placement. In accordance with 2006 NPS Management Policies (see Barrier Isalnd Appendix Chapter 2, the NPS Vision Statement Section III), the NPS has concluded that this one time placement of sand would most directly counteract the long term reduction in sand supply which has resulted in Ship Island being dimished to the point where it may have lost the ability to restore and maintain itselt s in the historical past. Natural re-building and maintenance of the barrier islands in the long term would then be supported by the continuing placement of sand back into the littoral zone during future maintenance dredging of navigation channels. Areas where continuing beneficial placement could be employed will be identified during additional sediment transport modeling conducted during the Engineering and Design phase prior to a contract award.

3.1.2.2 Location

The barrier islands of Mississippi are located 10 to 15 miles south of the mainland. Currently, there are five islands in the chain that extends for 45 miles west from a point south of the Alabama – Mississippi state line along the coast. Currently, Ship Island exists as two islands separated by Camille Cut. It was breached during Hurricane Camille in 1969 and remains today as West and East Ship Island. Two maintained navigation channels pass through the chain of islands. The Gulfport channel passes near the west end of West Ship Island and the Pascagoula channel passes near the end of Petit Bois Island. The present day location of the channels prevents any further westward migration of either island.

3.1.2.3 Existing Conditions

As is typical of most barrier island systems, the Mississippi islands are an ever-changing and dynamic landscape. Data shows that the islands have lost approximately 20 to 25 percent of their land mass since pre-Camille times. The islands have been heavily influenced by the various hurricanes including even the lower intensity ones. Hurricane George, in 1998, even though a small hurricane, proved to be devastating to the islands due heavy erosion from waves. Many of the higher dunes systems on the islands were destroyed and much of the elevation the islands once had is gone. Most of the islands are now very susceptible to over-wash during storms. Another result of being submerged during Hurricane Katrina was the loss of much of the maritime pine forest that existed on the islands. The trees, mostly now dead from the salt water submergence, played a major role in preventing erosion both from wind and any surges against the islands.

The westernmost island, Cat Island, has a similar origin from the other islands in the chain, but isolated from the littoral current by a historical birds-foot delta from the Mississippi River that cut off the path of the historical littoral zone. A change in wave climate has formed a T-shaped configuration. Sorting of the sediments has created a beach on the east facing portion of the island. Results of the sediment budget completed as part of this study indicates that little or no sand is being added to Cat Island from the littoral drift system that supplies sand to the other islands in the chain. The remainder of the islands have a westward drift that is more pronounced from the eastern-most Petit Bois Island and decreasing respectively to the west to West Ship Island.

3.1.2.4 Coastal and Hydraulic Data

The barrier islands protecting the Mississippi Sound experience a low energy wave climate, with average significant wave height at National Data Buoy Center (NDBC) Buoy 42007 (22 nautical miles south-southeast of Biloxi, in 45 ft depth) averaging 2.0 and 1.3 feet in the winter and summer months, with associated average peak wave periods of 4.0 to 3.5 s, respectively. Wave transformation modeling by Cipriani and Stone (2001) indicated that breaking wave heights on the barrier islands range from 1.0 to 2.0 feet. Waves in the Mississippi Sound are fetch and depth-limited. The Coastal Studies Institute's Wave-Current Surge Information System (WAVCIS) gage CSI-13 located at Ship Island Pass (23 foot depth) from June 1998 through July 2005 measured an average significant wave height of 0.3 feet and associated average wave period of 2.5 sec.

Tides in the Mississippi Sound are diurnal, with a tidal range of 1.5 and 2.8 feet for the mean and spring tides at Biloxi, Mississippi, respectively. However, the relatively shallow and large area of the Mississippi Sound create strong currents in the tidal passes between the barrier islands, ranging from 1.6 to 3.3 feet/sec and 6.0 to 11.5 feet/sec on flood and ebb tides, respectively (Foxworth et al. 1962). In the winter months, winds from the same direction and of a sufficient magnitude are capable of lowering water surface elevations in the bays and nearshore from 3.6 to 2.0 feet (U.S. Army Corps of Engineers Mobile District 1984).

For the Gulf barrier island beaches, net longshore sediment transport is from east to west, although local reversals in the net transport occur adjacent to the tidal passes. The primary sources of sediment are longshore sediment transport from east to west, and, potentially, the offshore shelf (Otvos 1979, Cipriani and Stone 2001). Cipriani and Stone (2001) discussed that a well-defined cellular structure exists for each barrier island in which, over historic times, little sand transfer exists between islands. However, dredging records at Horn Island and Ship Island Passes (called Pascagoula Bar Channel and Gulfport Bar Channel, respectively) suggest that infilling of sand from adjacent barrier islands occurs, indicating the potential for transport of sand between islands. Eastern Dauphin Island, with a Pleistocene core, is more stable than the other barriers although eastern Dauphin Island has been eroding in response to the dominant westerly-directed transport. Based on grain size analysis, Cipriani and Stone (2001) inferred that offshore sources may provide some sediment to central Petit Bois Island. The Mississippi Sound barrier islands range from very well vegetated, with maritime forests on east Dauphin Island, to low elevation barriers that are overwashed and breached during hurricanes. Long-term relative sea level rise for Dauphin Island, Alabama from 1966 to 1997 was 0.12 inch/year +/- 0.02 inch/year.

3.1.2.5 Option A – Restore Pre-Camille Island Footprint

As part of the Seven Step Strategy developed by the Governor of Mississippi, an option was developed to look at restoring the barrier islands to a pre-Camillr footprint. The pre-Camille footprint of the islands was obtained from historical records and the amount of area that has been lost to coastal erosion since that time was computed. Without accurate topography of the islands an assumption was made that some dunes had a top of elevation of 20 feet. It should be noted that some of the islands have migrated and any reconstruction would be to increase their footprint at their present location and not move them back to historical locations. Figures 3.1.2.1-1 through 3.1.2.1-4 showed the changes in the land mass of the islands from a pre-Camille condition to a post-Katrina condition. It was also recognized that NPS support for this option was unlikely due to conflicts with that agencies 2006 Management Policies and statutory responsibilities.

Several approaches to restoration of the islands were considered. This option will only include new land mass that is being added to the islands by using sand dredged and transported from an off-shore location. The shaping of the sand into beaches, dunes and marsh areas will not affect the existing islands other than that narrow strip of land that will form the boundary between the existing island and the new land mass. This option can be used in combination with other options under this line of defense should it be desired to restore habitat on the existing islands.

Restoration of Ship Island to a pre-Camille configuration includes closing the post-Katrina, 3-mile long breach to a 2000-foot width and with elevation 20.0 dunes, along with some rebuilding of the other islands to a larger land area. The land mass of each of the islands was estimated in a pre-Hurricane Camille condition using historical aerial photography. The difference in the size of the islands was then computed based on post-Hurricane Katrina aerial photography. The results of this are as follows:

The difference in the land mass over this period was then converted to an acreage that it would take to restore the size of the footprint. The width of the islands was maintained with the additional land mass being added as length. Each of the surface areas was converted to a quantity by using an average water depth of seven feet and raising the sand up to elevation of 10.0. It was assumed that approximately 25 percent loss of the material would occur during the process of placement.

Sand of sufficient quality in the quantities required for this type of project is not known to occur in close proximity to the islands. Proposed geophysical studies may locate sources near the existing islands. Prior studies of the St. Bernard Shoals (Oral Communication, USGS, 2006) are probably the best source of the sand. Additional studies and sampling will be required to ensure the source. As previously described, St. Bernard Shoals are a series of submerged barrier islands. The average water depth over the shoals is 60 feet which puts the sand within reach of a hopper type dredge, however the water depth near the islands is too shallow for the draft of hopper dredge that would be used in this type of operation. In order to accomplish this, a basin would be dredged near each of the islands to discharge the sand being transported from the borrow area. Any suitable sand (if encountered in sufficient quantities) would be added as part of the fill, otherwise the material will be transported to approved disposal areas per the Regional Sediment Management guidelines. Using this procedure, the hopper dredge could enter the basin and bottom dump the sand. This would be much faster than pumping off the sand. Doing this would also allow the basin be placed outside the boundaries of the National Seashore. As the basin is filled, a suction dredge would be mobilized to the site and using this type of the equipment, the sand could be moved to the area where the material is needed to create additional land mass. As the sand is placed on the new land mass, it would be sculpted into dunes and swales which would vary from elevation 0 (NAVD 88) up to heights of 20 feet. The amount of new land mass at each of the islands would be approximately the same as the amount shown as lost in Table 3.1.2.5-1. The anticipated amount of sand required for each island is as follows:

Cat Island – 14,600,000 cubic yards

Ship Island – 21,240,000 cubic yards

Horn Island – 21,240,000 cubic yards

Petit Bois Island – 9,300,000 cubic yards

Table 3.1.2.5-1.
The Amount of Land Mass Lost from each of the Mississippi Barrier Islands from pre-Camille conditions to post-Katrina Conditions

Island	Pre-Camille (acres)	Post-Katrina (acres)	Land Loss (acres)
Cat	2,344	1,957	387
Ship	1,172	631 (East and West)	541
Horn	3,612	3,077	535
Petit Bois	1,329	1,098	231

As the new land mass is added to the existing islands, portions of the new island will be planted with various type of vegetation to provide habitat and to aid against erosion. Review of photographs of the islands prior to Hurricane Katrina has provided data on the percentages of the islands that were associated with maritime forest, marsh, dunes, and open beach. The percentage of maritime forest varied among the islands from one percent up to 23 percent. For the new land mass of the islands under Option A additions, it was decided to use a quantity of 20 percent of the land mass for planting the trees consisting of longleaf pine. The lower elevations of the new land mass would be planted with emerging marsh species that would cover 38 percent of the area. This would include Spartina alterniflora, Spartina patens, and Juncus romerianus. Dunes planted with sea oats would make up two percent of the area and the beach areas would be left as open berms. With time, the dunes would transform themselves into a more natural state as wind shifted the sand and the planted vegetation established itself similar to the dunes shown in Figure 3.1.2.5-1.

3.1.2.5.1 Interior Drainage

The type of work anticipated for adding sand to increase the land mass of the islands will not require any type of drainage system. The addition of sand under this operation will be with dredge pipe discharge and all water will be allowed to run back to the sea.

Figure 3.1.2.5-1. Typical sands dunes on gulf coast barrier island

3.1.2.5.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. When the sea level returned to present condition, sediments filled the river channels and started to cover the area that would become the Mississippi Sound. Sandy deposits that had been transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with these deposits.

The actual Sound formed as an estuary after littoral drift of sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand is typically medium grained, white to light grey in color with well rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits, as shown in Figure 3.1.2.5-2, can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward. This process was added by formation of the St. Bernard delta of the Mississippi River that enclosed the western end of the Sound. The western-most island in the chain, Cat Island, is a product of the historic St. Bernard delta lobe. What remains today is a beach front face of the island where waves have sorted the material leaving the sand and deltaic deposits behind the beach.

Figure 3.1.2.5-2. Peat-like organic soils outcropping on the south beach of East Ship Island. These deposits are the remains of sediments and organic matter that settle in the bottom of the marshes and lakes that occur on the barrier islands. The deposits are exposed as the islands migrate northward.

East and West Ship Island, Horn Island and Petit Bois Island are migrating over Pleistocene formations that created a relatively stable platform for the constantly moving islands. Other Holocene deposits provide a relatively thin cover on the bottom of the Mississippi Sound and some areas south of the Islands and consist of a muddy mixture of sand and clay along with shell fragments or buried oyster shell beds.

After the islands formed, the Sound became a brackish estuary and deposits of mostly fine grained, muddy sediments began forming in the Sound. Other than Cat Island, the other islands such as they exist today, are migrating along the littoral drift and are mostly composed of sand. Local layers of peat-like organic soil that are forming in the inter-island lakes and marshes and can become exposed on the beaches as the sand migrates.

If increasing the land mass of the islands, it would be desirable to maintain the same quality sand that now makes up the existing islands. Sources of sand in the quantity that would be required for this option are extremely large, especially when considering the quality standard that must be met. Potential sources for sand were investigated both inland and offshore. Of concern is matching the sand to the sand on the beaches of the National Seashore. Samples taken from Dauphin and Pelican Island in Alabama are in the same island chain and have been tested for color, grain size and particle shape. These results, included in this section as Table 3.1.2.5-2 and Figures 3.1.2.5-3 and 3.1.2.5-4, can be used to match potential sand sources.

Table 3.1.2.5-2.
Munsell Soil Color Evaluation of Sand Samples Taken from the Barrier Islands of Alabama that is within the Littoral Drift Zone of the Mississippi Barrier Islands

Sample ID	Hue	Value	Chroma	Color
Composite 1	5YR	6	3	Pale Olive
Composite 2	10YR	8	1	White
Composite 3	10YR	8	2	White
Composite 4	2.5 YR	7	2	Light Grey

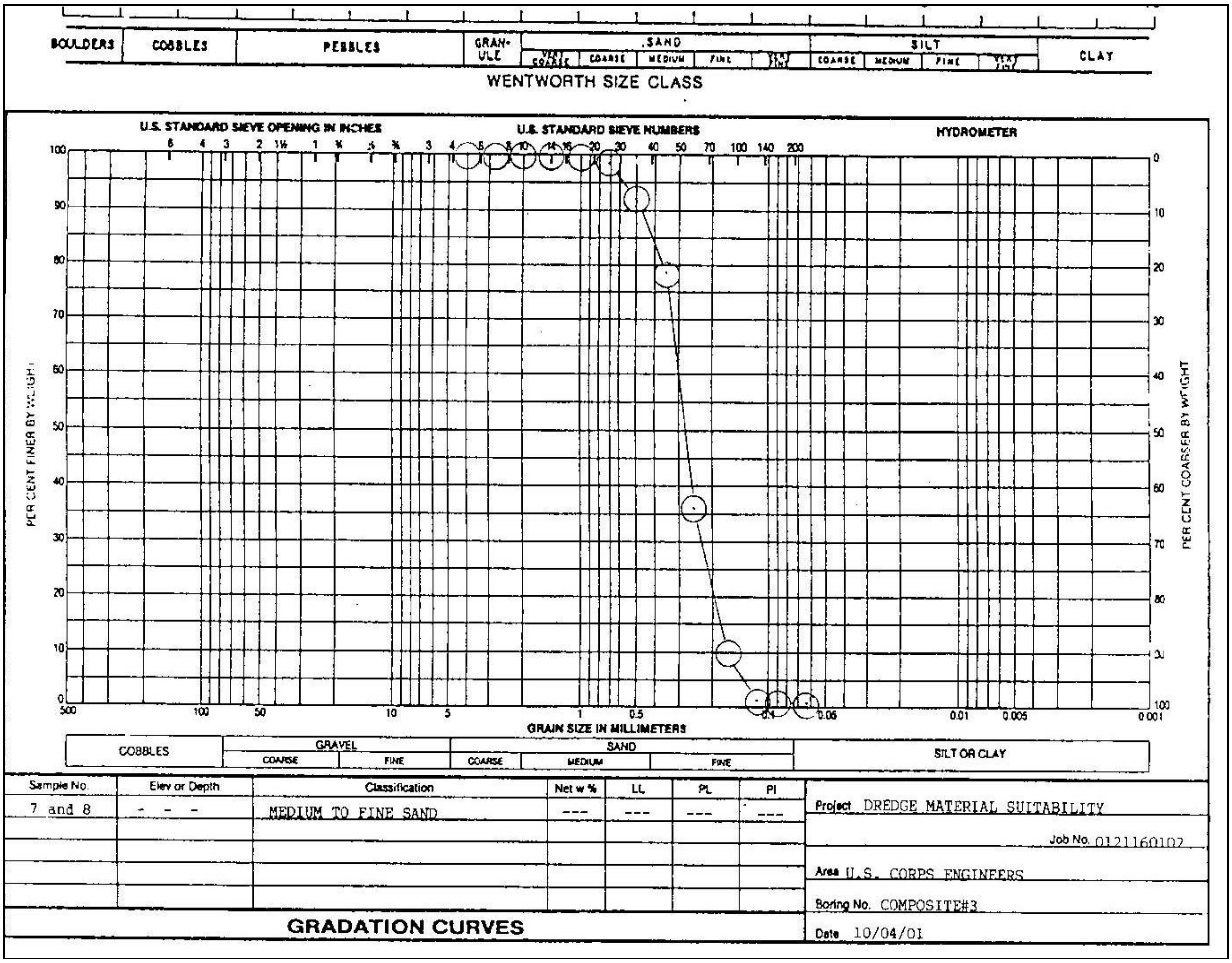


Figure 3.1.2.5-3. Composite gradation from sieve analysis of sand taken from the barrier islands of Alabama that is within the littoral drift zone of the Mississippi barrier islands.

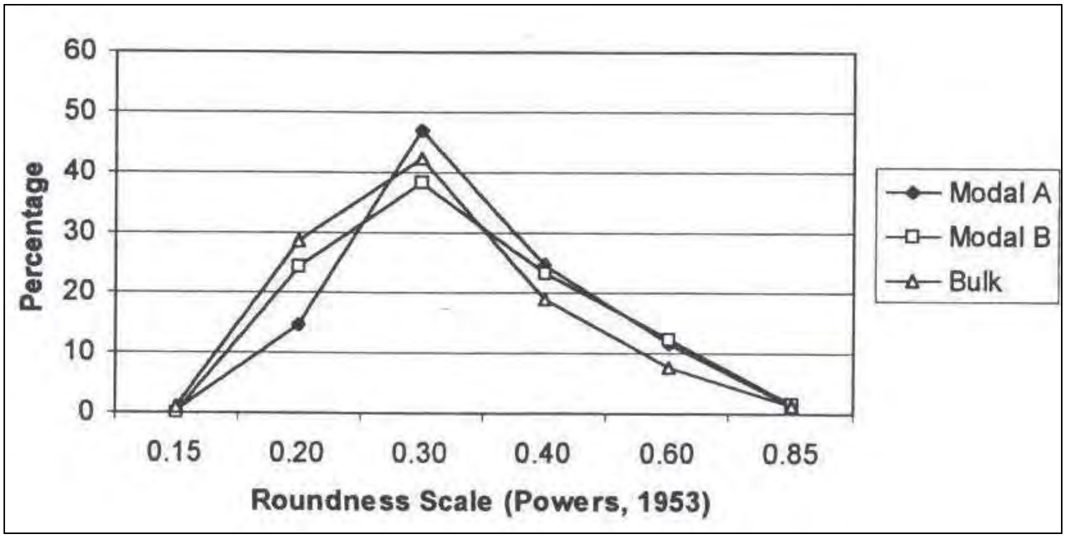


Figure 3.1.2.5-4. Grain Sphericity of composite sand sample taken the barrier islands of Alabama that are within the littoral drift zone of the Mississippi barrier islands Values for sphericity (roundness) are .15 - very angular, .20 - angular, .30 - sub-angular, .40 - sub-rounded, .60 – rounded, and .85 – well rounded.

These beaches are used for nesting by endangered sea turtles where grain size, particle shape and color of the sand are very important. The sand from inland river sources is not a perfect match to any of these criteria and its use was discounted for direct application on the islands. Using sand from the littoral drift zone around and between the islands would certainly be a good match, but it was generally felt that removing the quantity of sand required would be harmful to the future natural accretion of the islands in the future. Discussions with the USGS revealed that previous work by there agency has potentially identified a large source of high quality sand south of the existing islands. This source is a submerged chain of islands named St. Bernard Shoals, created when the sea level was lower in an interglacial period (see Figure 3.1.2.5-5). These islands are believed to have a sand of quality similar to what is found in the present day Mississippi islands and sufficient quantity to meet the needs of this option. Presently, limited geophysical profiling and samples have been completed, but additional work is being conducted by the USGS under a grant to the State of Mississippi by the Minerals Management Agency. This source is located approximately 45 miles south of the barrier islands and lies in about 60 feet of water.

3.1.2.5.3 Structural, Mechanical and Electrical

This option will have no structural, mechanical or electrical components.

3.1.2.5.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

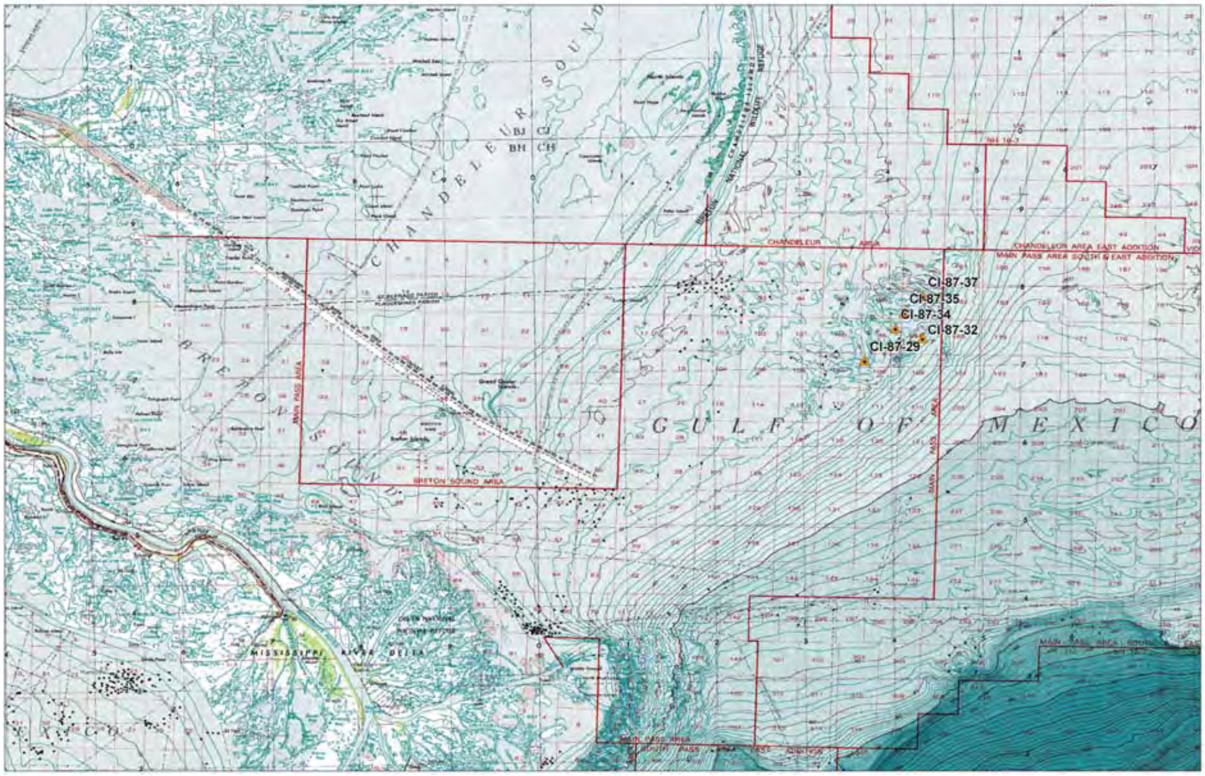


Figure 3.1.2.5-5. St. Bernard Shoals is shown as the area in the center right of the map with the numbered borings that were taken in the past to sample the sand sediments located there. Note the southern end of the Chandeleur Islands northwest of the Shoals.

3.1.2.5.5 Construction Procedures and Water Control Plan

To increase the size of the footprint of each island and restore them back to a pre-Camille footprint will involve several different operations, some of which can take place concurrently. The source of sand that has been designated as the potential borrow area will require additional investigation using both geophysical techniques and physical sampling. The sand is expected to be in submerged shoals that will have to be located and mapped prior to any removal of the sand. This will be completed during design and before the construction begins.

Each of the islands will require that a “dump basin” be excavated by dredging before any sand is transported from the borrow areas which is located about 45 miles south of the islands. These basin are required due to the depth of the water which is too shallow for the dredges to approach the islands. The basins will typically be located about one mile from the beach of the respective island where sand is being added to surrounding waters. These basins will be of sufficient size to allow a large quantity of sand to be stored after being bottom dumped from a hopper dredge. The material dredged from these basins is anticipated to be unsuitable for placement on the islands and is expected to be transported to permitted disposal areas. As each basin is completed, a hopper dredge can begin to remove sand from the borrow area and transport it to the basin where it can be quickly dumped, allowing the dredge to have minimal delays between trips. When the sand in a basin reaches a set capacity, a cutterhead, suction dredge will move the sand from the basin to the area where the sand is needed. Where needed, booster pumps will be utilized. The discharge from the suction dredge will be moved over the areas where the size of the island is being increased. As an area is filled to the desired grade, the sand will be shaped into dunes, basins and beaches. As this earthwork is completed for a given area, planting can begin. The suction dredge will be moved as needed to accommodate the excavation of the basins and the transfer of the sand from the basins to the islands. It is anticipated that the suction dredge will be moved, then remobilized several times during the entire process for completing an islands enlargement.

3.1.2.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.5.7 Operations and Maintenance

The placement of sand to increase the land mass of each of the islands will be a one-time event. Per an agreement with the National Park Service, no additional beach maintenance will be performed in the future. This project will provide a one-time supplement of the sand supply of the islands and the littoral system, after which, natural processes will be allowed to maintain and shape the islands in accordance with 2006 NPS Management Policies. Therefore, there will be no costs associated with operations and maintenance for this option.

3.1.2.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.11, Cost Summary. Total project costs for the various options are included in Table 3.1.2.11-1. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.5.9 Schedule for Design and Construction

This option will require extensive coordination with both state and Federal agencies to acquire the necessary permits that allow construction of this option. It is also anticipated that during the design process additional modeling will be required to assist in determining the most appropriate configuration of the additional land mass. Once the design is complete, construction may require several years due to the large quantity of sand that would be required and the distance from the borrow site to the island. Planting of vegetation can be concurrent with sand placement and shaping.

3.1.2.6 Option B – Replenish Sand in Littoral Zone, Inland Source

Another consideration to help restore the islands is to supplement the sand in the littoral system. This could be accomplished by adding sand in specific locations based on sediment transport modeling. This would allow the littoral currents to move the sand onto the islands where the natural process of island building could take place. This would not directly affect the present-day islands and would help mitigate any effects of dredging the ship channels that pass through the chain of islands where sand may have been lost from the system. Potential locations for sand placement are shown in Figure 3.1.2.6-1. NPS support for this option would be dependent on additional research, data collection, analysis and modeling, particularly with respect to sand compatibility and littoral zone placement.

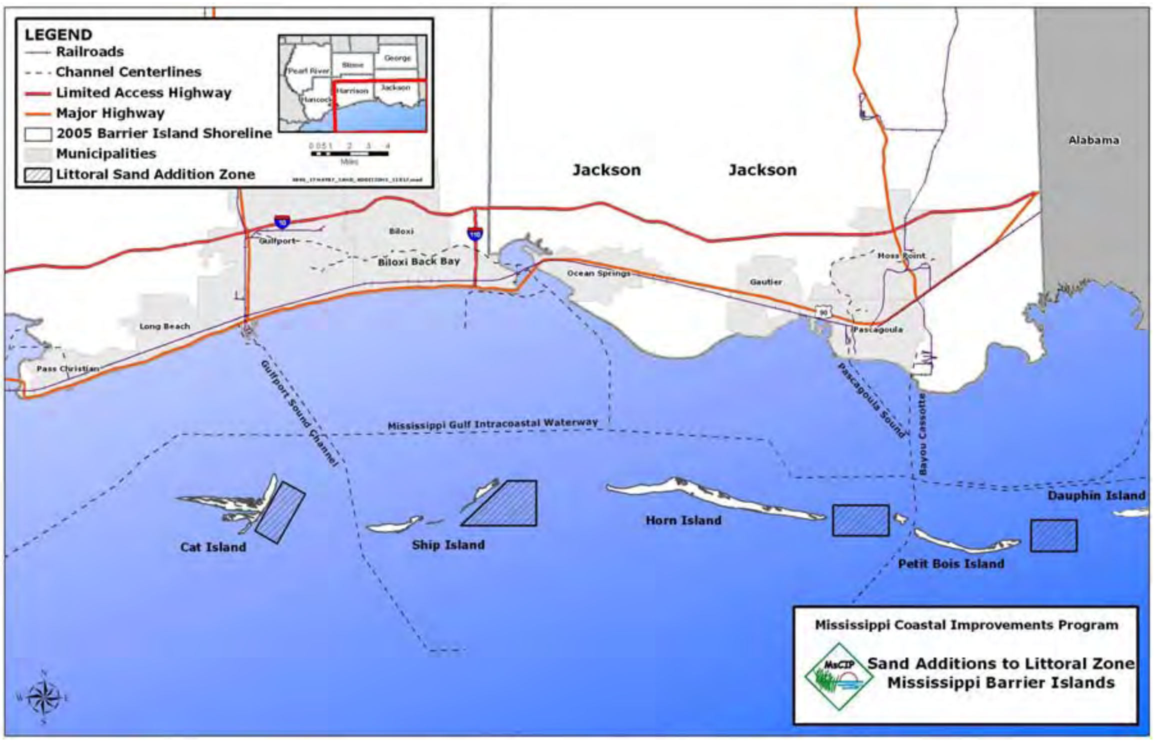


Figure 3.1.2.6-1. Potential areas for sand addition to the littoral drift zone at the Mississippi Barrier Islands. Actual locations would be based on sediment transport modeling.

As discussed in Part 1, the construction of inland waterways in Alabama and Mississippi has resulted in continuing maintenance dredging to maintain the channel depths and alignments. This dredged material is now accumulated in disposal areas along the banks of the river. Dredging of some of the areas along the river has produced large quantities of sand that have potential use for replenishment of littoral zones such as are found along the Mississippi Barrier Islands. An inventory of current disposal sites indicates that approximately 30,000,000 cubic yards of sand is 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 1.5-6 showed 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 among the islands littoral zone. The cost to store this type of dredged material is high and it has recently been estimated that removing the sand from the existing disposal areas would save the Government over

$100,000,000 at today's cost. This cost is based on a recent cost estimate for all costs, real estate, construction and mitigation, required to construct additional disposal areas.

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 than 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 more rounded than the river sand. The roundness of two typical samples of the river sand was described in the analyses shown in Figures 3.1.2.6-2 and 3.1.2.6-3. The majority of the sample is angular to sub-angular in particle shape.

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" (see Table 3.1.2.6-1). 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.

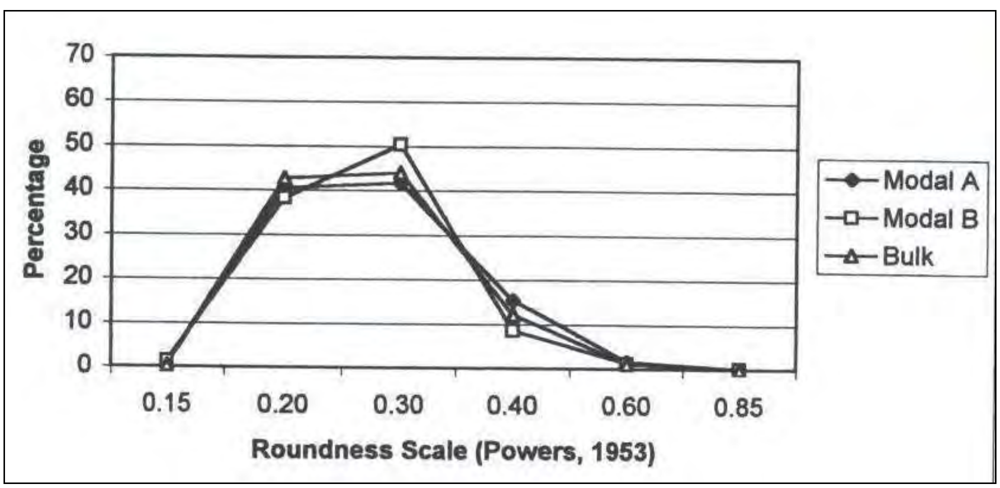


Figure 3.1.2.6-2. Grain Sphericity of composite sand sample taken from Baldbar disposal area on the Black Warrior – Tombigbee River system in Alabama. Values for sphericity (roundness) are .15 - very angular, .20 - angular, .30 - sub-angular, .40 - sub-rounded, .60 – rounded, and .85 – well rounded.

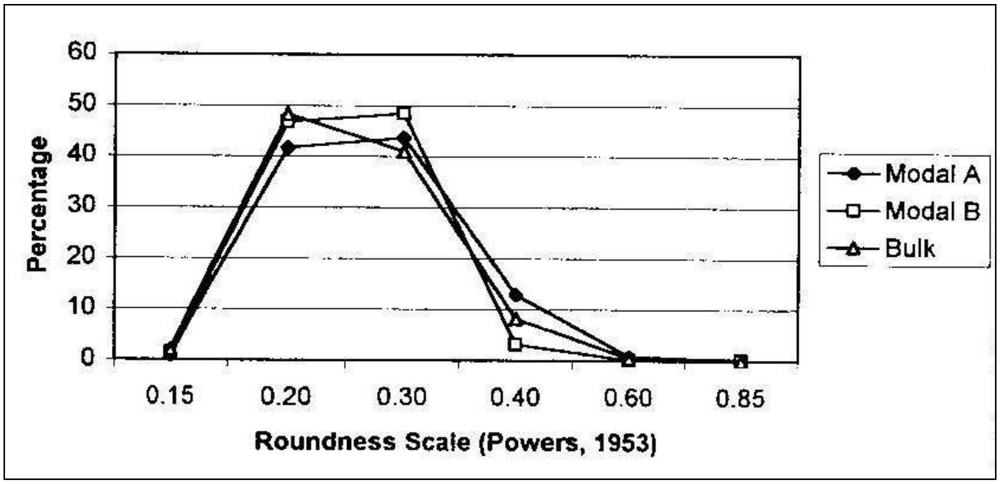


Figure 3.1.2.6-3. Grain Sphericity of composite sand sample taken from Buena disposal area on the Black Warrior – Tombigbee River system in Alabama. Values for sphericity (roundness) are .15 - very angular, .20 - angular, .30 - sub-angular, .40 - sub-rounded, .60 – rounded, and .85 – well rounded.

Table 3.1.2.6-1.
Munsell Soil Color Evaluation of Sand Samples Taken from the Alabama, Black Warrior and Tombigbee River Systems in Alabama

Sample ID	Hue	Value	Chroma	Color
Buena Vista 2 (surface)	10YR	7	3	Very Pale Brown
Buena Vista 2 (1.5” depth)	10YR	7	3	Very Pale Brown
Bald Bar/Big Sand	10YR	6	4	Light Yellow Brown
North Star Wreck	10 YR	7	4	Very Pale Brown

As discussed in Section 2.1, recent testing with a different type of abrasion process has concluded that the color of the sand is a grain surface staining should be removed as the sand abrades during littoral transport, (Baehr, 2007). The resulting sand should then be similar in color to the existing beaches. This process will be verified through additional controlled laboratory research and testing based on multiagency work group recommendations prior to any sand placement.

By spreading the sand over large areas to a small thickness, approximately one foot, the natural sediment transport process would blend the two sands together. The transport process may also tend to remove any staining from the sand grains and could help to round the individual particles through abrasion. Based on having 30,000,000 cubic yards of sand available, each of the islands was assigned a percentage of that quantity. This percentage was based on the amount of land loss (percentage of total loss) for each of the islands from pre-Camille to post-Katrina. The volumes of sand to be placed near each island are as follows:

Cat – 4,200,000 cubic yards

Ship – 9,600,000 cubic yards

Horn – 9,600,000 cubic yards

Petit Bois – 6,600,000 cubic yards

The entire process would consist of loading the sand onto river barges at the various disposal areas, moving the barges downriver and into the Mississippi Sound via tugboat tows, unloading the barges with a “hydraulic unloader”, and spreading the sand with a “spreader barge”. The process would require a continuous supply of loaded barges as the unloader only needs about an hour to remove the sand from a typical river barge. Staging this process from within the Mississippi Sound would also help with down time due to weather that would be more affected on the south side of the islands.

3.1.2.6.1 Interior Drainage

The type of work anticipated for adding sand into the littoral drift zone will not require any type of drainage system. The addition of sand under this operation will be with dredge pipe discharge into open water.

3.1.2.6.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. When the sea level returned to present condition, sediments filled the river channels and started to cover the area that would become the Mississippi Sound. Sandy deposits that had been transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with these deposits.

The actual Sound formed as an estuary after littoral drift of sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand is typically medium grained, white to light grey in color with a sub-angular to rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward. The estuary forming process was added by formation of the St. Bernard delta of the Mississippi River that enclosed the western end of the Sound. The western-most island in the chain, Cat Island, is a product of the historic St. Bernard delta lobe. What remains as Cat Island today is a beach front face of the island where waves have sorted the material leaving the sand and deltaic deposits behind the beach.

East and West Ship Island, Horn Island and Petit Bois Island are migrating within a littoral zone over Pleistocene formations that created a relatively stable platform for the constantly moving islands. By increasing the sand within the littoral zone, it would allow it to become subject to the same coastal processes that move the sand already in the system.

The beaches of the Mississippi Barrier Islands are used for nesting by endangered sea turtles where grain size, particle shape and color of the sand are very important. The sand from inland river sources is not a perfect match to these criteria, but if added into the existing system, it would be subject to the same forces that abrade the sand grains to a rounder particle shape. Using sand from the same littoral drift zone where the Mississippi Islands are located would certainly be a good match, but it was generally felt that removing the quantity of sand required would be harmful to natural accretion of the islands in the future.

3.1.2.6.3 Structural, Mechanical and Electrical

This option will have no structural, mechanical or electrical components.

3.1.2.6.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.6.5 Construction Procedures

To add off-site sand into the littoral system under this option, material from inland dredged material disposal sites would be transported by barge down the river system for use among the islands littoral zones.

Each of the areas designated for adding sand will require that a staging area where barges could be unloaded and the sand spread over the selected area. The sand would be transported from each of numerous disposal sites located up the river systems. The size of the locks on the river systems and the depth of associated channels will dictate the size of barges that can be used. As the barges are unloaded at each site, the sand would be pumped to spreader barges that would be able to cover an area sufficient to control the depth of sand placement.

3.1.2.6.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.6.7 Operations and Maintenance

The placement of sand into the littoral zone of each of the islands will be a one-time event. No additional beach maintenance is anticipated in the future, therefore, there will be no costs associated with operations and maintenance for this option.

3.1.2.6.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12, Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Project Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.6.9 Schedule for Design and Construction

This option will require extensive coordination with both state and Federal agencies to acquire the necessary permits that allow implementation of this option. It is also anticipated that during the design process additional sediment transport modeling will be required to assist in determining the most appropriate locations for the addition of sand into the littoral system. Once the design is complete, construction may require several years due to the large quantity of sand that would be required and the distance from the inland borrow sites to the island.

3.1.2.7 Option C – Replenish Sand in Select Littoral Zones, Offshore and Inland River Sources

Another consideration to help restore the islands is to supplement the sand in select littoral system zones with sand obtained from both inland river and offshore borrow areas. Like Option B, this could be accomplished by adding sand in specific locations based on sediment transport modeling. Potential areas where the sand may be added was shown in Figure 3.1.2.6-1, but for this option would be limited to the areas east of Ship Island and Petit Bois Island. These two areas were selected based on cooperation between the National Park Service (NPS, 2007) and the Corps of Engineers and is based on restoration policy of natural resources with the NPS. Both of these islands are affected by the presence of navigation channels that limit westward migration. Placement of sand into these two areas would add sediment into the system and would allow the littoral currents to move the sand onto the islands where the natural process of island building could take place. The sand that could be used in this option may come from the same offshore borrow area as Option A, the St. Bernard Shoals located about 45 miles south of the barrier islands and the lower inland river sand described in Option B. A hydrographic map showing the location of St. Bernard Shoals in relationship to the southern end of the Chandeleur Islands was shown in Figure 3.1.2.5-5. The sand from the inland river sources would be from the lower-most areas shown in Figure 1.5.6. NPS support for this option would be dependent on additional research, data collection, analysis and modeling, particularly with respect to sand compatibility and littoral zone placement.

The volume of sand that could be added into the littoral zone under this option could vary based on additional modeling, but for the volumes of sand to be placed near each island are as follows:

Ship – 5,000,000 cubic yards

Petit Bois – 4,000,000 cubic yards

These volumes were computed based on records from maintenance dredging for the Pascagoula Navigation Channel and represent that total volume less the sand that would be used to fill the breach between East and west Ship Island. The higher volume of sand for the littoral zone placement at the east end of East Ship Island was based on the professional judgement of a Multiagency group (including the NPS) that is working on the barrier island measures. These volumes could change based on additional sediment transport modeling that will assist in the exact placement locations.

3.1.2.7.1 Interior Drainage

The type of work anticipated for adding sand into the littoral drift zone will not require any type of drainage system. The addition of sand under this operation will be with dredge pipe discharge into open water.

3.1.2.7.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. Sandy deposits that were transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with deposits.

The actual Sound formed as an estuary after littoral drift of sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand is typically medium grained, white to light grey in color with well rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward. The estuary forming process was added by formation of the St. Bernard delta of the Mississippi River that enclosed the western end of the Sound. The western-most island in the chain, Cat Island, is a product of the historic St. Bernard delta lobe migrating across the historic littoral zone. What remains of Cat Island today is a T-shaped island with an east facing beach front face of the island where waves have reshaped the island and sorted the material leaving the east-west elongated sand ridges and deposits behind the beach.

East and West Ship Island, Horn Island and Petit Bois Island are migrating over Pleistocene formations that created a relatively stable platform for the constantly moving islands. Other Holocene deposits provide a relatively thin cover on the bottom of the Mississippi Sound and some areas south of the islands and consist of a muddy mixture of sand and clay along with shell fragments or buried oyster shell beds.

If increasing the sand within the littoral zone, it is desirable to maintain the same quality sand that now makes up the existing islands. Sources of sand in the quantity that would be required for this option are large, especially when considering the quality standard that must be met. Potential sources for this sand have potentially identified both offshore and from inland river sources. These are the same borrow areas that is being considered for Option A and B. Of concern is matching the sand being added to the littoral system to the physical characteristics of the sand on the beaches of the National Seashore. As discussed in Option A for the barrier islands, sand from the St. Bernard Shoals should be of similar quality to that presently on the islands. Discussions with the USGS revealed that this source is a submerged chain of islands created when the sea level was lower in an interglacial period. These islands are believed to have a sand of quality similar to what is found in the present day Mississippi islands and sufficient quantity to meet the needs of this option. This source is located approximately 45 miles south of the barrier islands and lies in about 60 feet of water. The sand from Option B may also be suitable for this option following further testing for compatibility.

3.1.2.7.3 Structural, Mechanical and Electrical

This option will have no structural, mechanical or electrical components.

3.1.2.7.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.7.5 Construction Procedures and Water Control Plan

To increase sand within the littoral zone from inshore and off-shore sources will involve several different operations, some of which can take place concurrently. The source of sand that has been designated as the potential borrow area will require additional investigation using both geophysical techniques and physical sampling. The offshore sand is expected to be dredged from submerged shoals that will have to be located and mapped prior to any removal of the sand. This will be completed during design and before the construction begins. The inland river sand will be loaded and brought down the river on barges for transportation to the area where it will be spread.

3.1.2.7.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.7.7 Operations and Maintenance

The placement of sand into the littoral zone of each of the islands will be a one-time event. No additional direct beach maintenance is anticipated in the future, therefore, there will be no costs associated with operations and maintenance for this option.

3.1.2.7.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12, Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.7.9 Schedule for Design and Construction

This option will require extensive coordination with both state and Federal agencies to acquire the necessary permits that allow implementation of this option. It is also anticipated that during the design process additional sediment transport modeling will be required to assist in determining the most appropriate locations for the addition of sand into the littoral system. Once the design is complete, construction may require several years due to the large quantity of sand that would be required and the distance from the inland borrow sites to the island.

3.1.2.8 Option D – Environmental Restoration w/ 2-foot Dune

This option would involve environmental restoration of the islands consisting of shaping existing sand into dunes on the beaches with planted vegetation and planting of maritime forests on the existing islands where they were mostly destroyed by Hurricane Katrina. Despite continual changes that occur, the barrier islands remain to buffer the mainland from storms and provide habitat for the rich, diverse wildlife residing within the area. On the southern portion of the islands, sea oats primarily, which are tolerant of high salt levels, thrive on the dune system which is located behind the beach area. Behind the primary dunes, trees and shrubs, such as short-leaf and long-leaf pines, can be found in the maritime forest. In the island interiors, emergent marshes collect fresh rainwater to help support its inhabitants. NPS support for this option is unlikely due to conflicts with that agency's 2006 Management Policies and statutory responsibilities.

Gulf Coast barrier islands and barrier spits can support stunted oak and yaupon shrublands. These scrub-scrub habitats are most often located on rises surrounded by black needlerush (*Juncus roemarianus*) salt marshes and have been reported from the Gulf Islands National Seashore (Natureserve Explorer 2002). Stunted slash pine may be present in the overstory, but most cover will be in a shrub layer dominated by yaupon, live oak, sand live oak, wax myrtle, saw palmetto, and salt bush (*Baccharis halimifolia*).

Immediately following Hurricane Katrina, most of the effort was spent protecting human life and securing structures throughout the impacted areas; therefore, few assessments of the vegetation impacts exist. For the barrier island system, most all of the vegetation recovered several months following Hurricane Katrina. The predominant vegetation that has long-term impacts consists of those pines found in the maritime forests. It is estimated that about 75% of these pine species were killed following the hurricane season of 2005, with most that attributable to Hurricane Katrina. The sea oats are still found in small patches due to the reduced dune system. Figure 3.1.2.8-1 is a photo of the south beach of Horn Island showing the lack of dunes and the damaged pine forest. An exception to the loss of vegetation is the emergent marsh habitat. It is thriving so well it actually looks as though hurricanes never past through the barrier island system.

One restoration option for the barrier islands would be to re-establish the vegetation that was destroyed by Hurricane Katrina. This option could involve restoration of the existing islands through adding sand dunes on the beaches along with planted vegetation (i.e. *Uniola paniculata*), planting of marshes (i.e. *Spartina alterniflora*, *Juncus roemerianus*, and Spartina patens) and maritime forests (i.e. *Pinus elliottii Engelm*, *Serenoa repens*, *Sabal minor*, etc.), and planting seagrasses (i.e. *Diplanthera wrightii*, *Cymodocea manatorum*, *Thalassia testudinum*, and *Ruppia maritime*) in the near-shore areas of the islands. Foremost, the vegetation would restore the island's natural setting, which allows for the diverse array of flora and fauna to persist. This plan would not involve adding any land mass to the islands other than the possibility of adding to the dune system. Vegetation would aid in reducing erosion from wind; thus helping in maintaining the stability of the islands. The vegetation would also aid in preventing erosion in the event that the islands gets overtopped by storm surge in a large hurricane.

Figure 3.1.2.8-1. Photo across the beach from the water on the south side of Horn Island. The wide, flat beach is now typical of the Mississippi Barrier Islands. The pine trees in the background are mostly dead, destroyed by the affects of Hurricane Katrina.

An environmental impact of the islands continuing to diminish in size is the increase in Mississippi Sound's salinity. Under current conditions, the islands provide a boundary between the sea water salinity [~33 parts per thousand (ppt)] of the open Gulf of Mexico and the brackish water found in the Sound. Salinity in the Sound during low flow periods range from 10 to 30 ppt. Highest salinities occur just south of Pascagoula and Gulfport and the lowest salinities in the Lake Borgne-Pearl River area. Loss of the islands would allow the salinity to greatly increase changing the ecological habitats that exist now. Mississippi Sound is one of the most productive systems on the Gulf coast. Changes in its salinity would impact not only fisheries but also the estuarine marshes, and the saltwedge in the area's rivers. This would impact shellfish and many other forms of marine life. Oysters currently found in concentrated Mississippi Sound areas would possibly cease to exist. At the Chandeleur Islands, loss in the island mass allows us to anticipate those potential environmental changes. Initial assessments are showing seagrasses diminishing, marsh erosion ongoing, and wave energy having no natural barrier.

The dune would be shaped from sand that would be removed from the surface between the constructed dune and the edge of the vegetation north of the dune. The dune would have a height of 2-feet, 1v to 3h slopes and a crest width of 6 feet. The dune would be continuous for the length of the gulf-side, south beach. While not designed as a structural defense against storms, the dune would be used as a platform to establish a line of sea oats that in turn would help in the natural process of creating larger and more pronounced sand dunes. The dunes would build with time as wind driven deposits of sand become trapped by the vegetation.

As previously discussed, the marsh grasses were not adversely affected by Hurricane Katrina. Island vegetation that was affected and would benefit the ecological community by a re-planting program is pine trees in the interior of the islands and sea oats on the beaches. The pines could be planted without any preparation, but the sea oats would benefit from a constructed dune to help become established. The quantities of vegetation for each island with a 2-foot constructed dune on the southern beach are shown in Table 3.1.2.8-1.

Table 3.1.2.8-1.
Quantities of Plantings for each Barrier Island

Island	2006 Acres	Pre-Katrina Acres Maritime Pine Forest	Replanting 75 Percent of Pine Forest Acres	Sea Oats - Planted 2-foot Dune Acres
Cat	1957	1% of Island	15 acres	6.3 acres
Ship (East & West)	631	3.7% of Island	18 acres	8.4 acres
Horn	3077	23% of Island	531 acres	23.4 acres
Petit Bois	1098	23%of Island	190 acres	13.2 acres

3.1.2.8.1 Interior Drainage

Interior drainage features are not applicable to the option.

3.1.2.8.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. When the sea level returned to present condition, sediments filled the river channels and started to cover the area that would become the Mississippi Sound. Sandy deposits that had been transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with these deposits.

The Mississippi Sound north of the islands formed as an estuary after littoral drift of the sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand that composes the islands is typically medium grained, white to light grey in color with well rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits, as shown in Figure 3.1.2.5-2, can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward. This process was added by formation of the St. Bernard delta of the Mississippi River that enclosed the western end of the Sound. The western-most island in the chain, Cat Island, is a product of the historic St. Bernard delta lobe. What remains today is a beach front face of the island where waves have sorted the material leaving the sand and deltaic deposits behind the beach. The islands, such as they exist today, are migrating along the littoral drift and are mostly composed of sand with local layers of peat-like organic soil that are forming in the inter-island lakes and marshes.

3.1.2.8.3 Structural, Mechanical and Electrical

Structural, Mechanical and Electrical is not applicable to this option.

3.1.2.8.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final selection of any sites associated with this option. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.8.5 Construction Procedures

This option will involve the planting of various types of vegetation in selected areas on the islands. Actual construction activities will take place only during the shaping of the small dunes on the beaches from existing the sand berm. Although the dune is of limited size, the total length of the dune construction will be approximately 30 miles for all the islands.

3.1.2.8.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.8.7 Operations and Maintenance

The initial planting of the various types of vegetation will have a warranty that will insure an approved survival rate. There will be no additional maintenance of the established plants under this option.

3.1.2.8.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12, Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.8.9 Schedule for Design and Construction

This option will require extensive coordination with both state and Federal agencies to acquire the necessary permits that allow implementation of this option. The actual design will be straight-forward with designated areas for the different types of planting vegetation and general guidance for the dune construction. The actual construction will require coordination with suppliers to furnish the large number of plants that are required for this option.

3.1.2.9 Option E – Environmental Restoration w/ 6-foot Dune

This option would involve environmental restoration of the islands consisting of shaping existing sand into dunes on the beaches with planted vegetation and planting of maritime forests on the existing islands where they were mostly destroyed by Hurricane Katrina. The sand required to construct a dune of this size would be more than could be removed from the existing beach berm and would come from the same offshore borrow area as the sand used in Option A. Placement of the sand would require moving the sand from a hopper dredge to a staging area on the beach, then moving the sand to the area of placement along the beach.

Despite continual changes that occur, the barrier islands remain to buffer the mainland from storms and provide habitat for the rich, diverse wildlife residing within the area. On the southern portion of the islands, sea oats primarily, which are tolerant of high salt levels, thrive on the dune system which is located behind the beach area. Behind the primary dunes, trees and shrubs, such as short-leaf and long-leaf pines, can be found in the maritime forest. In the island interiors, emergent marshes collect fresh rainwater to help support its inhabitants. NPS support for This option is unlikely due to conflicts with agency natural resources management policies.

Gulf Coast barrier islands and barrier spits can support stunted oak and yaupon shrublands. These scrub-scrub habitats are most often located on rises surrounded by black needlerush (*Juncus roemarianus*) salt marshes and have been reported from the Gulf Islands National Seashore (Natureserve Explorer 2002). Stunted slash pine may be present in the overstory, but most cover will be in a shrub layer dominated by yaupon, live oak, sand live oak, wax myrtle, saw palmetto, and salt bush (*Baccharis halimifolia*).

Immediately following Hurricane Katrina, most of the effort was spent protecting human life and securing structures throughout the impacted areas; therefore, few assessments of the vegetation impacts exist. For the barrier island system, most all of the vegetation recovered several months following Hurricane Katrina. The predominant vegetation that has long-term impacts consists of those pines found in the maritime forests. It is estimated that about 75% of these pine species were killed following the hurricane season of 2005, with most that attributable to Hurricane Katrina. The sea oats are still found in small patches due to the reduced dune system. An exception to the loss of vegetation is the emergent marsh habitat. It is thriving so well it actually looks as though hurricanes never past through the barrier island system.

One restoration option for the barrier islands with be to re-establish the vegetation that was destroyed by Hurricane Katrina. This option could involve restoration of the existing islands through adding sand dunes on the beaches along with planted vegetation (i.e. *Uniola paniculata*), planting of marshes (i.e. *Spartina alterniflora, Juncus roemerianus*, and *Spartina patens*) and maritime forests (i.e. *Pinus elliottii Engelm, Serenoa repens, Sabal minor*, etc.), and planting seagrasses (i.e. *Diplanthera wrightii, Cymodocea manatorum, Thalassia testudinum,* and *Ruppia maritime*) in the near-shore areas of the islands. Foremost, the vegetation would restore the island's natural setting, which allows for the diverse array of flora and fauna to persist. This plan would not involve adding any land mass to the islands other than the possibility of adding to the dune system. Vegetation would aid in reducing erosion from wind; thus helping in maintaining the stability of the islands. The vegetation would also aid in preventing erosion in the event that the islands gets overtopped by storm surge in a large hurricane.

An environmental impact of the islands continuing to diminish in size is the increase in Mississippi Sound's salinity. Under current conditions, the islands provide a boundary between the sea water salinity [~33 parts per thousand (ppt)] of the open Gulf of Mexico and the brackish water found in the Sound. Salinity in the Sound during low flow periods range from 10 to 30 ppt. Highest salinities occur just south of Pascagoula and Gulfport and the lowest salinities in the Lake Borgne-Pearl River area.

Loss of the islands would allow the salinity to greatly increase changing the ecological habitats that exist now. Mississippi Sound is one of the most productive systems on the Gulf coast. Changes in its salinity would impact not only fisheries but also the estuarine marshes, and the saltwedge in the area's rivers. This would impact shellfish and many other forms of marine life. Oysters currently found in concentrated Mississippi Sound areas would possibly cease to exist. At the Chandeleur Islands, loss in the island mass allows us to anticipate those potential environmental changes. Initial assessments are showing seagrasses diminishing, marsh erosion ongoing, and wave energy having no natural barrier.

The dune would be shaped from sand that would be removed from the surface between the constructed dune and the edge of the vegetation north of the dune. The dune would have a height of 6-feet, 1v to 3h slopes and a crest width of 6 feet. The dune would be continuous for the length of the gulf-side, south beach. While not designed as a structural defense against storms, the dune would be used as a platform to establish a line of sea oats that in turn would help in the natural process of creating larger and more pronounced sand dunes. The dunes would build with time as wind driven deposits of sand become trapped by the vegetation.

As previously discussed, the marsh grasses were not adversely affected by Hurricane Katrina. Island vegetation that was affected and would benefit the ecological community by a re-planting program is pine trees in the interior of the islands and sea oats on the beaches. The pines could be planted without any preparation, but the sea oats would benefit from a constructed dune to help become established. The quantities of vegetation for each island with a 6-foot high constructed dune on the southern beach are shown in Table 3.1.2.9-1.

Table 3.1.2.9-1.
Quantities of Plantings for each Barrier Island

Island	2006 Acres	Pre-Katrina Acres Maritime Pine Forest	Replanting 75 Percent of Pine Forest Acres	Sea Oats - Planted 6-foot Dune Acres
Cat	1957	1% of Island	15 acres	14.9 acres
Ship (East & West)	631	3.7% of Island	18 acres	19.9 acres
Horn	3077	23% of Island	531 acres	55.3 acres
Petit Bois	1098	23%of Island	190 acres	31.2 acres

3.1.2.9.1 Interior Drainage

Interior drainage features are not applicable to the option.

3.1.2.9.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. When the sea level returned to present condition, sediments filled the river channels and started to cover the area that would become the Mississippi Sound. Sandy deposits that had been transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with these deposits.

The Mississippi Sound north of the islands formed as an estuary after littoral drift of the sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand that composes the islands is typically medium grained, white to light grey in color with well rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward. This process was added by formation of the St. Bernard delta of the Mississippi River that enclosed the western end of the Sound. The western-most island in the chain, Cat Island, is a product of the historic St. Bernard delta lobe. What remains today is a beach front face of the island where waves have sorted the material leaving the sand and deltaic deposits behind the beach. The islands, such as they exist today, are migrating along the littoral drift and are mostly composed of sand with local layers of peat-like organic soil that are forming in the inter-island lakes and marshes.

3.1.2.9.3 Structural, Mechanical and Electrical

Structural, Mechanical and Electrical is not applicable to this option.

3.1.2.9.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final selection of sites associated with this option. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.9.5 Construction Procedures

This option will involve placement of dredged material onto the existing beaches and shaping the sand into low dunes as described. Other activities will involve the planting of various types of vegetation in selected areas on the islands.

3.1.2.9.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.9.7 Operations and Maintenance

The initial planting of the various types of vegetation will have a warranty that will insure an approved survival rate. There will be no additional maintenance of the established plants under this option.

3.1.2.9.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12, Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.9.9 Schedule for Design and Construction

This option will require extensive coordination with both state and Federal agencies to acquire the necessary permits that allow implementation of this option. The actual design will be straight-forward with designated areas for the different types of planting vegetation and general guidance for the dune construction. The actual construction will require coordination with suppliers to furnish the large number of plants that are required for this option. The quantity of sand required for this project, while not extremely large will require an off-shore source and could take considerable time to dredge, transport and place.

3.1.2.10 Option F – Environmental Restoration of Sea Grass Beds

This option would involve environmental restoration of the sea grass beds that have historically existed on the north side of the islands in the Mississippi Sound as shown in Figure 3.1.2.10-1. Despite continual changes that occur, the barrier islands remain to buffer the mainland from storms and provide habitat for the rich, diverse wildlife residing within the area. Knowledge of submerged aquatic vegetation (SAVs) is limited to reports by Humm (1956) and Humm and Caylor (1957) before the Gulf of Mexico Estuarine Inventory (GMEI) Study (1973). They reported the occurrence of five flowering species known as “seagrasses” and 77 algal species all along the Mississippi barrier islands. Studies carried out by the GMEI personnel revealed that there were about 17,000 acres of SAVs in Mississippi Sound.

High turbidity and lack of suitable substrate have limited distribution of SAVs in Mississippi. SAVs have been restricted to relatively quiet waters along the mainland and barrier island shores. Isolated patches occur only several hundred acres in size along mostly the northern portions of the barrier islands. In turbid waters of the Sound, seagrass beds are typically found in shallow water less than six feet in depth, most in two or less. With the exception of shoal grass (*Halodule wrightii*), which grows on hard sandy bottoms, the species characteristic of Mississippi Sound area prefer soft muddy substrates. A study of the Mississippi portion of Mississippi Sound by Eleuterius in 1969 indicated that about 17,000 acres of SAVs were present including turtle grass (*Thalassia testudinum*), manatee grass (*Cymodocea manatorum*), shoal grass, *Halophilia engelmanni* (no common name), and widgeon grass (*Ruppia maritima*). In 1969, Hurricane Camille destroyed the majority of SAVs along the Mississippi Gulf coast (Eleuterius 1973). Moncreiff (1998) identified the northern shorelines of Ship, Horn, and Petit Bois Islands as potential habitat for seagrass beds. These areas have historically supported populations of shoal grass, *Halophila engelmannii*, manatee grass, and turtle grass. Currently, these locations only appear to support beds of shoal grass. In areas where SAVs are present, significant quantities of benthic and epibenthic macroalgae are found, such as red, brown, and green species.

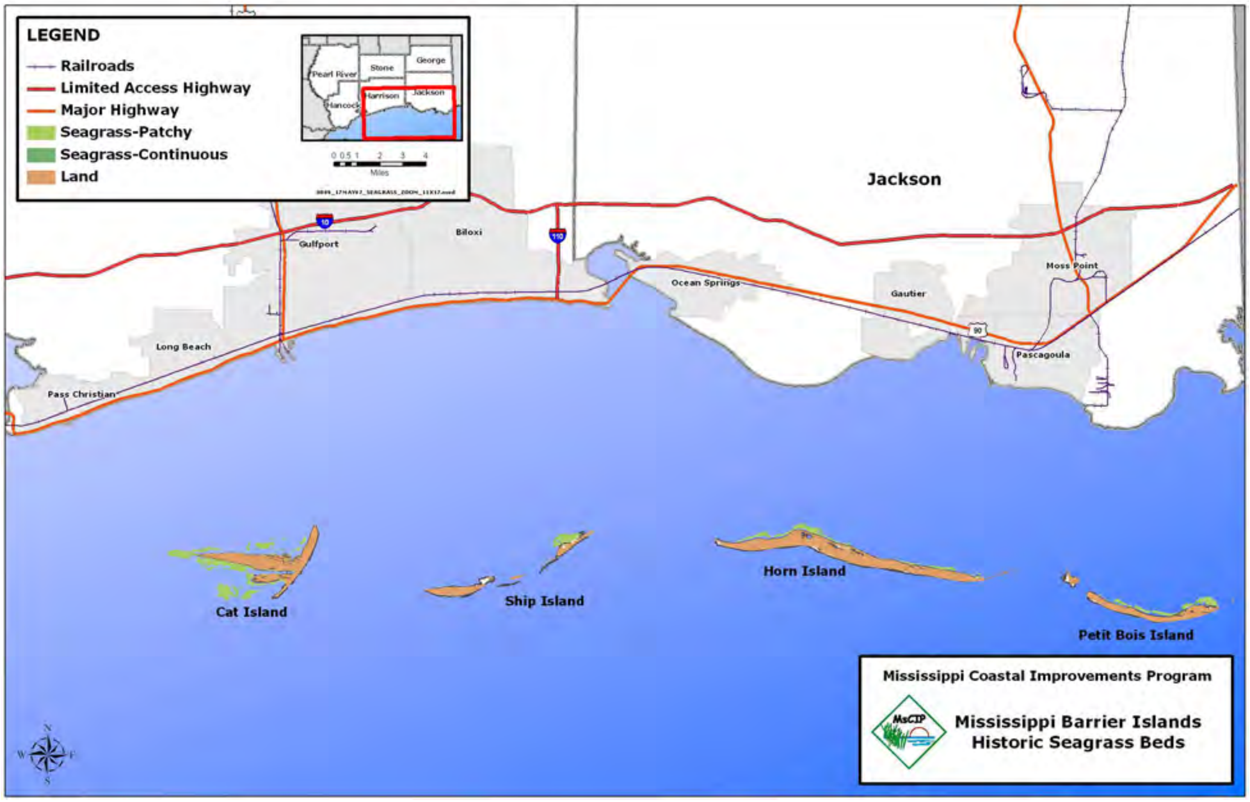


Figure 3.1.2.10-1. Location of Historical Sea Grass Beds near the Mississippi Barrier Islands

The Mississippi Department of Marine Resources (DMR) has provided the estimated pre-Camille acreage of the grass beds and the current amount of beds that exist today. The types of grass that would be planted include Diplanthera wrightii (i.e Shoal Grass), Cymodocea manatorum (i.e. Manatee Grass), Thalassia testudinum (i.e. Turtle Grass) and Ruppia maritima (i.e. widgeon grass). The planting would occur at selected locations in coordination with DMR and would cover 50 percent of the historical acreage. Due to the large number of plants required for this option, the supply of available stock would have to be matched to the planting schedule. The amount of acres of sea grasses to be planted at each island, based on 50 percent of pre-Camille acreage, is as follows:

Cat – 210 acres

Ship – 760 acres

Horn – 2,650 acres

Petit Bois – 780 acres

3.1.2.10.1 Interior Drainage

Interior drainage is not applicable to this option.

3.1.2.10.2 Geotechnical Data

The actual Sound formed as an estuary after littoral drift of sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. East and West Ship Island, Horn Island and Petit Bois Island are migrating over Pleistocene formations that created a relatively stable platform for the constantly moving islands. Other Holocene deposits provide a relatively thin cover on the bottom of the Mississippi Sound and some areas south of the Islands and consist of a muddy mixture of sand and clay along with shell fragments or buried oyster shell beds.

3.1.2.10.3 Structural, Mechanical and Electrical

Structural, Mechanical and Electrical is not applicable to this option.

3.1.2.10.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste at the sites. These studies, if deemed necessary, will be conducted during the next phase of work after the final selection of sites. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.10.5 Construction Procedures

This option will only involve the planting of various types of marine aquatic vegetation in selected areas around the islands. No actual construction activities will take place. The extremely large quantity of plants required for this type of project would require that the project would have to have an extended project life to allow the procurement of the vegetation that would not be readily available in today's market.

3.1.2.10.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.10.7 Operations and Maintenance

The initial planting of the various types of sea grass will have a warranty that will insure an approved survival rate. There will be no additional maintenance of the established plants under this option.

3.1.2.10.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12, Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.11 Option G – Restore Ship Island Breach

The most predominate affect of Hurricane Katrina on the Mississippi Barrier Islands was the large increase in size of the breach in Ship Island commonly known as the Camille Cut, (see Figure 3.1.2.11-1). This photo was taken after Hurricane Katrina, but, would be similar to conditions after Hurricane Camille.

The pre-Camille footprint of Ship Island was obtained from historical records and this data shows the area that was breached during Hurricane Camille now forming two separate islands, West and East Ship Island. Two major historic sites, one on each island, are in danger from the continuing erosion of the barrier islands. Current studies by the Corps indicate that restoring the two islands to a single island, pre-Camille condition may prevent the rapid erosion of the beaches that is now occurring as well as potentially helping to provide wave erosion on the mainland. Estimates indicated that the total restoration of Ship Island to a pre-Camille footprint, single land mass off the Mississippi coast will involve approximately 21 million cubic yards of sand. Other variances of filling only the breach and some areas along the northern shores with lesser quantities of sand may also provide opportunity for a natural healing of the island. This limited sand placement, approximately 13,000,000 cubic yards, has the support of the NPS (NPS, 2007) and will be the basis of this option. This volume is based on computing the the sand needed to fill the breach to a 1,000-foot width and to a elevation of 2.0. The total volume of sediment removed during all historical maintenance dredging for the Pascagoula Navigation Channel was compiled and the balance of that total will be used for littoral zone placements under Option C as previously described. As happened during Hurricane Camille, the breach was opened during Hurricane Katrina leaving two islands with approximately three miles of open water between the remaining portions. This portion of the island has also been breached during other prior hurricanes and while most of the island has reformed to a low bar over time, it never gained enough sand to form dunes and establish vegetation along this center portion. Consequently, even small storms easily washed over and eroded this part of the island and reopened the breach. Natural healing from the littoral drift is hindered by the large amount of sand that must rebuild the bar across the breach from the east. This is further aggravated by the fact that Ship Island is the last island in a littoral system that extends westward from its main source of sand on the panhandle of Florida, a distance of about 250 miles. Numerous opportunities exist along this pathway for the amount of sand in the system to be diminished. An additional consideration is the ebb tidal flushing in the deeper portion of the pass just east of West Ship Island when sand is moved southward thus starving the northern shore of West Ship Island. To mitigate this problem, the breach could be filled as single operation with planted dune vegetation that will become established and promote stable dune growth. With an understanding that all barrier islands are dynamic in nature and change constantly, the object of restoration would be to establish the island with sufficient sand mass and enough vegetation to again have the island as a somewhat stable member in the island chain. Fort Massachusetts located on the northern shore of West Ship Island and the French Warehouse located on the northern shore of East Ship Island would benefit from this option. Both of these sites are endangered by on-going erosion of the shoreline with Mississippi Sound. Another site, known as the Quarantine Station, has already been lost to erosion as shown in comparing Figure 3.1.2.11-1 and Figure 3.1.2.11-2.

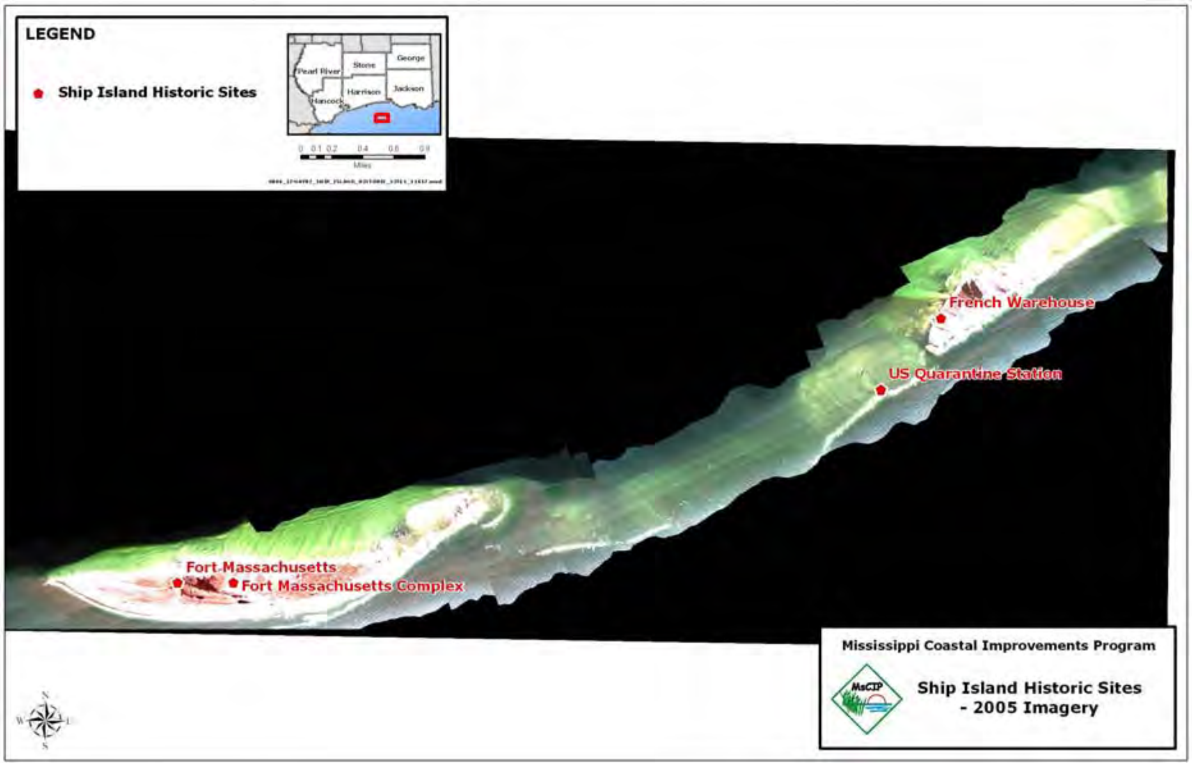


Figure 3.1.2.11-1. Aerial photo of West and East Ship Island taken in 2005 after Hurricane Katrina showing the locations of listed historical sites.

Figure 3.1.2.11-2. Aerial photo of West and East Ship Island taken in 2001. Note the sand spit extending westward from East Ship Island and the pass between the two islands.

Fort Massachusetts was originally built on the western tip of Ship Island. The westward migration of sand along the southern shore and erosion of the northern shore now has put the fort almost a mile from the western tip of the island, but dangerously close to being in the Sound (see Figure 3.1.2.11-3). Several emergency beach re-nourishments have taken place over the last 35 years through use of the beneficial use of dredged material from maintenance of the federally authorized Gulfport Harbor Navigation Project to protect the fort from wave action during winter storms. At present, the NPS is again requesting that the Corps place sand along the shore near the fort in conjunction with dredging operations at the Federal Gulfport Harbor navigation channel. This emergency placement of sand is being repeated about every five to six years. Figure 3.1.2.11-3 shows that in July, 2007, the north side of the fort showing and the relationship of the shore to the structure. Note the small jetty that has created severe scour at the down-current end.

Figure 3.1.2.11-3. Photo of erosion on north side of Fort Massachusetts showing relationship to encroaching waters to the structure. Note the small jetty that has created severe scour at the down-current end.

The French Warehouse site has not had any sand placement on its shoreline in the past. The erosive process is slower at that location, but now there are concerns from the NPS about the integrity of the site. Unlike the location of the fort, the warehouse site is covered by maritime forest which may be contributing to slowing the erosion of the shore due to the vegetation and the higher surface elevation.

The filling of the Camille Cut to provide a longer term solution to the erosion on the northern shores will require modeling to better understand the benefits that are believed to be associated with this plan. The costs will be substantial due to the large quantities of high quality sand that will be required to fill the breach. Initial estimates for sand requirements are approximately 8 million cubic yards. The fill would be expected to prevent the continuing loss of sand to West Ship Island, but it is also understood that the islands are a dynamic system, ever changing to nature’s forces. As well as the sand placement, this plan would include planting sea oats or other dune grasses to restore dune habitat on the newly created land. The volume of sand estimated for this project is based on an assumed average water depth of 5-feet in the existing breach.

There are many characteristics for the sand that must be considered during the design of the projects. Ideally, any sand used for beach construction or re-nourishment would come from the same littoral system so it would have the same gradation, particle shape and color. Ship Island is but one of many barrier islands that extend westward toward Louisiana that is located within a continuous littoral drift zone originating in Florida. The sand that migrates along this drift zone could be envisioned as moving from one island to another over very long periods of time. With this in mind, any sand of similar quality that is added into the drift zone would become part of the migration and be mixed with existing material. This added sand would also be available to the islands as a source for their beaches during the natural process of aggradations.

Sand of sufficient quality in the quantities required for this type of project is not known to occur in close proximity to the islands. Proposed geophysical studies may locate sources near the western end of West Ship Island, but this source has not yet been confirmed. Review of literature indicates that suitable sand can be obtained from St. Bernard Shoals which is a chain of submerged barrier islands that are located about 45 miles south of Ship Island. This sand should be very high quality material and could be used in the island reconstruction Prior studies of the St. Bernard Shoals (Oral Communication, USGS, 2006) are probably the best source of the sand. Additional studies and sampling will be required to ensure the source. As previously described, St. Bernard Shoals are a series of submerged barrier islands. The average water depth over the shoals is 60 feet which puts the sand within reach of a hopper type dredge, however the water depth near the islands is shallow for the draft of hopper dredge that would be used in this type of operation. In order to accomplish this, the dredge will have to pump-off from an offshore location.

Another source of sand could be sand from inland river systems. This sand could be considered as a source for direct placement, but the material stored on the lower Tombigbee River would require additional testing of physical characteristics to assure it meets the required quality standards. As discussed under Option B, dredging of the inland rivers produces large quantities of well sorted sand that may have potential use for sand replacement as described above. An inventory of current disposal sites on the Mobile River system indicates that approximately 30,000,000 cubic yards of sand is 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 lower Tombigbee River which contain over 8,000,000 cubic yards of sand. Material from these sites could easily be transported by barge down the river system for use.

The sand selected for use, regardless of the source, would have a quality control program to ensure that it meets any established criteria prior to placement. The existing breach on Ship Island is approximately three miles in length. With an average water depth of five feet, an island width of approximately 1,000 feet the project will take 8,000,000 cubic yards of sand including a typical 30% loss of material during placement. Some of this material would also be placed along the north shore on either side of Camille Cut to repair existing erosion. Planting of the newly created land surface would be initiated when placement progress allowed. The planting would include dune grasses in two strips, one on each shoreline. The planting would consist of plants on 30-inch centers with the width of the planted strips set at 60-feet. The planted strips would extend along all shorelines where new beach is being created. With time, the dunes grasses will trap wind-blown sand and create dunes. The newly formed land mass will transform itself into a more natural state as wind shifts the sand and the planted vegetation establishes dunes similar to the beach scene shown in Figure 3.1.2.11-4.

Figure 3.1.2.11-4. Typical Mature Dands Funes on Gulf Coast Barrier Island

This potential option as a stand-alone measure will not provide any appreciable storm surge benefits based on modeling of the islands, but will provide benefits from storm induced wave damage on the shoreline. In addition, the role of the islands in maintaining the ecology of the Mississippi Sound has been realized and this alone may well be justification for additional study of filling Camille Cut. With this area under the control of the NPS, their endorsement is valuable to continued study.

3.1.2.11.1 Interior Drainage

The type of work anticipated for adding sand to increase the land mass of the islands will not require any type of drainage system. The addition of sand under this operation will be with dredge pipe discharge and all water will be allowed to run back to the sea.

3.1.2.11.2 Geotechnical Data

The barrier islands are composed of Holocene aged deposits, mostly sand. These deposits formed after erosion of the Pleistocene formations during the last regression and transgression of the sea. This occurred during the Wisconsinan glacial stage during the Late Pleistocene. As the sea regressed, rivers incised channels and transported sediments southward. When the sea level returned to present condition, sediments filled the river channels and started to cover the area that would become the Mississippi Sound. Sandy deposits that had been transported into the Gulf began to move westward from northwest Florida as wind driven littoral currents formed numerous barrier islands across the northern Gulf Coast, including most of those in coastal Mississippi. As the sea level continued to rise, the bays and associated river channels into the gulf also began to fill with these deposits.

The actual Sound formed as an estuary after littoral drift of sandy sediments from the Alabama coast formed a shoal south of the Mississippi mainland. These shoals became barrier islands as currents, waves and wind pushed the sand above the water surface. The sand is typically medium grained, white to light grey in color with well rounded particle shape. Within the interior of the islands, marshes and fresh water lakes have created highly organic soils with a peat-like character. These deposits, as shown in Figure 3.1.2.11-5, can be observed as beach outcrops on the southern shore of East Ship Island after the island has migrated northward.

Figure 3.1.2.11-5. Peat-like organic soils outcropping on the south beach of East Ship Island. These deposits are the remains of sediments and organic matter that settle in the bottom of the marshes and lakes that occur on the barrier islands. The deposits are exposed as the islands migrate northward.

East and West Ship Island are migrating over Pleistocene formations that created a relatively stable platform for the constantly moving islands. Other Holocene deposits provide a relatively thin cover on the bottom of the Mississippi Sound and some areas south of the Islands and consist of a muddy mixture of sand and clay along with shell fragments or buried oyster shell beds.

If increasing the land mass of the islands, it would be desirable to maintain the same quality sand that now makes up the existing islands. Sources of sand in the quantity that would be required for this option are extremely large, especially when considering the quality standard that must be met. Potential sources for sand were investigated both inland and offshore. Of concern is matching the sand to the sand on the beaches of the National Seashore. Samples taken from Dauphin and Pelican Island in Alabama are in the same island chain and have been tested for color, grain size and particle shape. These results, included in this section, can be used to match potential sand sources.

3.1.2.11.3 Structural, Mechanical and Electrical

This option will have no structural, mechanical or electrical components.

3.1.2.11.4 HTRW

Due to the extent of the islands and lack of prior development, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The construction costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.1.2.11.5 Construction Procedures and Water Control Plan

Prior to any additional detailed design, this project will require extensive modeling to predict the effects of partially or completely filling the breach. The modeling will be conducted to assist in location of sand placement, quantities of sand that may be required for a partial filling, and to help predict the amount of sand that would be required for future re-nourishment of the island's north shore.

To fill the breach and associated shorelines will involve several different operations, some of which can take place concurrently. The source of sand that has been designated as the potential borrow area will require additional investigation using both geophysical techniques and physical sampling. The sand is expected to be in submerged shoals that will have to be located and mapped prior to any removal of the sand. This will be completed during design and before the construction begins.

3.1.2.11.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

The lowest level of physical security (Level 1) was selected for use in this study. Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.1.2.11.7 Operations and Maintenance

The direct placement of sand to fill Camille Cut and will be a one-time event. Per an agreement with the National Park Service, no additional beach maintenance will be performed in the future. This project will provide a boost in the existing sand within the littoral system, then in accordance with the 2006 NPS Management Policies, nature will take it course. Therefore, there will be no costs associated with operations and maintenance for this option. Changes in future maintenance dredging practices at both Gulfport and Pascagoula Navigation Channels will ensure that more sand in the littoral zone will be available for natural beach building. This option will not preclude the NPS from performing sand additions at Fort Massachusetts or the French Warehouse to protect these structures from erosion of beaches that endangers these historic sites.

3.1.2.11.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.1.2.12 Cost Summary. Total project costs for the various options are included in Table 3.1.2.12-1. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.1.2.11.9 Schedule for Design and Construction

This project will require additional study and investigation to verify borrow areas. These can be accomplished within a one-year time frame after funding at which time placement of sand can be initiated pending all required permits.

3.1.2.12 Cost Estimate Summary

The total project costs for all options are shown in Table 3.1.2.12-1. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.1.2.12-1.
Summary of Total Project Costs

Option	Total Project Costs
Option A – Restoration of Island Footprints	$942,200,000
Option B – Replenish Littoral Zone w/ Inland River Sand	$1,013,800,000
Option C – Replenish Littoral Zone w/ Off-shore Sand	$147,400,000
Option D – Environmental Restoration w/ 2-foot Dune	$14,200,000
Option E – Environmental Restoration w/ 6-foot Dune	$39,200,000
Option F – Environmental Restoration of Sea Grass Beds	$264,500,000
Option G – Restore Ship Island Breach	$181,400,000

Note: There are no Operational and Maintenance costs for the barrier island options.

3.1.2.13 References

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3.2 Line of Defense 2 – Beach/Dune Construction

3.2.1 General

The Mississippi Mainland shoreline extends approximately 68 miles, and is divided into three coastal counties: Jackson, Harrison, and Hancock Counties, Figure 3.2.1-1. The Mississippi coast beaches are a valuable asset and provide vital environmental, cultural, recreational, and economic resources; they assist in maintaining the health and productivity of adjacent waters and provide for diverse cultural and recreational activities. They are also important in limiting infrastructure damage and providing protection to the seawalls along the coast (Schmid 2002). This study evaluated berm and dune options for approximately 35 miles of shoreline along the three Mississippi coastal counties as outlined in Figure 3.2.1-1. The coastal processes modeling analysis to evaluate the future without and with project berm and dune systems were conducted through application of the engineering-economic model Beach-fx. The purpose of the analysis was to evaluate the physical performance of the beach and dune system for anticipated future without-project and with project conditions. The development of the coastal processes input data and physical performance results of the Beach-fx analysis are provided in detail in Section 2.3 of this report. For this study, the exploration of the costal processes and economic inventorying was conducted. Further study would be required to combine the observed data and to evaluate the eleven alternatives previously mentioned. More detail on the further study can be found in the MsCIP Comprehensive Plan Main Report.

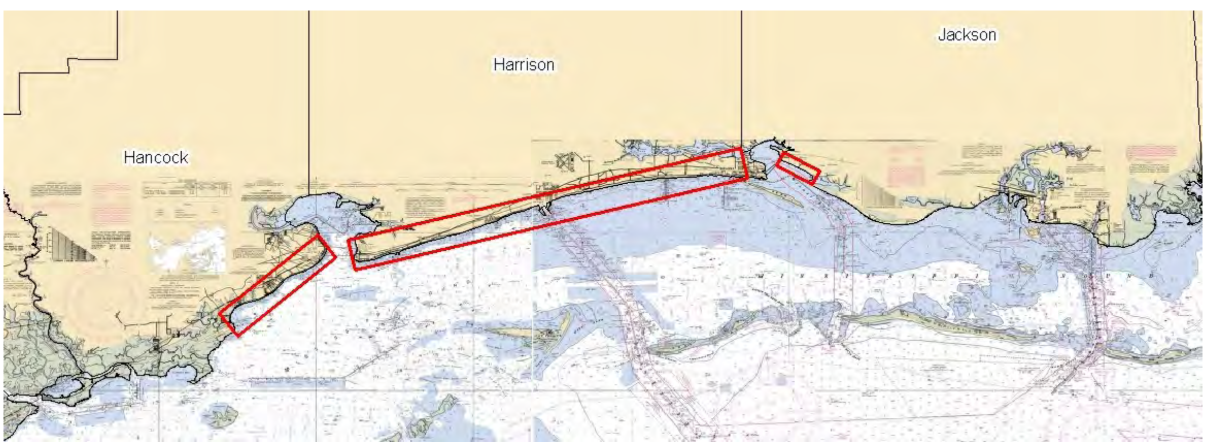

Figure 3.2.1-1. Project Location, Mississippi Coast Beach Evaluations

3.2.2 Hancock County Beaches

3.2.2.1 General

The purpose of this section is to provide engineering information and data for the planning and design of shore protection and restoration to the shoreline along Hancock County, MS following impacts from Hurricane Katrina, 29 August 2005. Hurricane Katrina severely damaged approximately six miles of public beaches along the shoreline from the US 90 bridge extending southwest to Beach Road.

3.2.2.2 Location

The Mississippi mainland shoreline is divided into three coastal counties: Jackson, Harrison, and Hancock Counties. Hancock County, Figure 3.2.2-1, is the western-most coastal county in Mississippi and is located approximately 95 miles west of Mobile, Alabama and approximately 40-miles east of New Orleans, Louisiana. Hancock County is bordered to the east by Harrison County, MS, and to the west by the Mississippi-Louisiana state line. The County consists of two municipalities: Bay St. Louis and Waveland. The beaches along the Hancock County shoreline, Figure 3.2.2-1, are separated in two sections: the reach extending approximately 6-miles from Grand Bayou in Waveland to the US 90 bridge in Bay St Louis, and the reach extending northeastward approximately 1-mile from Cadet Bayou.

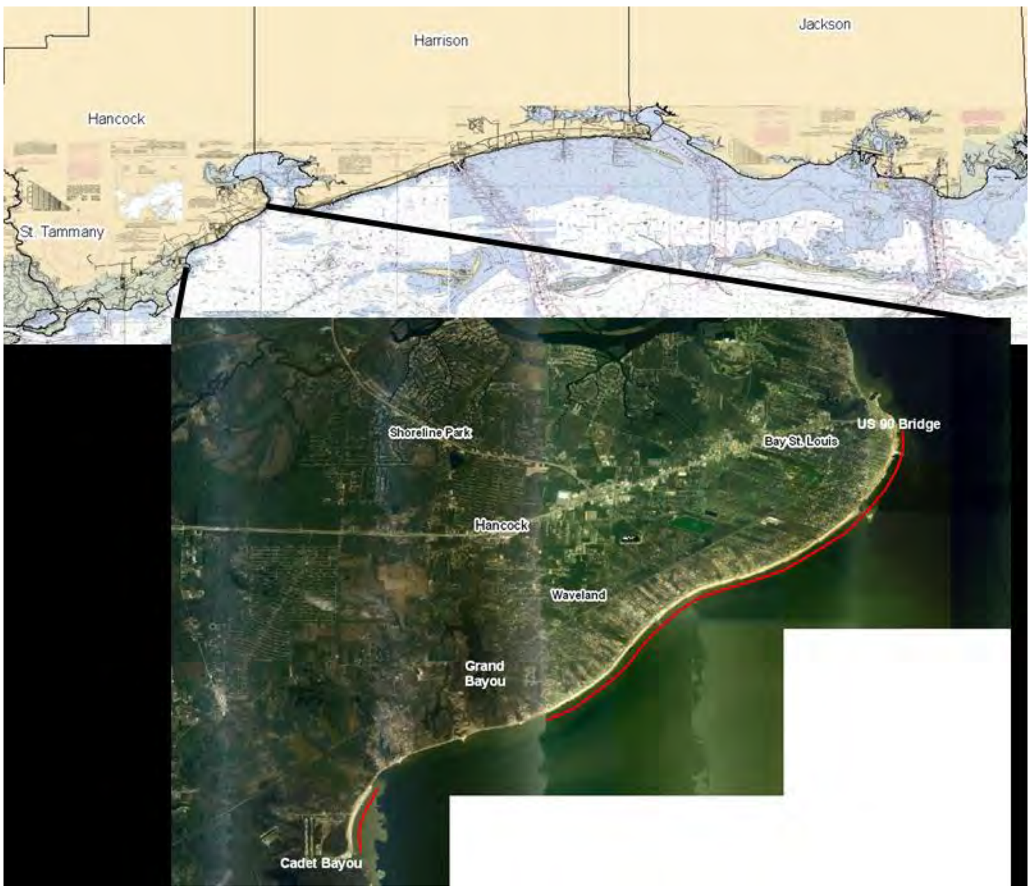

Figure 3.2.2-1. Project Location, Hancock County Beaches

3.2.2.3 Existing Conditions

The Hancock County shoreline south of the US 90 bridge is protected by an 8 mile long, approximately+ 5 ft elevation seawall extending from the US 90 bridge to Cadet Bayou. The Hancock County beaches were constructed for shore protection; however, the area provides added outdoor recreation and environmental benefits. The area experienced wave and wind erosion and is therefore periodically maintained or renourished with sand. The elevation of the seawall ranges between +3.8 and +5.0 feet (NAVD). The seawall fronting the downtown Bay St Louis beaches is significantly higher. A sand beach was constructed along approximately 6 miles of the seawall in 1967 as part of the emergency repair and protection following Hurricane Betsy (September 1965).

The approximate 1 mile section of beach fronting the downtown Bay St Louis area was constructed during the construction of the US 90 bridge. The 1 mile section extending from Bayou Cadet was constructed in 2005.

The Hancock County beaches were renourished in 1994 with material from a borrow area located approximately 1000 feet offshore. The beaches fronting downtown Bay St Louis, the northeast section of the beaches, were again renourished in 1996 with material from a borrow area located on the north side of the US 90 bridge. After renourishment, beach width was maintained by scraping upper portions of the beach and moving sediment to widen the beach (Schmidt 2002).

The existing Hancock County beach profile consists of a berm only feature which extends approximately 150 ft from the seawall to the Mississippi Sound. The berm elevation varies from approximately 5.0 ft at the seawall to 3.5 ft at the slope break to the Mississippi Sound. The downtown Bay St Louis area beaches include a bluff with an elevation of about +12 feet. Access ramps and pavements are located along the beach, and storm water culverts pass beneath the roadway adjacent to the beach to the shoreline to drain sections of Hancock County. A typical cross section for the existing condition is shown in Figure 3.2.2-2.

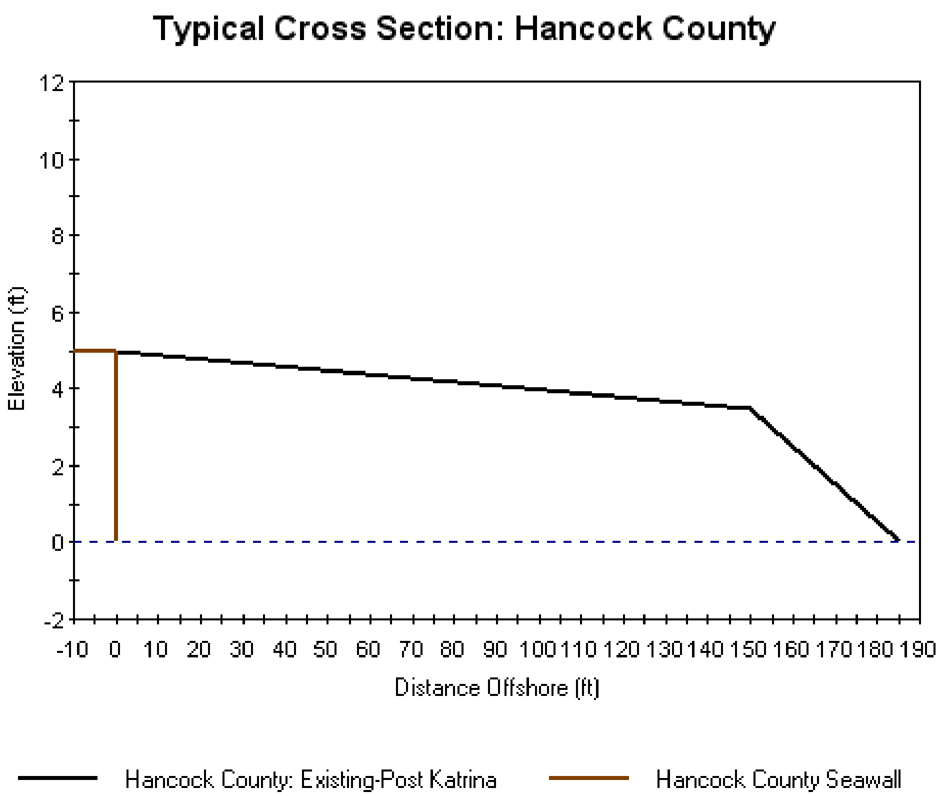

Figure 3.2.2-2. Typical Cross Section, Hancock County Existing Conditions

3.2.2.4 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 ft 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-Hurricane Katrina high water mark measurements in the area suggest storm surges on the order of 20 to 25 feet or more.

Transport is generally from northeast to southwest, although there are areas with reversals. From 1994 and 2000, 60 percent of the shoreline eroded at least -5.0 feet/year (ft/yr) which corresponds to volumetric losses of approximately -12,000 CY/yr. A portion of this erosion was likely due to adjustment of the renourished beaches in 1993-1994 and 1996. From 1997 to 2001, a period without post-nourishment adjustment, only 30 percent of the beach retreated at rates higher than -5.0 ft/yr. Schmidt estimated that renourishment would be required in 2012 if present retreat rates continued. For the Bay St Louis Downtown beach, more than 2/3 was retreating at rates greater than -5.0 ft/yr /yr, and Schmidt estimated that renourishment would be required earlier than 2012.

3.2.2.5 Future Without-Project Conditions

The future without-project conditions assumed continuation of the present maintenance activities in Hancock County; maintenance occurs on an annual basis by truck haul placement. Two cross sections or scenarios were considered as future without project conditions. The first scenario examined continued maintenance of the existing post-Katrina cross section which included a berm only feature, Figure 3.2.2-2 and Figure 3.2.2-3. Scenario 1, continued maintenance of the post-Katrina berm only feature, consists of a berm which extends approximately 150 ft from the seawall to the Mississippi Sound. The berm elevation varies from approximately 5.0 ft at the seawall to 3.5 ft at the slope break to the Mississippi Sound. The downtown Bay St Louis area beaches include a bluff with an elevation of about +12 feet. A typical cross section for the post-Katrina existing condition is shown in Figure 3.2.2-3.

The second scenario included a dune feature and was identified as an interim project to this study with funding appropriated for construction. Therefore both scenarios were considered in the evaluation of future without-project conditions. Scenario 2 consists of a 7 ft (NAVD 88) dune elevation with a 10 ft wide dune crest comprised of approximately 1.6 CY/ft of sand. The dune would be constructed approximately 50 ft seaward of the existing seawall. To provide environmental habitat and to reduce sand transport due to the strong winds, which frequently occur during storms, the dunes will be vegetated and protected with sand fencing. A typical cross section for Scenario 2 is shown in Figure 3.2.2-3.

3.2.2.5.1 Results-Future Without-Project Conditions

The coastal processes modeling analysis to evaluate the future without project berm and dune systems were conducted through application of the engineering-economic model Beach-*fx.* The purpose of the analysis was to evaluate the physical performance of the beach and dune system for anticipated future without-project project conditions. The development of the coastal processes input data for the Beach-*fx* analysis are provided in Section 2.3 of this report. The economic results of the Beach-*fx* analysis are documented in the Economic Appendices of this report.

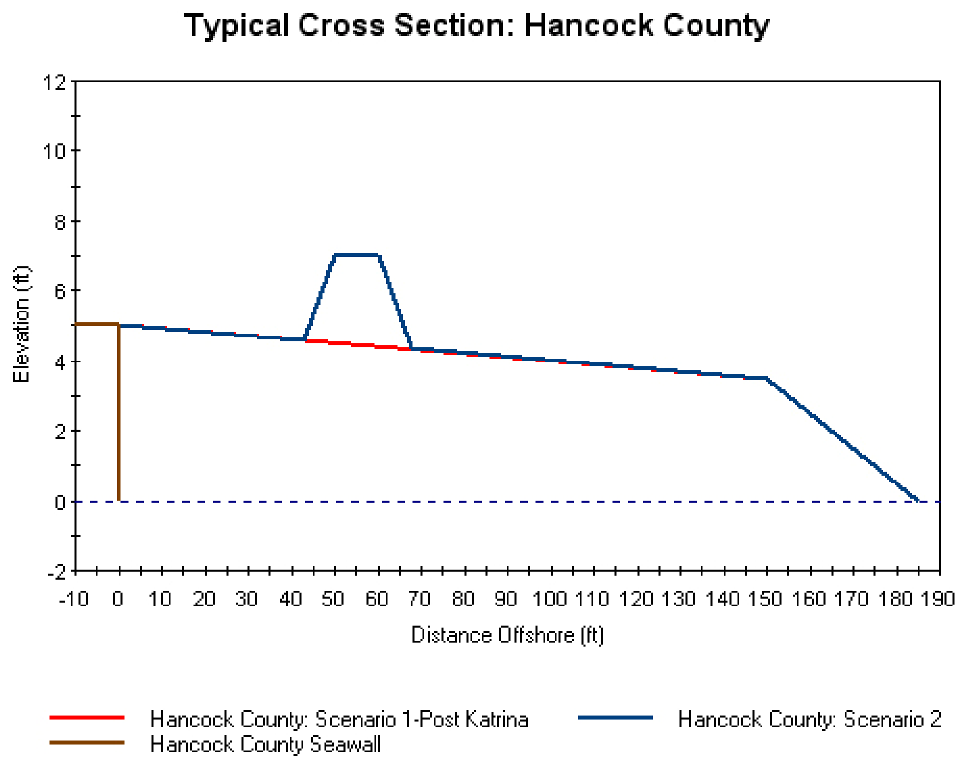

Figure 3.2.2-3. Typical Cross Sections, Hancock County Scenarios 1 and 2

Table 3.2.2-1 summarizes the results of the Hancock County without-project Beach-*fx* simulations. The data in Table 3.2.2-1 indicate that existing beach maintenance practices will require approximately 304 CY/ft of beach over a 100 year project life assuming the existing rate of sea level rise persists into the future. If the future rate of sea level rise increases, the simulations indicate that the potential moderate rate of future sea level rise will result in about a 51 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about a 69 percent increase in project volume requirements.

Table 3.2.2-1.
Hancock County Without-Project Summary

Scenario Name[1]	Number of Nourishments				Nourishment Volume (CY/ft)			
	mean	**sd**	**max**	**min**	**mean**	**sd**	**max**	**min**
Scenario 1 ESLR	100	0	100	100	297.7	28.1	379.7	250
Scenario 2 ESLR	100	0	100	100	310.3	31.6	396.9	250
Scenario 1 MSLR	100	0	100	100	443.7	47.0	581.7	302.9
Scenario 2 MSLR	100	0	100	100	473.1	53.5	607.0	285.7
Scenario 1 HSLR	100	0	100	100	497.2	53.1	654.8	351.9
Scenario 2 HSLR	100	0	100	100	531.5	32.1	619.7	452.1

[1] ESLR refers to "existing" sea level rise, MSLR refers to a "moderate" potential future sea level rise rate, and HSLR refers to a "high" potential future sea level rise rate.

As a result of the difference in maintenance cycles in Harrison and Hancock counties the project volume requirements in Hancock County exceed those in Harrison County by approximately 225 percent for without project conditions under existing sea level rise conditions. For the potential future sea level rise scenarios the increase in volume requirement is about 180 percent.

3.2.2.6 Future With-Project Options

The future with-project evaluations for Hancock County included 11 options which were evaluated for environmental restoration and enhancement of environmental habitat. Options A through D include four design cross-sections with varying dune and berm configurations. The berm and dune options would be constructed adjacent to the seawall along the length of the beach. For environmental and economic purposes, Options E through H further evaluated the four design cross-sections to include sand fencing and plantings on the dune to provide environmental habitat and to reduce sand transport due to the strong winds, which frequently occur during storms. The wider dune features would provide for a larger spatial extent with which to create environmental habitat. Options A through H were evaluated in conjunction with the Line of Defense 3 seawall.

Option A consists of a 10 ft dune elevation, 40 ft dune crest width, with a dune slope of 1:3, and a berm with an 80 ft width, an upper berm elevation of 5.5 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option B consists of an 8 ft dune elevation, 50 ft dune crest width, with a dune slope of 1:3, and a berm with an 80 ft width, an upper berm elevation of 5.5 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option C consists of a 10 ft dune elevation, 20 ft dune crest width, with a dune slope of 1:3, and a berm with a 100 ft width, an upper berm elevation of 5.5 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option D consists of an 8 ft dune elevation, 30 ft dune crest width, with a dune slope of 1:3, and a berm with a 100 ft width, an upper berm elevation of 5.5 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Dune volumes for the Hancock County design options are 10.7 CY/ft, 7.3 CY/ft, 6.6 CY/ft, and 4.7 CY/ft, for Options A, B, C, and D, respectively. Typical cross sections for Options A through D are shown in Figure 3.2.2-4. The same cross sections were used for Options E through H. For Options E through H, sea oats would be planted on the seaward dune face in an 18 by 18 inch grid pattern, with a total of three rows of plants starting at the seaward toe of the dune.

Options I and J are comparative with-project options, for future evaluation, consisting of a design cross-section which includes a dune and berm constructed as a stand alone project which does not incorporate the Line of Defense 3 seawall. Option I consists of a dune feature constructed approximately 50 ft seaward of the seawall at an elevation of 10 ft (NAVD 88), with a crest width of 55 ft, and a dune slope of 1:3. The berm width would be extended to accommodate the placement of the dune feature. Sand fencing would be placed on the dunes to reduce sand transport due to the strong winds which frequently occur during storms. The cross section for Option J is the same as Option I; however the dune would be planted to provide for additional environmental habitat. For Option J, sea oats would be planted on both the landward and seaward dune face in an 18 by 18 inch grid pattern, with a total of three rows of plants starting at the landward and seaward toes of the dune. The dunes will require initial and continued maintenance of vegetation and sand fencing. A typical cross section for Options I and J is shown in Figure 3.2.2-5.

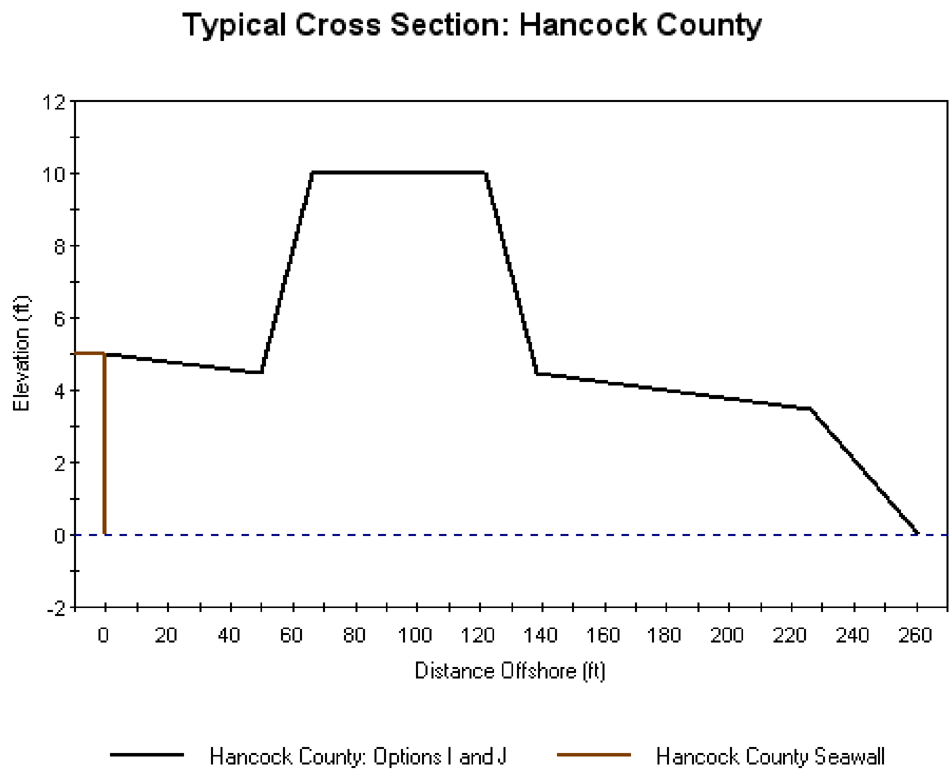

Figure 3.2.2-4. Typical Cross Sections, Hancock County Options A-D and E-H

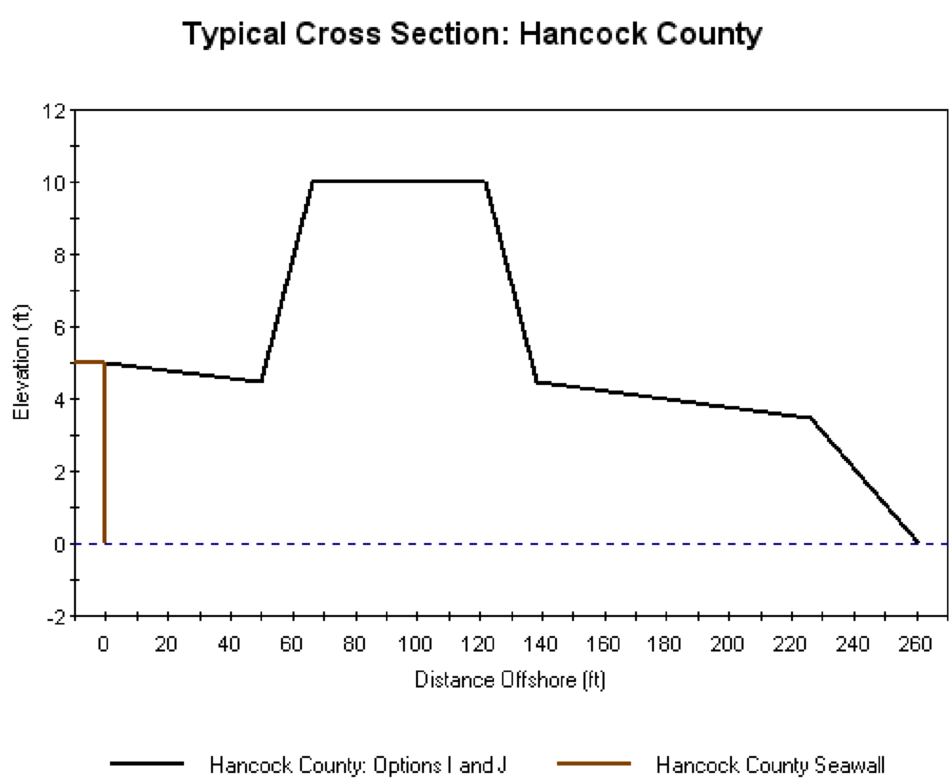

Figure 3.2.2-5. Typical Cross Section, Hancock County Comparative Dune Options I and J

Option K is also an option for future evaluation which consists of an elevated berm section constructed primarily for the creation of environmental habitat. Option K would be constructed as a stand alone option which does not incorporate the Line of Defense 3 seawall. The elevated berm section would be constructed approximately 50 ft seaward of the existing seawall to an elevation 2 ft above the existing berm with a width of approximately 60 ft. The berm width would not be extended to accommodate the placement of the elevated berm feature. The new feature would be vegetated and sand fencing would be placed to create environmental habitat and to reduce sand transport due to the strong winds which frequently occur during storms. For Option K, sea oats would be planted in a 30 by 30 inch grid pattern over the entire elevated berm area. The new feature will require initial and continued maintenance of vegetation and sand fencing. A typical cross section for Option K is shown in Figure 3.2.2-6.

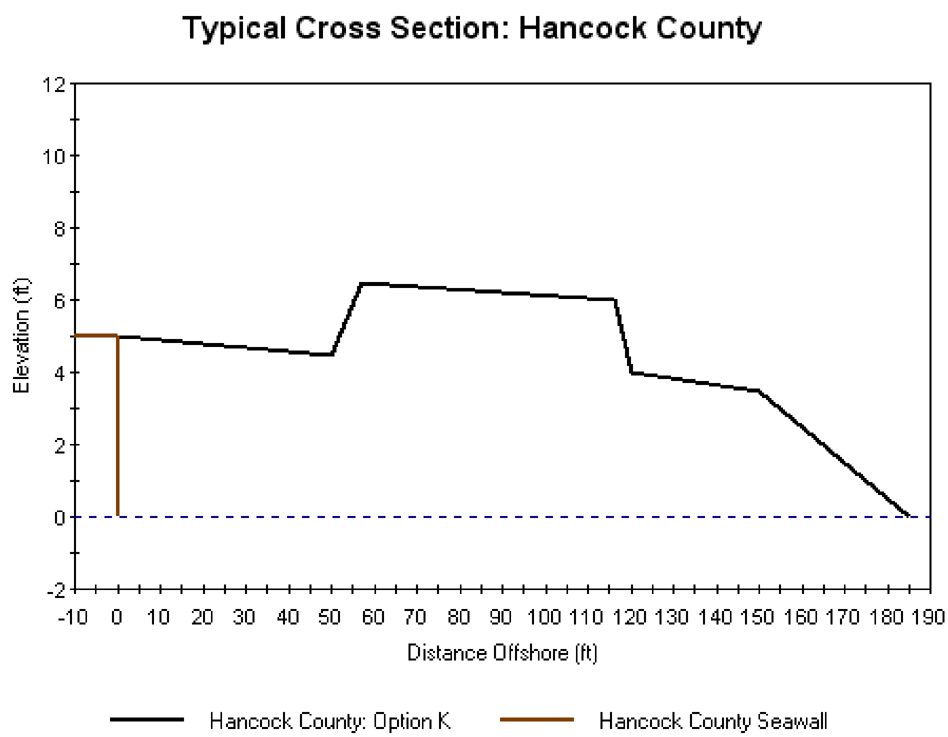

Figure 3.2.2-6. Typical Cross Section, Hancock County Option K

3.2.2.6.1 Results-Future With-Project Options

The coastal processes modeling analysis to evaluate the future with project berm and dune systems, Options A through D, were conducted through application of the engineering-economic model Beach-*fx.* The purpose of the analysis was to evaluate the physical performance of the beach and dune system for anticipated future with-project conditions and to estimate the economic costs and benefits of each. The development of the coastal processes input data for the Beach-*fx* analysis are provided in Section 2.3 of this report. The economic results of the Beach-*fx* analysis are documented in the Economic Appendices of this report. The environmental benefits of Line of Defense 2 are documented in the Environmental Appendices of this report.

Table 3.2.2-2 summarizes the Hancock County with-project Beach-*fx* simulations. The data in Table 3.2.2-2 indicate that with-project nourishment volumes for the existing rate of sea level rise are approximately 369 CY/ft of beach over a 100-year project life. If the future rate of sea level rise increases, the simulations indicate that the potential moderate rate of future sea level rise will result in about a 75 percent increase in volume requirements. A high rate of future sea level rise will result in about a 102 percent increase in project volume requirements.

Table 3.2.2-2.
Hancock County With-Project Summary

Option Name[1]	Number of Nourishments				Nourishment Volume (CY/ft)			
	mean	sd	max	min	mean	sd	max	min
Option AESLR	100	0	100	100	384.1	68.3	829.8	283.6
Option B ESLR	100	0	100	100	380.6	65.8	748.8	294.7
Option C ESLR	100	0	100	100	352.9	61.8	758.5	272.0
Option D ESLR	100	0	100	100	358.1	75.7	1,117.7	279.3
Option A MSLR	100	0	100	100	690.1	121.9	1,034.5	445.8
Option B MSLR	100	0	100	100	674.1	136.3	1,059.4	410.3
Option C MSLR	100	0	100	100	587.5	93.0	877.4	404.7
Option D MSLR	100	0	100	100	587.4	100.1	887.6	371.6
Option AHSLR	100	0	100	100	835.8	107.6	1,252.4	624.3
Option B HSLR	100	0	100	100	704.0	80.5	1,012.8	549.6
Option C HSLR	100	0	100	100	682.1	77.3	883.9	490.9
Option D HSLR	100	0	100	100	599.9	63.7	853.2	449.3

[1] ESLR refers to “existing” sea level rise, MSLR refers to a “moderate” potential future sea level rise rate, and HSLR refers to a “high” potential future sea level rise rate.

3.2.2.6.2 Summary-Future With-Project Options

The coastal processes analysis conducted as a part of this study has provided a number of useful insights with respect to morphology change, coastal evolution, and the primary drivers for storm-induced damages along the Mississippi Sound shoreline. First, the Mississippi Sound shoreline is primarily a stable, low energy coast that is dramatically impacted by tropical storm events. In the absence of tropical storm events the shoreline is expected to be only slightly erosive with shoreline change rates on the order of -1 ft/year. In general, moderate storm events produce more coastal erosion and volumetric beach change along the Mississippi Sound shoreline than do major hurricanes. This is because the large storm surge associated with the very intense storms completely inundates the beach system and protects it from the high energy dissipation associated with wave breaking, which results in less overall shoreline change and volumetric erosion of the beach. Damages to upland infrastructure are largely driven by inundation and direct wave attack as opposed to erosion, partly because most of the infrastructure is located landward of the sea wall that runs along Highway 90 in Harrison County and Beach Boulevard in Hancock County.

For with project conditions, the volume requirements in Hancock County exceed those in Harrison County by approximately 190 percent. Because the beach is restored to design conditions every year, if needed, in Hancock County the volume requirements are much larger than the volume requirements in Harrison County. In Harrison County, the beach is restored to design conditions once every 12 years. If the beach in Harrison County is damaged by a major storm in the year following reconstruction of the design template the beach remains vulnerable for the remainder of the 11 year nourishment cycle. Essentially, the present analysis indicates that the nourishment cycle in Harrison County should be shortened or augmented with a provision for emergency dune reconstruction after the occurrence of a major storm event.

3.2.2.6.3 Interior Drainage

This section is not applicable.

3.2.2.6.4 Geotechnical Data

Geology. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical. The Line 2 defense provides for the installation of dunes on the Mississippi Sound side of the existing seawalls. These dunes are intended to provide toe protection for the seawall when subjected to storm surges in the range of 3 to 5 ft. The dune slopes will be constructed to one vertical to three horizontal side slopes with a ten ft crest. The dunes for Options E through H and J thorough K will be reinforced with plantings of native sea grasses and fencing. The sand used for the dune construction would come from upland sources within 10 miles of the work area. The sands will be compatible with the existing sand with respect to grain size and color.

3.2.2.6.5 Structural, Mechanical and Electrical

This section is not applicable.

3.2.2.6.6 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.2.2.6.7 Construction Procedures and Water Control Plan

Respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, etc. and the foundation prepared for the new work. Access ramps shall be created and temporary haul routes shall be established. All temporary haul routes shall be regraded upon completion of the work.

3.2.2.6.8 Project Security

Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.2.2.6.9 Operations and Maintenance

The features that require periodic operations will be the regrading of the dune materials within the beach system and the replacement of any appreciable loss of the sea grasses and the replacement of any damaged fence sections.

3.2.2.6.10 Cost Estimate

The costs for the various options are presented in Section 3.2.2.7 Cost Estimate Summary. Total project costs for the various options are included in Table 3.2.2-3 and costs for the annualized Operation and Maintenance of the options are included in Table 3.2.2-4. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost. The project costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25 percent to cover the Cost Growth of the project.

3.2.2.6.11 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months to complete comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require approximately one year.

3.2.2.7 Cost Estimate Summary

Construction costs for the various options are included in Table 3.2.2-3 and costs for the annualized Operation and Maintenance (O&M) of the options are included in Table 3.2.2-4. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost.

3.2.2.8 References

Schmidt, K. 2002. Biennial report of sand beaches, Hancock County, 2001. Mississippi Department of Environmental Quality, Office of Geology, Open-File Report 110B, April, 53 p.

Table 3.2.2-3.
Hancock County LOD2 - Project Cost

Option	Description						Project Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	10	40	1:3	80			$8,070,000
B*	8	50	1:3	80			$6,100,000
C*	10	20	1:3	100			$4,960,000
D*	8	30	1:3	80			$4,030,000
E*	10	40	1:3	80	X	X	$8,400,000
F*	8	50	1:3	80	X	X	$6,440,000
G*	10	20	1:3	100	X	X	$5,300,000
H*	8	30	1:3	100	X	X	$4,360,000
I**	10	55	1:3	Extend to accommodate		X	$19,100,000
J**	10	55	1:3	Extend to accommodate	X	X	$19,450,000
K**				Add 2ft, 60 ft width	X	X	$4,640,000

* Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

Table 3.2.2-4.
Hancock County LOD2 – Operation and Maintenance Cost

Option	Description						O&M Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	10	40	1:3	80			$2,167,694
B*	8	50	1:3	80			$1,638,530
C*	10	20	1:3	100			$1,332,313
D*	8	30	1:3	80			$1,082,504
E*	10	40	1:3	80	X	X	$2,256,336
F*	8	50	1:3	80	X	X	$1,729,857
G*	10	20	1:3	100	X	X	$1,423,640
H*	8	30	1:3	100	X	X	$1,171,146
I**	10	55	1:3	Extend to accommodate		X	$5,130,478
J**	10	55	1:3	Extend to accommodate	X	X	$5,224,492
K**				Add 2ft, 60 ft width	X	X	N/A

* Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

3.2.3 Harrison County Beaches

3.2.3.1 General

The purpose of this section is to provide engineering information and data for the planning and design of shore protection and restoration to the shoreline along Harrison County, MS following impacts from Hurricane Katrina, 29 August 2005. Hurricane Katrina severely damaged approximately 26 miles of public beaches along the shoreline across the entire coastline of the county's shoreline.

3.2.3.2 Location

The Mississippi mainland shoreline is divided into three coastal counties: Jackson, Harrison, and Hancock Counties. Harrison County, Figure 3.2.3-1, extends approximately 27-miles, has the largest population, and the greatest number of municipalities. It is bordered on the east by industrialized Jackson County, on the west by Hancock County and the John C. Stennis Space Center and to the north by primarily rural Stone County. The County consists of five municipalities: Biloxi, D'Iberville, Gulfport, Long Beach, and Pass Christian. The Harrison County Federal Shore Protection Project, Figure 3.2.3-1, extends approximately 26-miles from Biloxi on the east to Henderson Point on the west.

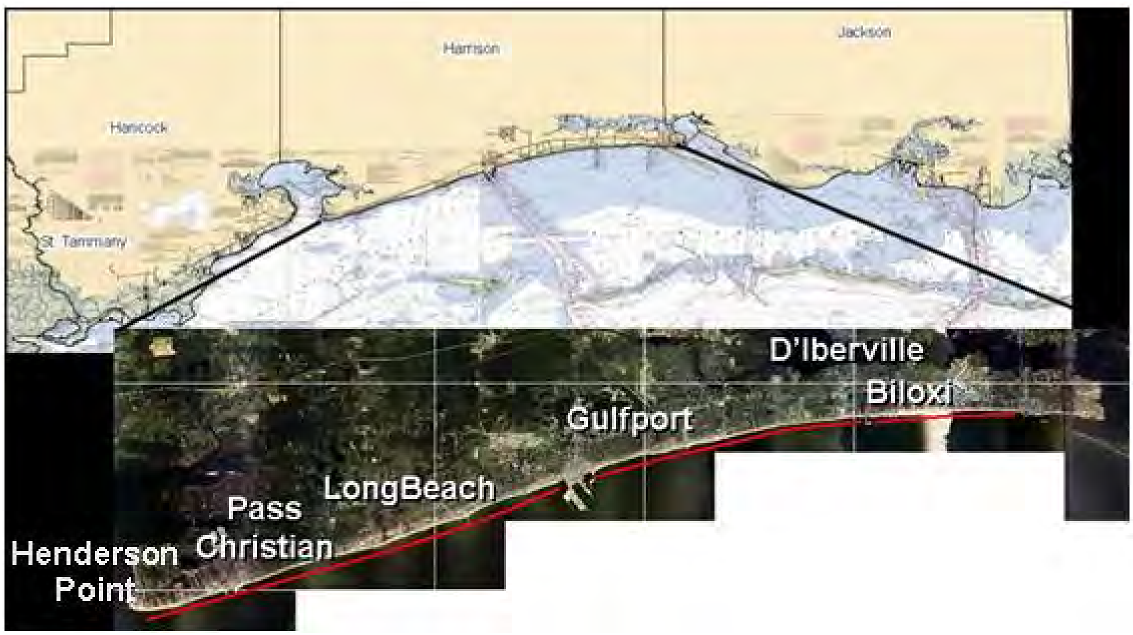

Figure 3.2.3-1. Project Location, Harrison County Beaches

3.2.3.3 Existing Conditions

As a result of the 1915 hurricane which destroyed half of U.S. 90, a seawall was constructed to protect the roadway and beach front property. After the hurricane in 1947 and due to ongoing loss of sediment, the Harrison County, Mississippi Federal Beach Erosion Control Project was constructed in 1952 under the Section 2 authority of the River and Harbor Act approved July 3, 1930. The project was constructed to protect the seawall and US 90, which provides an evacuation route for residents. Broken concrete groins were constructed to compartmentalize the beach, and a total of 6 million CY of fill was hydraulically pumped from borrow areas offshore of Gulfport Harbor.

The authorized Harrison County project provides for a beach profile consisting of a berm only feature which extends approximately 265 ft from the seawall to mean sea level (MSL). The berm elevation varies from an elevation of approximately 7.2 ft (NAVD 88) at the seawall to 3.5 ft at the slope break to the Mississippi Sound. An approximately 10 ft wide boardwalk, located adjacent to the seawall, extends along most of the Harrison County seawall. Access ramps and pavements are located along the beach, and storm water culverts pass beneath US 90 to the shoreline to drain sections of Biloxi, Long Beach, and Pass Christian. A typical cross section for the existing condition is shown in Figure 3.2.3-2.

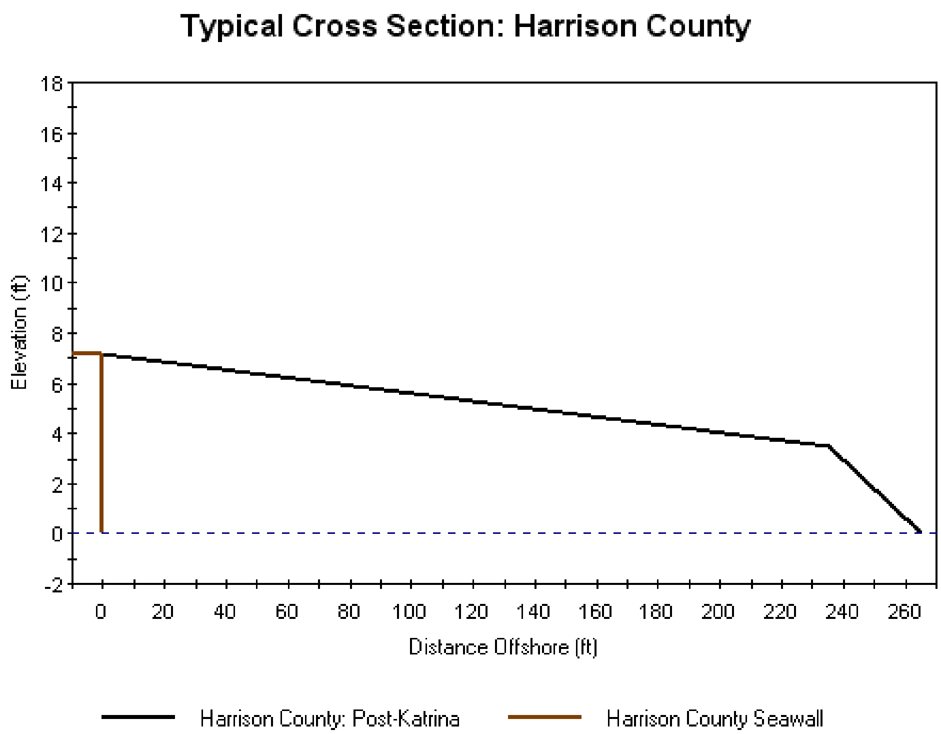

Figure 3.2.3-2. Typical Cross Section, Harrison County Existing Conditions

The Harrison County beaches were last renourished in 2001, which placed approximately 1.1 million CY of beach quality sand obtained from borrows sites located about 1,500 ft offshore of the Harrison County shoreline.

During Hurricane Katrina on 29 August 2005, the project experienced erosional damage due to wind driven waves, debris scour, storm surge and subsequent return flow after the hurricane.

3.2.3.4 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 ft, with the majority of the sound less than 30 ft deep. The offshore slope of the Sound is relatively flat with the 6 ft contour located a few hundred yards offshore to as far as 1.5 miles offshore. Bed materials are primarily fine grained 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 winds. The mean diurnal tide range is 1.6 ft, and the extreme (except during storms) is about 3.5 ft.

The velocity of normal tidal currents ranges from 0.5 to 1.0 ft 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-Hurricane Katrina high water mark measurements in the area suggest storm surges on the order of 20 to 25 ft or more.

In General, longshore sediment transport is low in magnitude and directed from east to east, although seasonal reversals can occur. Areas dominated by marsh vegetation have minimal or no longshore transport. In some sections groins or drainage structures reduce or block sediment transport.

Sand accumulated to the east of each groin indicating a weak net transport direction to the west, and a series of five profiles taken within each groin compartment indicated losses from the beach extending from Henderson Point to the Biloxi lighthouse from 1951 to 1953 were approximately 32, 500 CY/year.

Byrnes et al. (1993a, 1993b) evaluated shoreline position change rates for the mainland beach in Harrison County from 1851/52 to 1986, and found that long-term beach change has been minor (0.7 ft/yr), with change from 1951-1986 erosive at -1.6 ft/year. The greatest shoreline change has been associated with beach nourishment projects and impoundment or erosion at recently-constructed littoral barriers. The coastal highway was protected by construction of a seawall in 1928, and thus shoreline retreat has been limited by the structure. Seven geomorphic zones defined by coastal structures and harbor complexes essentially block littoral transport from the east to the west across each structure. The analysis showed impoundment of sand on the east side and erosion west of each complex or structure, indicating littoral transport from east-to-west. More than 100 smaller structures (e.g., water drainage pipes and canals that block littoral transport) and periodic beach scraping resulted in variability in shoreline position within each geomorphic zone.

3.2.3.5 Future Without-Project Conditions

The future without-project conditions assumed continuation of the present maintenance activities in Harrison County; maintenance occurs on a 12 year interval at which time the without project template is restored by hydraulic placement of fill material obtained from offshore sand sources. Two cross sections or scenarios were considered as future without project conditions. The first scenario examined continued maintenance of the existing post-Katrina cross section which included a berm only feature, Figure 3.2.3-2 and Figure 3.2.3-3. Scenario 1, continued maintenance of the post-Katrina berm only feature, consists of a berm which extends approximately 230 ft from the seawall to the Mississippi Sound. The berm elevation varies from approximately 7.2 ft (NAVD 88) at the seawall to 3.5 ft at the slope break to the Mississippi Sound. A typical cross section for the post-Katrina existing condition is shown in Figure 3.2.3-3.

The second scenario included a dune feature and was identified as an interim project to this study with funding appropriated for construction. Therefore both scenarios were considered in the evaluation of future without-project conditions. Scenario 2 consists of a 10 ft (NAVD 88) dune elevation with a 10 ft wide dune crest comprised of approximately 2.9 CY/ft of sand. The dune would be constructed approximately 50 ft seaward of the existing seawall. To provide environmental habitat and to reduce sand transport due to the strong winds, which frequently occur during storms, the dunes will be vegetated and protected with sand fencing. A typical cross section for Scenario 2 is shown in Figure 3.2.3-3.

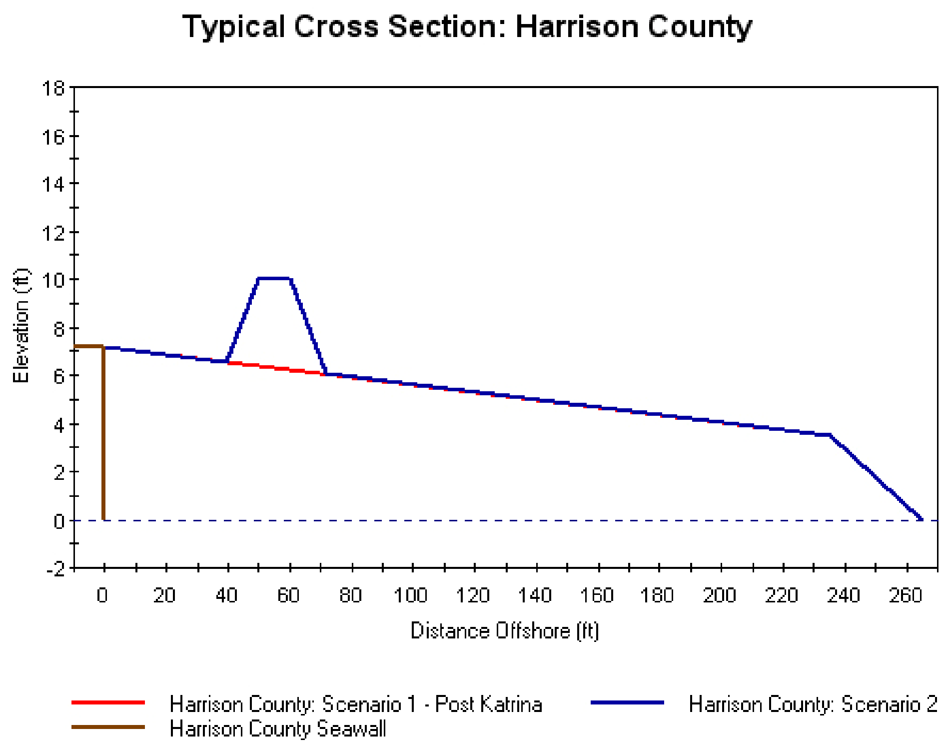

Figure 3.2.3-3. Typical Cross Sections, Harrison County Scenarios 1 and 2

3.2.3.5.1 Results-Future Without-Project Conditions

Table 3.2.3-1 summarizes the results of the Harrison County without-project Beach-*fx* simulations. The data in Table 3.2.3-1 indicate that existing beach maintenance practices will require approximately 130 CY/ft of beach over a 100 year project life assuming the existing rate of sea level rise persists into the future. If the future rate of sea level rise increases, the simulations indicate that the potential moderate rate of future sea level rise will result in about a 90 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about a 115 percent increase in project volume requirements.

Table 3.2.3-1.
Harrison County Without-Project Summary

Scenario Name[1]	Number of Nourishments				Nourishment Volume (CY/ft)			
	mean	sd	max	min	mean	sd	max	min
Scenario 1 ESLR	6	1	8	4	142.3	22.0	214.7	85.6
Scenario 2 ESLR	7	1	8	5	124.9	22.1	208.6	72.5
Scenario 1 MSLR	8	1	8	5	278.1	36.6	385.3	179.6
Scenario 2 MSLR	8	0	8	7	229.2	26.1	310.4	169.4
Scenario 1 HSLR	8	0	8	6	324.4	39.2	437.7	211.3
Scenario 1 HSLR	8	0	8	7	248.9	28.5	338.9	192.1

[1] ESLR refers to “existing” sea level rise, MSLR refers to a “moderate” potential future sea level rise rate, and HSLR refers to a “high” potential future sea level rise rate.

As a result of the difference in maintenance cycles in Harrison and Hancock counties the project volume requirements in Hancock County exceed those in Harrison County by approximately 225 percent for without project conditions under existing sea level rise conditions. For the potential future sea level rise scenarios the increase in volume requirement is about 180 percent.

3.2.3.6 Future With-Project Options

The future with-project evaluations for Harrison County included 11 options which were evaluated for environmental restoration and enhancement of environmental habitat. Options A through D included four design cross-sections with varying dune and berm configurations. The berm and dune options would be constructed adjacent to the seawall along the length of the beach. For environmental and economic purposes, Options E through H further evaluated the four design cross-sections to include sand fencing and plantings on the dune to provide environmental habitat and to reduce sand transport due to the strong winds, which frequently occur during storms. The wider dune features would provide for a larger spatial extent with which to create environmental habitat. Options A through H were evaluated in conjunction with the Line of Defense 3 seawall.

Option A consists of a 15 ft dune elevation, 35 ft dune crest width, with a dune slope of 1:3, and a berm with a 160 ft width, an upper berm elevation of 7.2 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option B consists of a 13 ft dune elevation, 45 ft dune crest width, with a dune slope of 1:3, and a berm with a 160 ft width, an upper berm elevation of 7.2 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option C consists of a 15 ft dune elevation, 25 ft dune crest width, with a dune slope of 1:3, and berm with a 170 ft width, an upper berm elevation of 7.2 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Option D consists of a 13 ft dune elevation, 15 ft dune crest width, with a dune slope of 1:3, and a berm with a 160 ft width, an upper berm elevation of 7.2 ft, and seaward berm elevation of 3.5 ft, and a foreshore slope of 1:10. Dune volumes for the Hancock County design options are 10.7 CY/ft, 7.3 CY/ft, 6.6 CY/ft, and 4.7 CY/ft, for Options A, B, C, and D, respectively. The dunes will be constructed to accommodate the approximately 10 ft wide boardwalk which extends along most of the Harrison County seawall. Typical cross sections for Options A through D are shown in Figure 3.2.3-4. The same cross sections were used for Options E through H. For Options E through H, sea oats would be planted on the seaward dune face in an 18 by 18 inch grid pattern, with a total of three rows of plants starting at the seaward toe of the dune.

Options I and J are comparative with-project options, for future evaluation, consisting of a design cross-section which includes a dune and berm constructed as a stand alone project which does not incorporate the Line of Defense 3 seawall. Option I consists of a dune feature constructed approximately 50 ft seaward of the seawall at an elevation of 15 ft (NAVD 88), with a crest width of 55 ft, and a dune slope of 1:3. The berm width would be extended to accommodate the placement of the dune feature. Sand fencing would be placed on the dunes to reduce sand transport due to the strong winds which frequently occur during storms. The cross section for Option J is the same as Option I; however the dune would be planted to provide for additional environmental habitat. For Option J, sea oats would be planted on both the landward and seaward dune face in an 18 by 18 inch grid pattern, with a total of three rows of plants starting at the landward and seaward toes of the dune. The dunes will require initial and continued maintenance of vegetation and sand fencing. A typical cross section for Options I and J is shown in Figure 3.2.3-5.

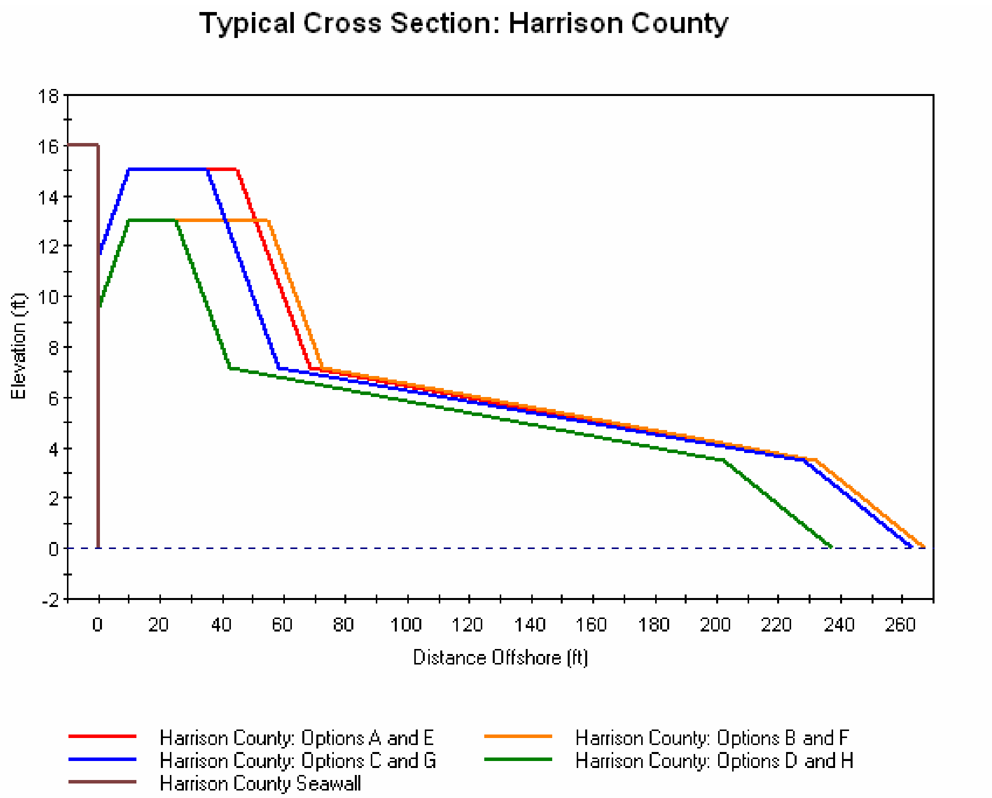

Figure 3.2.3-4. Typical Cross Sections, Harrison County Options A-D and E- H

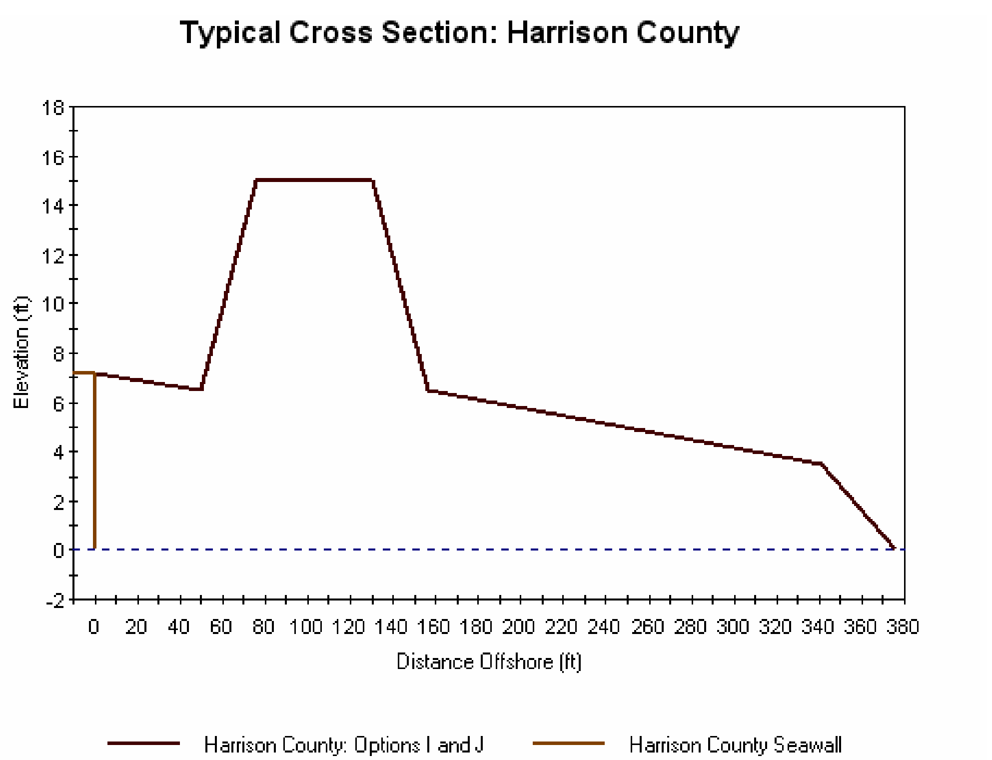

Figure 3.2.3-5. Typical Cross Section, Harrison County Comparative Dune Options I and J

Option K is also an option for future evaluation which consists of an elevated berm section constructed primarily for the creation of environmental habitat. Option K would be constructed as a stand alone option which does not incorporate the Line of Defense 3 seawall. The elevated berm section would be constructed approximately 50 ft seaward of the existing seawall to an elevation 2 ft above the existing berm with a width of approximately 60 ft. The berm width would not be extended to accommodate the placement of the elevated berm feature. The new feature would be vegetated and sand fencing would be placed to create environmental habitat and to reduce sand transport due to the strong winds which frequently occur during storms. For Option K, sea oats would be planted in a 30 by 30 inch grid pattern over the entire elevated berm area. The new feature will require initial and continued maintenance of vegetation and sand fencing. A typical cross section for Option K is shown in Figure 3.2.3-6.

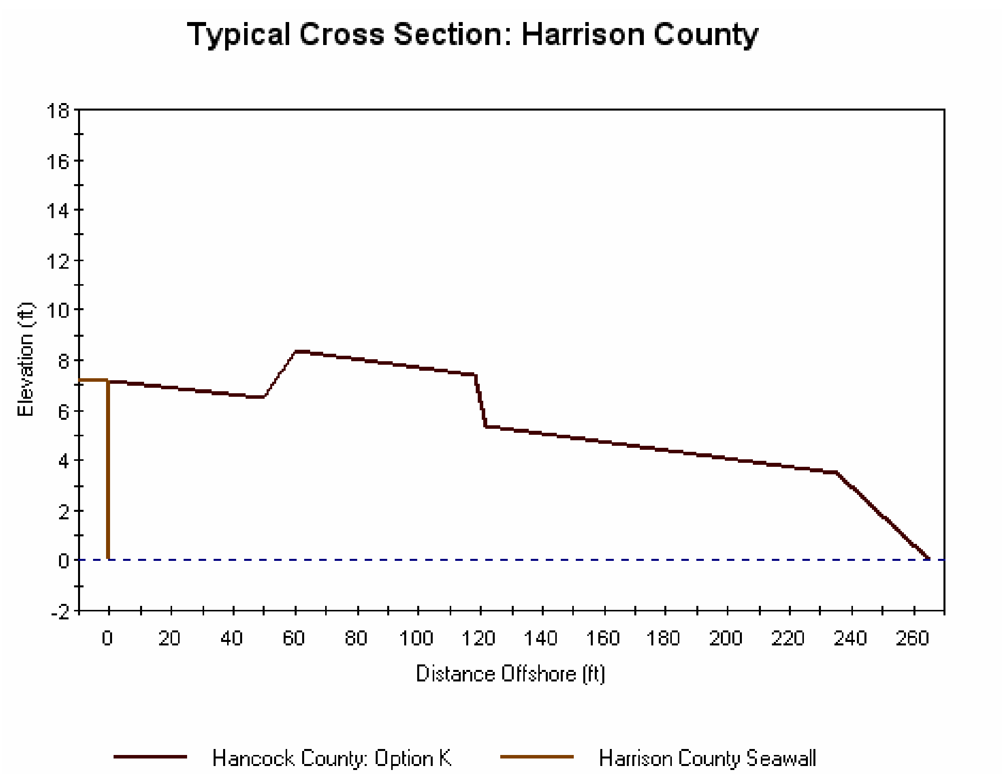

Figure 3.2.3-6. Typical Cross Section, Harrison County Option K

3.2.3.6.1 Results-Future With-Project Options

The coastal processes modeling analysis to evaluate the future with project berm and dune systems, Options A through D, were conducted through application of the engineering-economic model Beach-*fx.* The purpose of the analysis was to evaluate the physical performance of the beach and dune system for anticipated future with-project conditions and to estimate the economic costs and benefits of each. The development of the coastal processes input data for the Beach-*fx* analysis are provided in Section 2.3 of this report. The economic results of the Beach-*fx* analysis are documented in the Economic Appendices of this report. The environmental benefits of Line of Defense 2 are documented in the Environmental Appendices of this report.

Table 3.2.3-2 summarizes the Harrison County with-project Beach-*fx* simulations. The data in Table 3.2.3-2 indicate that, in general, nourishment is required at the end of every nourishment cycle (the maximum number nourishments is 9) for the moderate and high potential future sea level rise rate.

However, for the existing rate of sea level rise, on average, 2 nourishment cycles can be skipped for Option A and one nourishment cycle can be skipped for Options C and D. Nourishment volume requirements over the 100-year project life are approximately 197 CY/ft of beach assuming the existing rate of sea level rise persists into the future. If the future rate of sea level rise increases, the simulations indicate that the potential moderate rate of future sea level rise will result in about a 65 percent increase in volume requirements, whereas, a high rate of future sea level rise will result in about an 86 percent increase in project volume requirements.

Table 3.2.3-2.
Harrison County With-Project Summary

Option Name[1]	Number of Nourishments				Nourishment Volume (CY/ft)			
	mean	sd	Max	min	mean	sd	max	min
Option A ESLR	7	1	9	4	201.3	37.3	328.0	116.6
Option B ESLR	8	1	9	4	203.7	38.0	351.9	122.8
Option C ESLR	7	1	9	4	198.7	37.2	360.1	99.4
Option D ESLR	8	1	9	4	180.7	35.5	321.3	82.8
Option A MSLR	9	1	9	7	365.7	48.9	506.2	239.9
Option B MSLR	9	1	9	7	351.5	46.7	483.7	235.0
Option C MSLR	9	0	9	7	359.1	47.6	488.2	242.8
Option D MSLR	9	0	9	7	296.9	40.1	396.2	203.8
Option A HSLR	9	0	9	7	420.8	49.4	538.3	311.9
Option B HSLR	9	0	9	7	418.0	44.7	540.8	294.9
Option C HSLR	9	0	9	7	409.5	49.0	539.7	277.9
Option D HSLR	9	0	9	7	335.5	36.8	437.1	247.8

[1] ESLR refers to “existing” sea level rise, MSLR refers to a “moderate” potential future sea level rise rate, and HSLR refers to a “high” potential future sea level rise rate.

3.2.3.6.2 Summary-Future With-Project Options

The coastal processes analysis conducted as a part of this study has provided a number of useful insights with respect to morphology change, coastal evolution, and the primary drivers for storm-induced damages along the Mississippi Sound shoreline. First, the Mississippi Sound shoreline is primarily a stable, low energy coast that is dramatically impacted by tropical storm events. In the absence of tropical storm events the shoreline is expected to be only slightly erosive with shoreline change rates on the order of -1 ft/year. In general, moderate storm events produce more coastal erosion and volumetric beach change along the Mississippi Sound shoreline than do major hurricanes. This is because the large storm surge associated with the very intense storms completely inundates the beach system and protects it from the high energy dissipation associated with wave breaking, which results in less overall shoreline change and volumetric erosion of the beach. Damages to upland infrastructure are largely driven by inundation and direct wave attack as opposed to erosion, partly because most of the infrastructure is located landward of the sea wall that runs along Highway 90 in Harrison County and Beach Boulevard in Hancock County.

For with project conditions, the volume requirements in Hancock County exceed those in Harrison County by approximately 190 percent. Because the beach is restored to design conditions every year, if needed, in Hancock County the volume requirements are much larger than the volume requirements in Harrison County. In Harrison County, the beach is restored to design conditions once every 12 years. If the beach in Harrison County is damaged by a major storm in the year following reconstruction of the design template the beach remains vulnerable for the remainder of the 11 year nourishment cycle. Essentially, the present analysis indicates that the nourishment cycle in Harrison County should be shortened or augmented with a provision for emergency dune reconstruction after the occurrence of a major storm event.

3.2.3.6.3 Interior Drainage

This option will not require any interior drainage considerations.

3.2.3.6.4 Geotechnical Data

Geology. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical. The Line 2 defense provides for the installation of dunes on the Mississippi Sound side of the existing seawalls. These dunes are intended to provide toe protection for the seawall when subjected to storm surges in the range of 3 to 5 ft. The dune slopes will be constructed to one vertical to three horizontal side slopes with a ten ft crest. The dunes for Options E through H and J thorough K will be reinforced with plantings of native sea grasses and fencing. The sand used for the dune construction would come from established off shore sources within one mile of the work area. The sands will be compatible with the existing with respect to grain size and color.

3.2.3.6.5 Structural, Mechanical and Electrical

This section is not applicable.

3.2.3.6.6 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.2.3.6.7 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, etc. and the foundation prepared for the new work. Access ramps shall be created and temporary haul routes shall be established. All temporary haul routes shall be regraded upon completion of the work.

3.2.3.6.8 Project Security

Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.2.3.6.9 Operations and Maintenance

The features that require periodic operations will be the regarding of the dune materials within the beach system and the replacement of any appreciable loss of the sea grasses and the replacement of any damaged fence sections.

3.2.3.6.10 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.2.3.7 Cost Summary. Total project costs for the various options are included in Table 3.2.3-3 and costs for the annualized Operation and Maintenance of the options are included in Table 3.2.3-4. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost. The total project costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25 percent to cover the Cost Growth of the project.

3.2.3.6.11 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months to complete comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in approximately one year.

3.2.3.7 Cost Estimate Summary

Total project costs for the various options are included in Table 3.2.3-3 and costs for the annualized Operation and Maintenance (O&M) of the options are included in Table 3.2.3-4. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost.

Table 3.2.3-3.
Harrison County LOD2 - Project Cost

Option	Description						Project Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	15	35	1:3	160			$21,840,000
B*	13	45	1:3	160			$18,600,000
C*	15	25	1:3	170			$18,100,000
D*	13	15	1:3	160			$10,400,000
E*	15	35	1:3	160	X	X	$22,970,000
F*	13	45	1:3	80	X	X	$19,760,000
G*	15	25	1:3	170	X	X	$19,210,000
H*	13	15	1:3	160	X	X	$11,520,000
I**	15	55	1:3	Extend to accommodate		X	$40,290,000
J**	15	55	1:3	Extend to accommodate	X	X	$41,460,000
K**				Add 2ft, 60 ft width	X	X	$9,680,000

* Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

Table 3.2.3-4.
Harrison County LOD2 – Operation and Maintenance Cost

Option	Description						O&M Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	15	35	1:3	160			$5,866,473
B*	13	45	1:3	160			$4,996,172
C*	15	25	1:3	170			$4,861,867
D*	13	15	1:3	160			$2,793,559
E*	15	35	1:3	160	X	X	$6,170,004
F*	13	45	1:3	80	X	X	$5,307,761
G*	15	25	1:3	170	X	X	$5,160,025
H*	13	15	1:3	160	X	X	$3,094,403
I**	15	55	1:3	Extend to accommodate		X	$10,822,354
J**	15	55	1:3	Extend to accommodate	X	X	$11,136,629
K**				Add 2ft, 60 ft width	X	X	N/A

***** Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

3.2.3.8 References

Byrnes, M.R., M.W. Hiland, and R.A. McBride. 1993a. Historical shoreline position change for the mainland beach in Harrison County, Mississippi. Proceedings, Coastal Zone ’93, American Shore and Beach Preservation Association, ASCE, July 19-23, 1408-1419.

Byrnes, M.R., M.W. Hiland, and R.A. McBride. 1993b. Harrison County, Mississippi, pilot erosion rate study: phase III. Prepared for Federal Emergency Management Administration, Office of Risk Assessment, Washington, D.C., under Cooperative Agreement No. EMW-90-K-3267, 45 p.

3.2.4 Jackson County Beaches

3.2.4.1 General

The purpose of this section is to provide engineering information and data for the planning and design of shore protection and restoration to the shoreline along Jackson County, MS following impacts from Hurricane Katrina, 29 August 2005. Hurricane Katrina severely damaged approximately 7 miles of public beaches.

3.2.4.2 Location

The Mississippi mainland shoreline is divided into three coastal counties: Jackson, Harrison, and Hancock Counties. Jackson County, Figure 3.2.4-1, is the eastern-most coastal county in Mississippi and is bordered on the east by the Mississippi-Alabama state line and on the west by Harrison County. Jackson County consists of four municipalities: Pascagoula, Moss Point, Gautier, and Ocean Springs. Unlike the beaches of Harrison County, the Ocean Springs beaches are off of US 90 with less traffic and congestion. The beaches along the Ocean Springs shoreline are divided into two reaches: Front Beach extending approximately 1 mile southeastward from US 90 along Front Beach drive to the Ocean Springs Harbor, and East Beach extending approximately 1 mile from the Ocean Springs Harbor to Halstead Road, Figure 3.2.4-1.

Figure 3.2.4-1. Project Location, Jackson County Beaches

3.2.4.3 Existing Conditions

The shoreline of Ocean Springs, Mississippi has undergone many changes since seaside tourism first became popular in the area a century ago. Discontinuous Pleistocene dune bluffs, interspersed with wetland-fringed bayous, were formerly fronted by muddy tidal flats containing varying amounts of shell material. Seawalls were constructed along the shoreline fronting the developed sections of Ocean Springs in the late 1920s. Two decades later, beach nourishment projects created sand beaches in front of two seawall segments, and the modern shoreline reaches of Front Beach and East Beach became named. Front Beach, more exposed to wave and tidal forces, experienced greater levels of erosion, and renourishment with dredged material was conducted in the 1970s. At wave-sheltered East Beach, marsh vegetation colonized the beachfront intertidal zone and thus assisted in the stabilization of the shoreline. These new wetlands became modified by routine beach maintenance activity in the 1980s, and shoreline retreat appears to have become more pronounced by the early 1990s (Meyer-Arendt, 1992).

Both Front Beach and East Beach systems only consist of a berm with landward elevations ranging from approximately 2.5 to 5 ft and berm widths of about 100 ft. Access ramps and pavements are located along the beach, and storm water culverts pass beneath the roadway adjacent to the beach to the shoreline to drain sections of Jackson County.

3.2.4.4 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.

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 ft, and the extreme (except during storms) is about 3.5 ft. The magnitude of normal tidal currents ranges from 0.5 to 1.0 ft 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 ft per second (cfs). Wave heights in Mississippi Sound exceed 5 ft 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 Storms Arlene and Cindy, and Hurricanes Dennis and Katrina, all in 2005.

3.2.4.5 Future Without-Project Conditions

Evaluation of the Jackson County beaches was based on the analysis of the Hancock and Harrison County beaches, and information was extracted and transferred to this study area. Therefore, the reader is referred to Sections 3.2.2.5 and 3.2.2.5.1 for information regarding future without project conditions for Hancock County.

3.2.4.6 Future With-Project Options

Evaluation of the Jackson County beaches was based on the analysis of the Hancock County beaches, and information was extracted and transferred to this study area. The Jackson County beach options are the same design as the Hancock County beaches; therefore, the reader is referred to Section 3.2.2.6 for information regarding the Hancock County future with-project options.

3.2.4.6.1 Interior Drainage

This option will not require any interior drainage considerations.

3.2.4.6.2 Geotechnical Data

Geology. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical. The Line 2 defense provides for the installation of dunes on the Mississippi Sound side of the existing seawalls. These dunes are intended to provide toe protection for the seawall when subjected to storm surges in the range of 3 to 5 ft. The dune slopes will be constructed to one vertical to three horizontal side slopes with a ten ft crest. The dunes for Options E through H and J thorough K will be reinforced with plantings of native sea grasses and fencing. The sand used for the dune construction would come from upland sources within 10 miles of the work area. The sands will be compatible with the existing sand with respect to grain size and color.

3.2.4.6.3 Structural, Mechanical and Electrical

This section is not applicable.

3.2.4.6.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.2.4.6.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, etc. and the foundation prepared for the new work. Access ramps shall be created and temporary haul routes shall be established. All temporary haul routes shall be regraded upon completion of the work.

3.2.4.6.6 Project Security

Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

3.2.4.6.7 Operations and Maintenance

The features that require periodic operations will be the regarding of the dune materials within the beach system and the replacement of any appreciable loss of the sea grasses and the replacement of any damaged fence sections.

3.2.4.6.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.2.4.7 Cost Summary. Total project costs for the various options are included in Table 3.2.4-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.2.4-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost. The total project costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Contingency developed and assigned at 25 percent to cover the Cost Growth of the project.

3.2.4.6.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months to complete comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in approximately one year.

3.2.4.7 Cost Estimate Summary

Total project costs for the various options are included in Table 3.2.4-1 and costs for the annualized Operation and Maintenance (O&M) of the options are included in Table 3.2.4-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 2007. Estimates excludes project Escalation and HTRW Cost.

Table 3.2.4-1.
Jackson County LOD2 - Project Cost

Option	Description						Project Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	10	40	1:3	80			$1,910,000
B*	8	50	1:3	80			$1,450,000
C*	10	20	1:3	100			$1,180,000
D*	8	30	1:3	80			$960,000
E*	10	40	1:3	80	X	X	$1,990,000
F*	8	50	1:3	80	X	X	$1,530,000
G*	10	20	1:3	100	X	X	$1,260,000
H*	8	30	1:3	100	X	X	$1,040,000
I**	10	55	1:3	Extend to accommodate		X	$4,490,000
J**	10	55	1:3	Extend to accommodate	X	X	$4,570,000
K**				Add 2ft, 60 ft width	X	X	$1,110,000

* Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

Table 3.2.4-2.
Jackson County LOD2 – Operation and Maintenance Cost

Option	Description						O&M Cost
	Dune			Berm			
	Elevation (ft)	Width (ft)	Side Slope	Width (ft)	Plantings	Sand Fencing	
A*	10	40	1:3	80			$513,048
B*	8	50	1:3	80			$389,487
C*	10	20	1:3	100			$316,961
D*	8	30	1:3	80			$257,867
E*	10	40	1:3	80	X	X	$534,537
F*	8	50	1:3	80	X	X	$410,975
G*	10	20	1:3	100	X	X	$338,450
H*	8	30	1:3	100	X	X	$279,356
I**	10	55	1:3	Extend to accommodate		X	$1,206,065
J**	10	55	1:3	Extend to accommodate	X	X	$1,227,554
K**				Add 2ft, 60ft width	X	X	N/A

* Options are in conjunction with the LOD3 Seawall
** Options are without a seawall

3.2.4.8 References

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3.3 Line of Defense 3 – Elevated Roadways/Seawalls and Ring Levees

3.3.1 General

As previously mentioned, all of the beaches described as LOD-2 have a roadway landward of the beach. The roads vary from local or county roads to US Highway 90, a major, four-lane, highway that extends across the entire Harrison County coast. The existing roadways vary in elevation from four to five feet in Jackson and Hancock County and up to about 15 feet above sea level in Harrison County. All of these roads are evacuation routes and all have been damaged in past hurricanes. In a damaged or destroyed condition, these roads make re-entry to the area difficult after a hurricane has passed. Raising and using these roadways as barriers or having an associated seawall defines a portion of the 3rd line of defense, LOD-3. This line will be the first hard engineered structure that will not be affected by erosion from a storm such as a dune system.

Initial strategy was to study three elevations for the structure, elevations 12.0, 18.0 and 24.0. It was understood that due to limited heights, it would only provide protection from more frequent, smaller storms, but would be overtopped by some large storms. This coastal barrier will coincide with the beaches where they exist. Raising the beach-front road did present some engineering challenges due to the numerous intersections with other streets and roads. With several feet of elevation, the intersecting roads would require ramps that would be extremely long to have a reasonable grade. Each of these ramps would also create areas where rainfall would collect and have to be removed during a storm. It also soon became apparent that public opinion was against any structure that would block the view of the beaches and water from the roadways or adjoining properties immediately north of the roads. This was voiced in public meetings and also from agencies that were involved in the study. To maintain some level of support for this defense, it was decided to raise the roadways an average of six feet. This allowed reasonable road intersection construction and allowed the aesthetic view of the water to be maintained and would not be perceived as a high seawall along the coast. Review of the typical roadway elevations allowed raising the roadways in Jackson and Hancock County to Elevation 11.0 and Highway 90 in Harrison County to Elevation 16.0. It was decided to study these elevations without other options as the main part of LOD-3 with the understanding that these structures would not provide protection from large storms. As described above, the LOD-2 dunes could also be constructed against the elevated roadway to help protect the toe of the structural wall associated with the road.

This line of defense would be connected to Line 4, described below, at the mouth of Biloxi Bay and St. Louis Bay. It would also extend northward to higher ground or to Line 4 in Jackson County and Hancock County. The bays are an inlet for storm surge that will be controlled by surge gates that are a part of Line 4. It was also recognized that if LOD-3 was constructed without LOD-4, surge gates across the bays would have to be included as part of LOD-3.

As the first structural defense, Line 3 will exclude some areas that may be considered potential areas of retreat or have other non-structural solutions. This may be due to low population density, ecological sensitivity, areas that contain numerous waterway crossings or areas that could not function with a structural barrier in place. In Jackson County, Line 3 will encompass the southern portion of Ocean Springs, but due to extended marshes and streams, it will extend northeastward from near the eastern end of East Beach Road to higher ground. Areas east of this location contain numerous marshes, streams, and scattered development. Ring levees will be evaluated for housing developments in some areas. Further east in Jackson County are the cities of Gautier, Pascagoula and Moss Point. The presence of numerous streams and inlets will make a continuous barrier very difficult and these areas are also envisioned to have individual ring levees. While alignments were selected that provided the maximum protection for the most developed areas, some portions could be excluded due to cost and technical issues with closing off drainages. Redrawing the alignments would place some areas into a non-structural solution and could be considered as potential options for further study. These alternate alignments were drawn for Pascagoula/Moss Point, Bell Fontaine, and Gulf Park Estates.

At the western end of LOD-3, the barrier will extend down North Beach Boulevard for several miles to near Bayou Caddy and then turn north to tie in with higher ground. By following this path, the existing roadway will provide an alignment and it will encompass much of the developed waterfront from Bay St. Louis to Waveland, MS. Further west, the town of Pearlington will be evaluated for construction of a ring levee.

As with the main portion of LOD-3, the ring levees were initially considered with the same three elevations of 12.0, 18.0 and 24.0. Closer study revealed that in many cases, the elevation 12.0 was too low based on existing ground surfaces and the elevation 24.0 may not be high enough to be certified by FEMA for a 100-year storm event. The elevations to be studied for the ring levees then was changed to 20.0 and 30.0 with the assumption that the 100-year event would fall between these elevations and that the elevation 30.0 design would be sufficiently high for even a 500-year event. A 100-year minimum event is necessary for levee certification by FEMA. Having a conceptual design with cost estimates for these two elevations would allow for a cost curve to help predict the costs for certain storm events once the modeling studies were complete and stage frequency curves developed.

Modeling for storms that could hit the Mississippi Coast will define the predicted return frequency for LOD-3 structures based on the location and type of structure. While many options were reviewed for the type of structure to be used along the roadways, a simple elevated roadway associated with an extension of the existing seawall was chosen for reliability reasons. A structure that did not mainly rely on powered systems or with multiple moving systems was deemed more suitable for the purposes of this line of defense. As previously described, numerous conceptual designs were considered including inflatable barriers, concrete sidewalks or roadways that could be hydraulically rotated upwards to form a seawall, sliding panel gates within a seawall, and structural concrete seawalls. The ring levees were all designed as earthen structures. It should be understood that all of these LOD-3 structures would provide less protection than would be required for a Camille or Katrina-like storm. LOD-3 storm damage reduction levels are limited and will be determined based on public and local government acceptance and the amount of risk that Mississippi is willing to accept.

As previously mentioned, this line is dependent on having the ability of closure across the two bays to prevent the storm surge from running inside the mouths of the bays. While the plan calls for surge gates to be associated with Line 4, surge gates would also have to be incorporated with Line 3 if Line 4 was not selected as an alternative. The top elevation of surge gates used solely for Line 3 would be of an elevation that would be compatible with the rest of that barrier. To develop a cost curve for the barriers, cost estimates for elevations of 20.0, 30.0 and 40.0 have been completed and will be used in conjunction with both LOD-3 and LOD-4. More detailed discussion of the surge gates is found below under the LOD-4 section.

Interior drainage behind these barriers must be considered. Any large rainfall event would require that the water trapped behind the barrier have a means to drain or even be mechanically pumped.

The amount of storage that a given watershed could provide behind a barrier during surge conditions will vary. The means to block surge but allow drainage as the surge passes may include conduits with flap valves or gated culverts up to surge gates across large bodies of water. The areas where pumping is required are numerous, but necessary to prevent residual damages associated with this blockage of normal drainage.

The pumping stations, where required, must survive any storm damage and continue to operate until the storm event has passed. This will require hardened structures to house the pumps and power systems and be constructed to a height that corresponds to the risk associated with that line of defense.

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that the majority of roadways and all railways crossing the levee alignment would be retained.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required. See Figures 3.3.1-1 and 3.3.1-2 for geometric plan representations of typical types of roadway crossing structures. All gates up to and including 9 feet high would be roller gates. All above 9 feet high would be dual leaf swing gates.

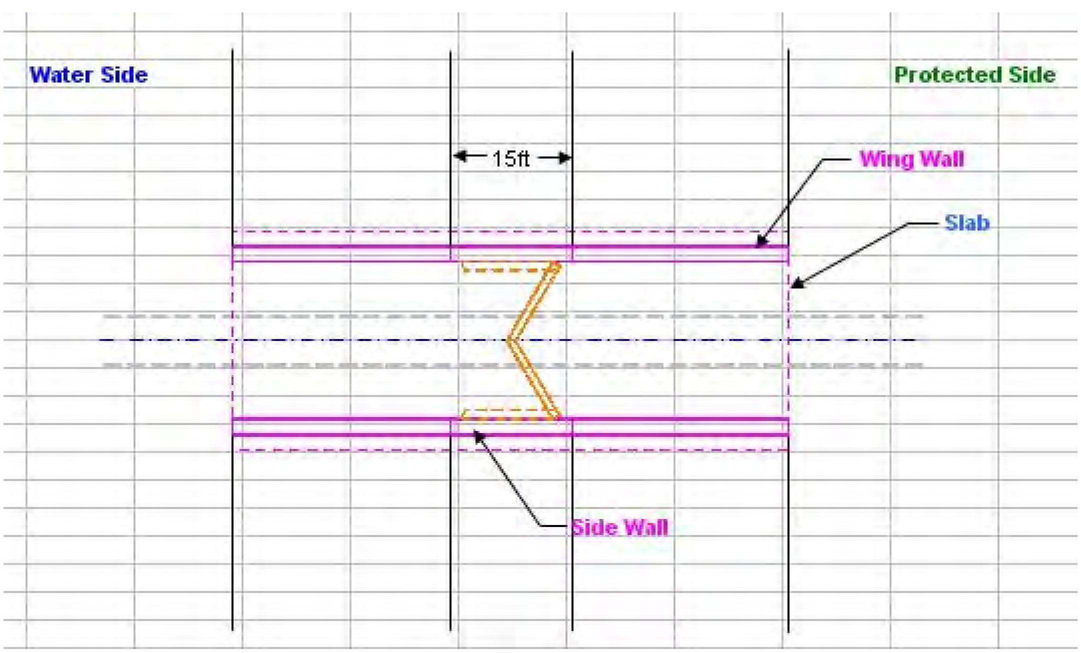


Figure 3.3.1-1. Crossings Under 9ft (two lane gate shown; gate and structure would be mirrored to provide for four-lane highway)

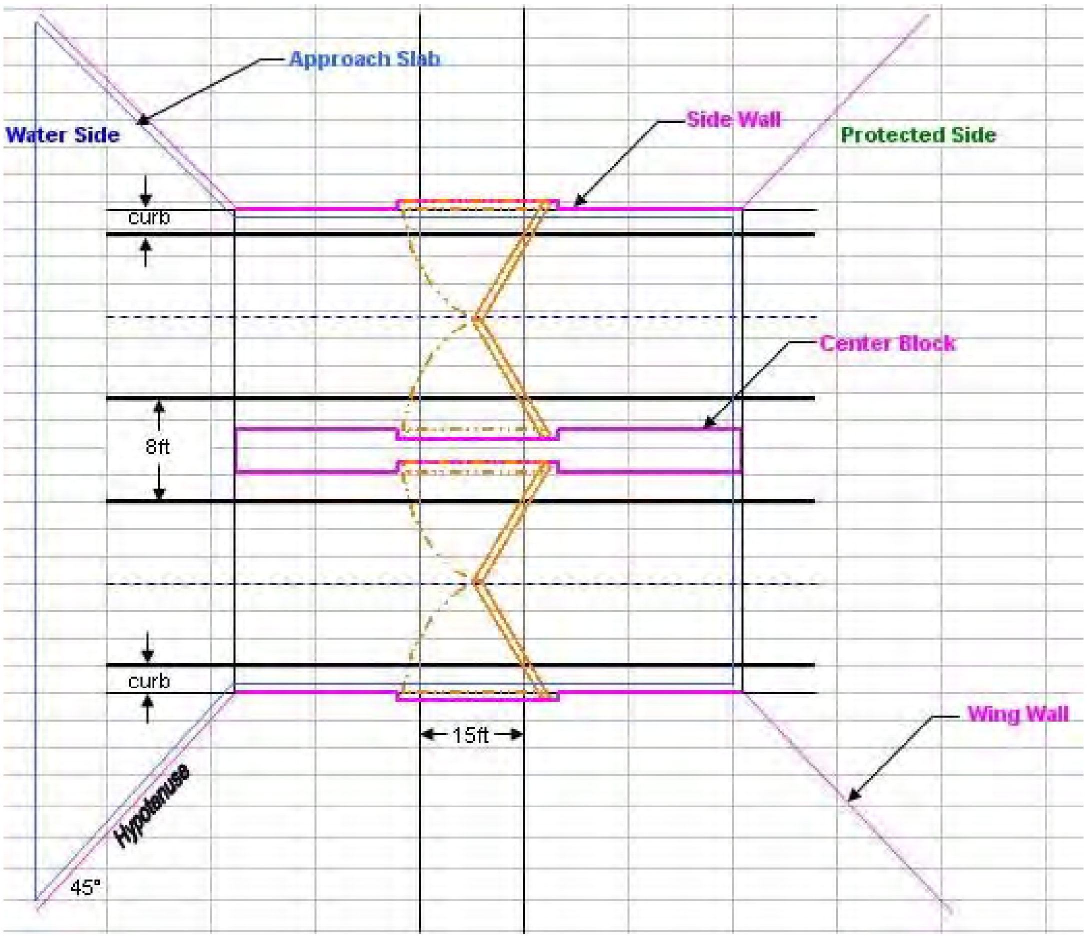


Figure 3.3.1-2. Crossing Over 9ft

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures or much more expensive tunnel structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee. See Figure 3.3.1-3 for geometric plan representation of railroad crossing structures. All railroad gates were assumed to be dual leaf swing gates extending to the full height of levee.

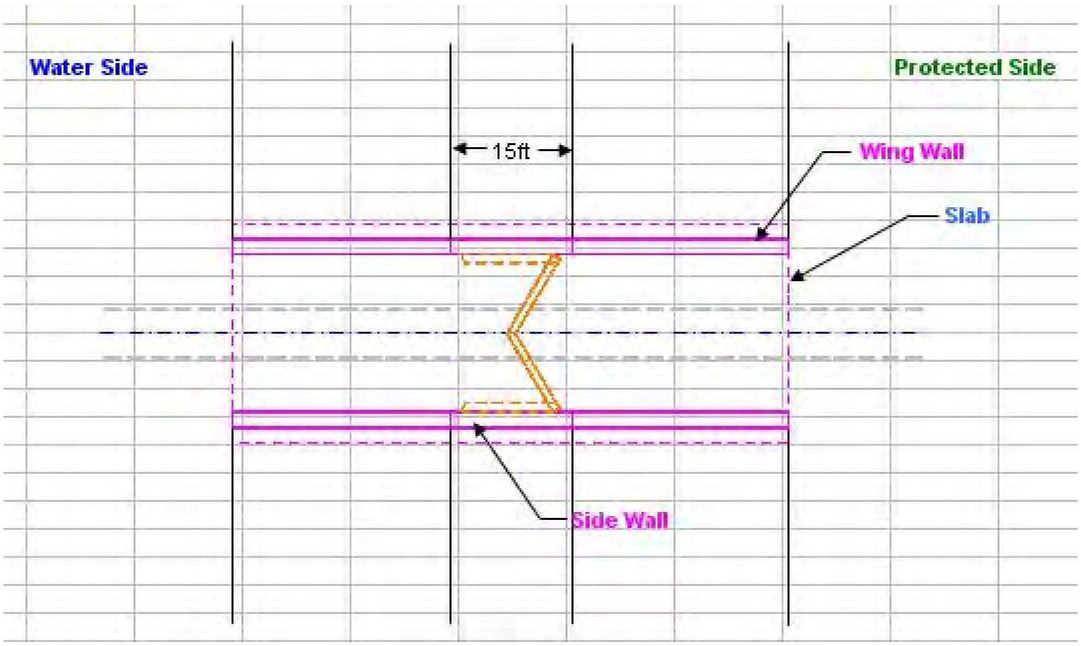


Figure 3.3.1-3. Railroad Crossings

3.3.2 Hancock County Ring Levees, Pearlington

3.3.2.1 General

Pearlington was an extremely hard hit area during the 2005 hurricane season. Water reached a depth of 10-14 ft over the whole community. An earthen ring levee was evaluated for protection of this area. The levee was evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this protection option was done by comparing benefits computed by Hydrologic Engineering Center's (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.2.2 Locations

The location of the ring levee at Pearlington is shown below in Figure 3.3.2-1 and in Figure 3.3.2-2.

Figure 3.3.2-1. Vicinity Map, Pearlington

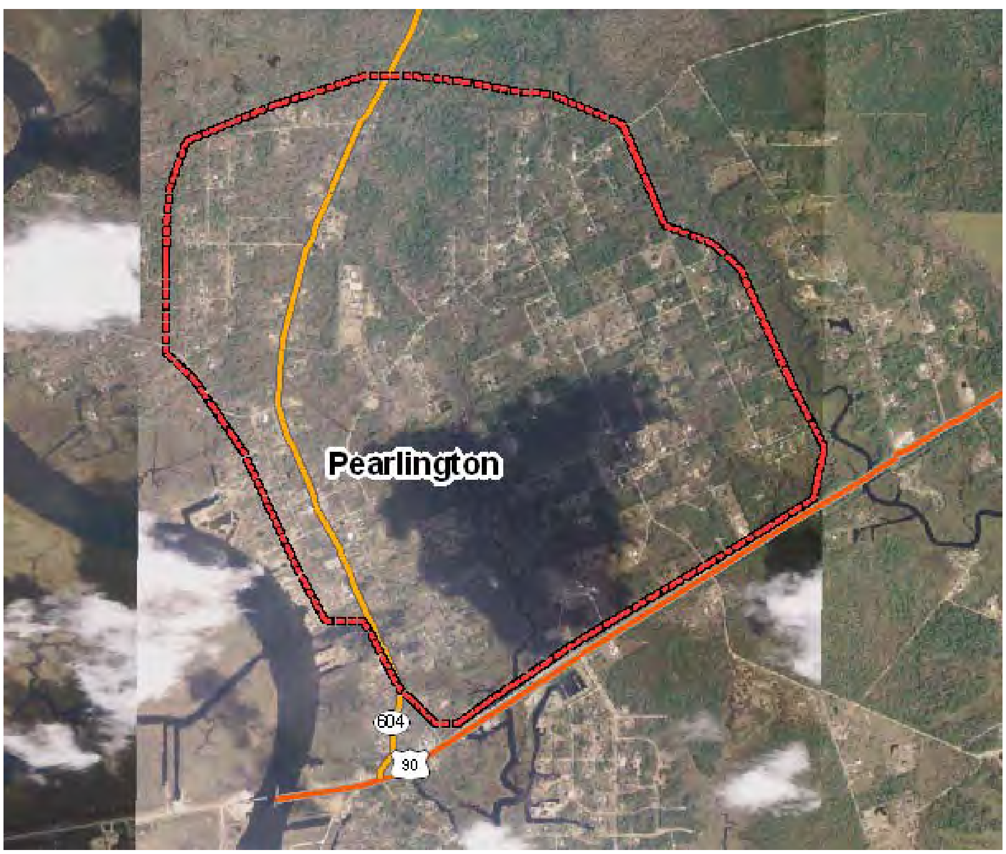


Figure 3.3.2-2. Pearlington Ring Levee

3.3.2.3 Existing Conditions

The town of Pearlington lies on the bank of the Pearl River about 5 miles from the Mississippi Sound. Ground elevations over most of the residential and business areas are very low between elevation 6-10 ft NAVD88. The city limits as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown below in Figure 3.3.2-3.

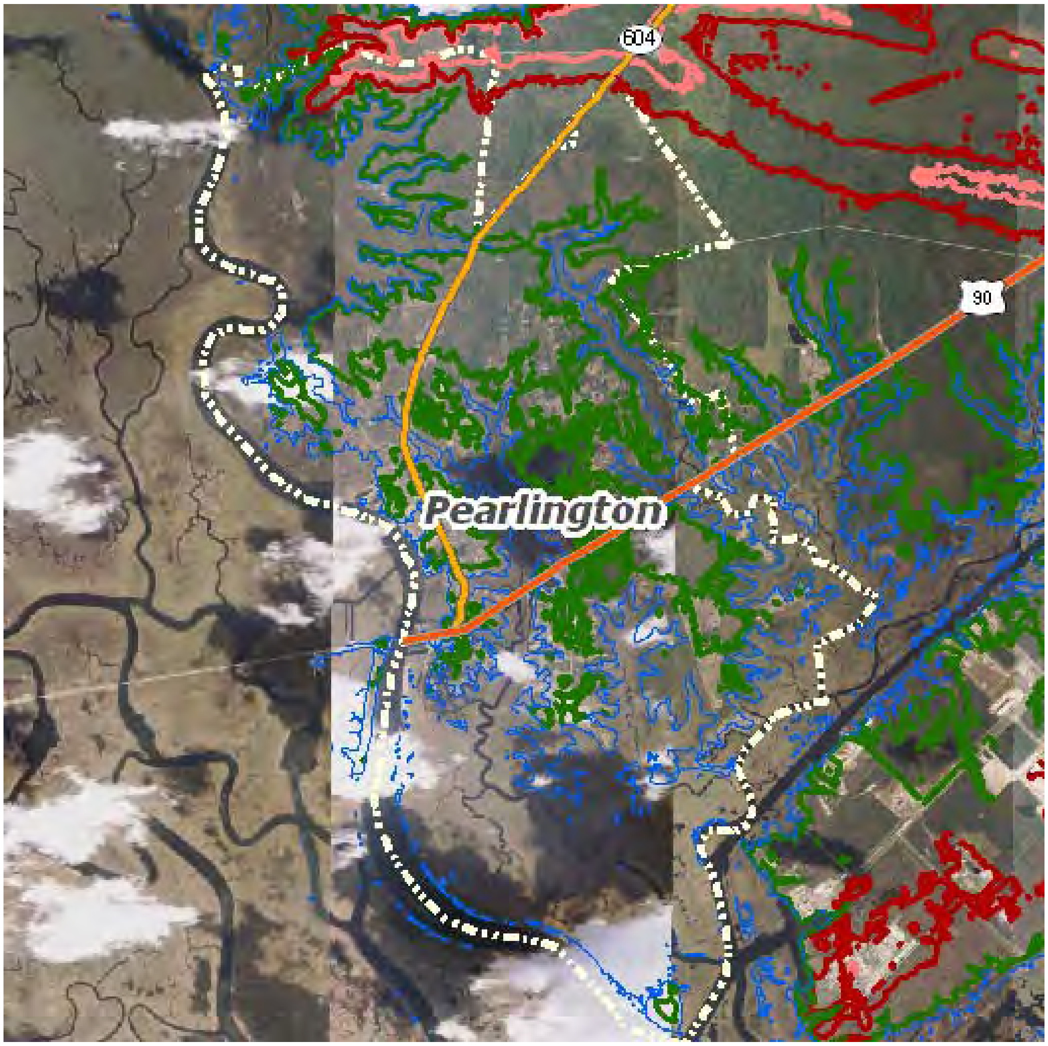


Figure 3.3.2-3. Pearlington Ground Contours and City Limits

Drainage is mostly through natural drainage ways to the Pearl River.

Impacts from hurricanes can be devastating. Damage from Hurricane Katrina in August, 2005 in the Pearlington area are shown below in Figures 3.3.2-4 and 3.3.2-5.

Source: http://ngs.woc.noaa.gov/storms/katrina/24615651.jpg

Figure 3.3.2-4. Hurricane Katrina Damage, Pearlington, MS

Source: wndyfront, http://www.flickr.com/photos/wndyfrost/230684420/

Figure 3.3.2-5. Hurricane Katrina Damage, Pearlington, MS

3.3.2.4 *Coastal and Hydraulic Data*

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.2-6. The data indicates the water was as high as 18-20 ft NAVD88 near the site, totally inundating the entire area.

Figure 3.3.2-6. Ground Contours and Katrina High Water, Pearlington

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Pearlington at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.2-7.

Existing Condition Stage –Frequency data for Save Point 62, at Highway 90 in Pearlington, is shown below in Figure 3.3.2-8. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

It should be noted that the frequency curve shown above reflects only that flooding resulting from storm surge in the gulf. Riverine flooding is not incorporated into this curve.

Figure 3.3.2-7. Hydrodynamic Modeling Save Point near Pearlington

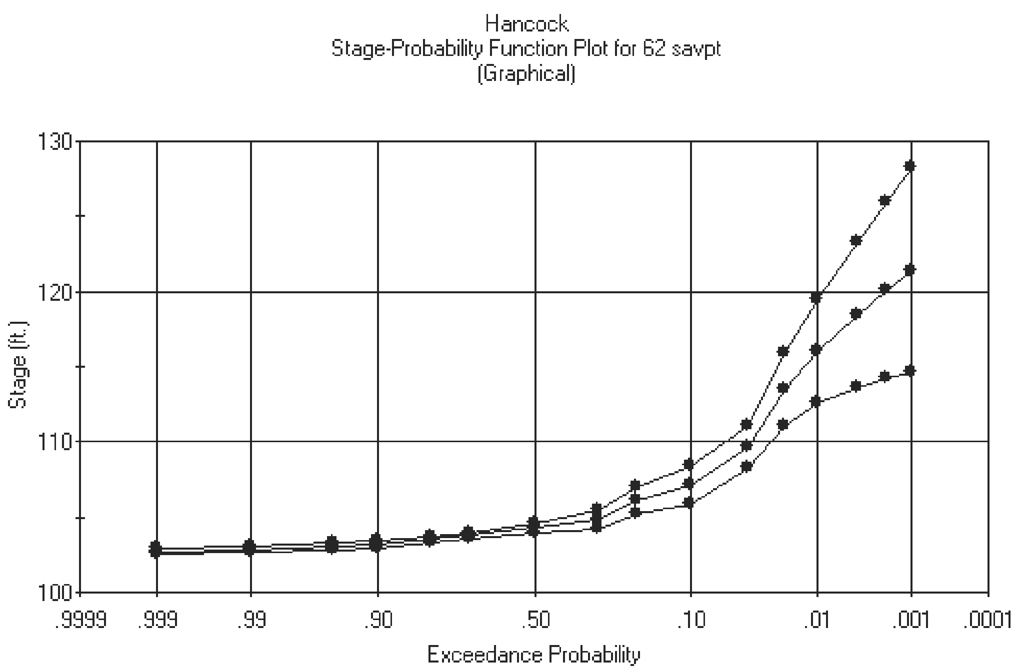


Figure 3.3.2-8. Existing Conditions at Save Point 62, near Pearlington, MS

3.3.2.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 1217 acres around the most densely populated areas of Pearlington as shown on the following Figure 3.3.2-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal.

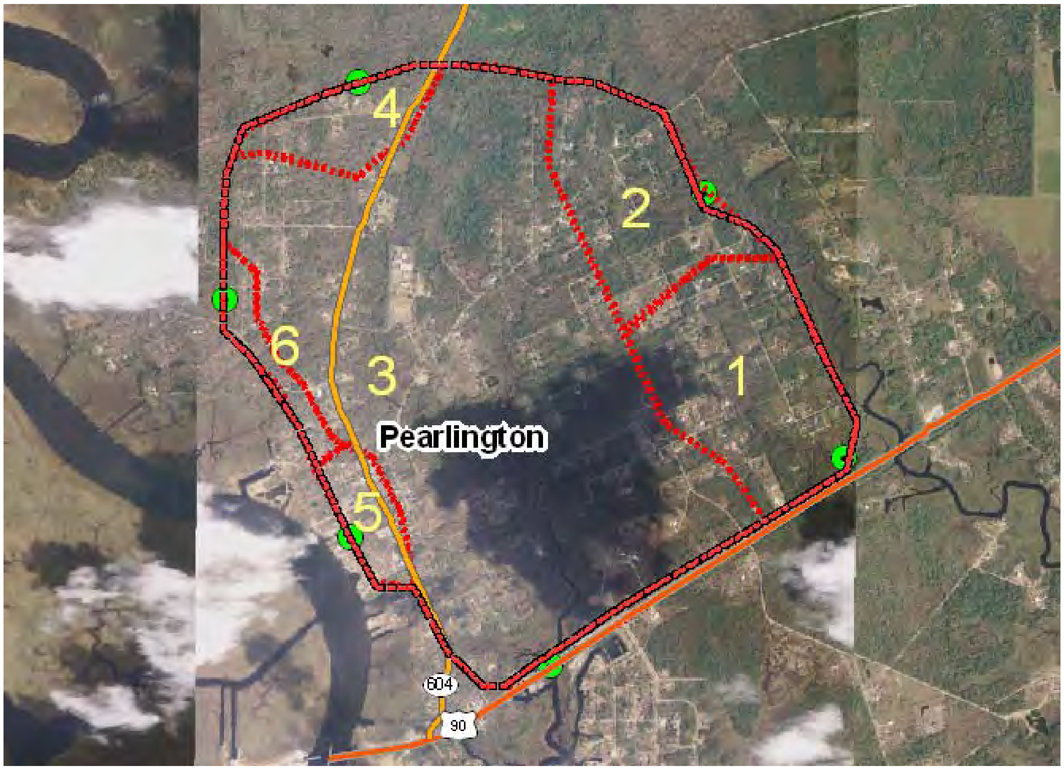


Figure 3.3.2-9. Pump/Culvert/Sub-basin Site Locations

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown below in Figure 3.3.2-10.

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.2-11 was caused by approximately 1-2 ft of overtopping crest depth.

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.2-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.2-10. North Sea, Germany, March 1976

Source: ERDC, Steven Hughes

Figure 3.3.2-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

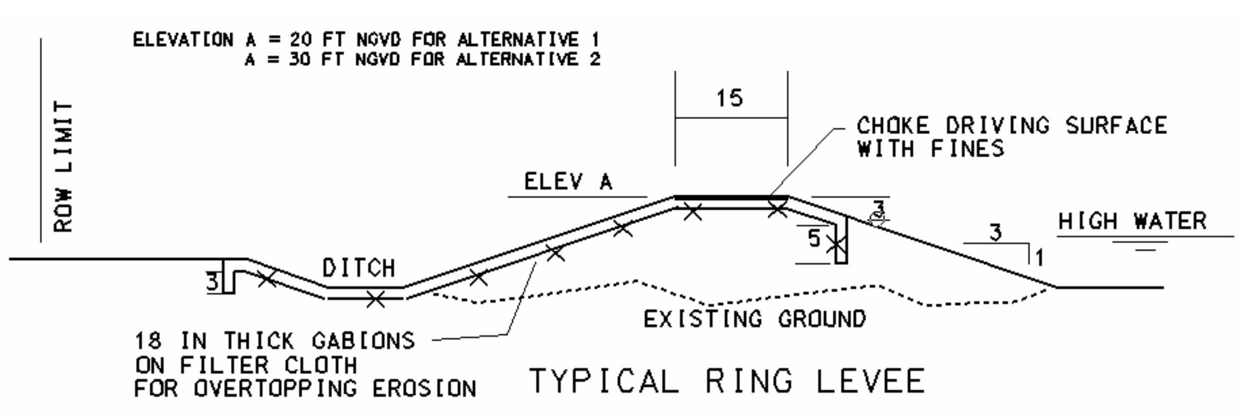


Figure 3.3.2-12. Typical Section at Ring Levee

3.3.2.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.2-13.

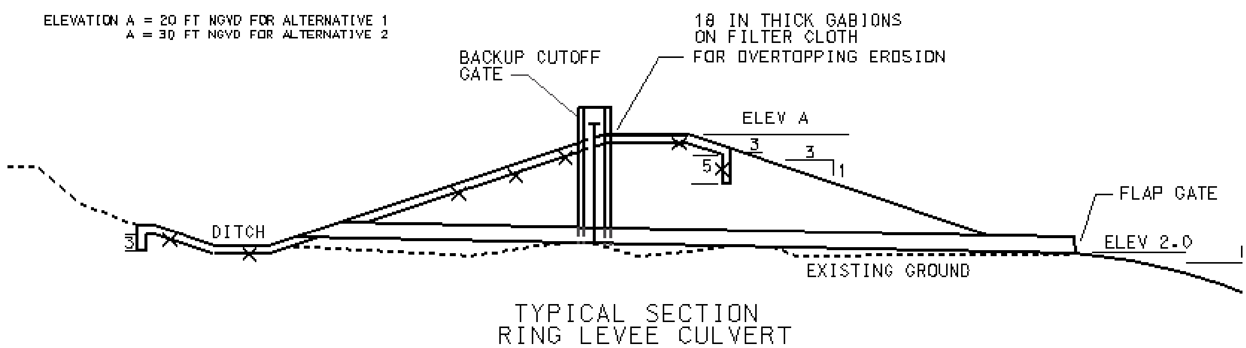


Figure 3.3.2-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown below in Figure 3.3.2-14 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types, hydrologic soil groups, and major sub-basins are shown below in Figure 3.3.2-14.

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

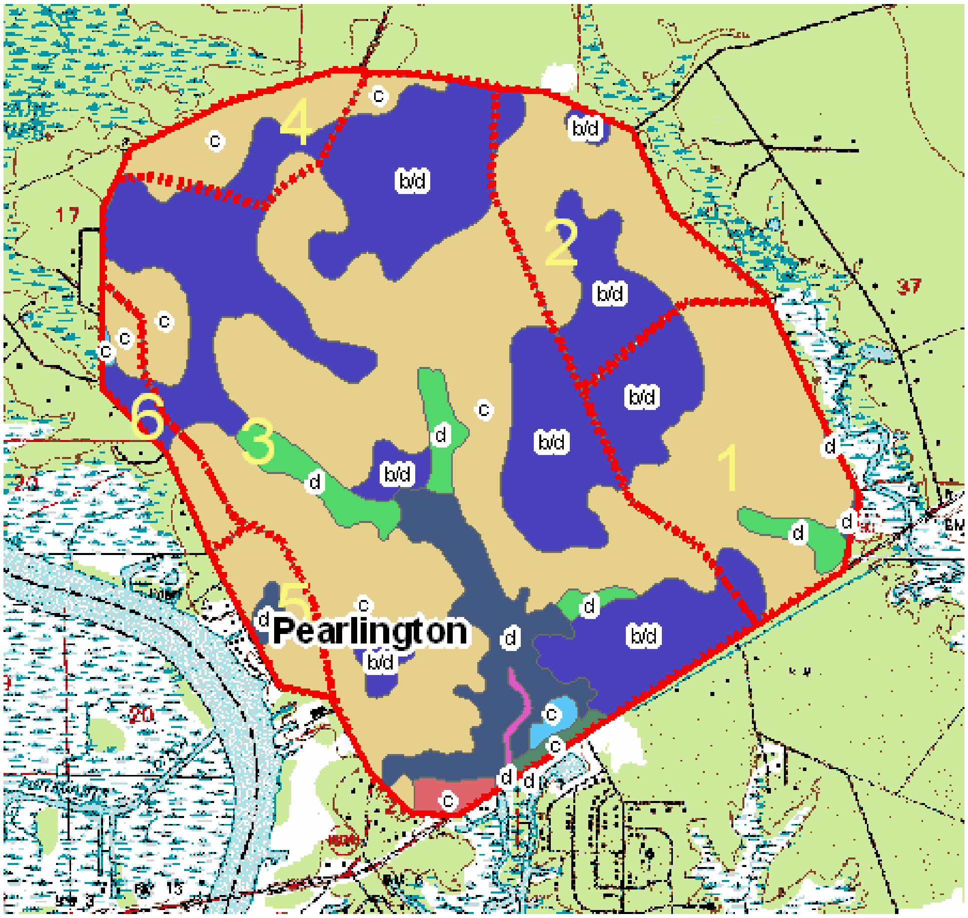


Figure 3.3.2-14. Pearlington Hydrologic Soil Groups

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.2.5.2 Geotechnical Data

Geology: Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.2.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts and pumping facilities. The sites are shown above.

3.3.2.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.2.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.2.5.3.3 Pumping Stations Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer's stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.2.5.3.4 Pumping Stations Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.2.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 6 pumping facilities included in the Pearlington Ring Levee system for the elevation 20 protection level were constant at approximately 15 feet and the corresponding flows required varied from 47,127 to 594,701 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 290 horsepower pumps, to one having eight 60-inch diameter pumps each running at 560 horsepower.

3.3.2.5.3.6 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.2.5.3.7 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.2.5.3.8 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Pearlington area to elevation 20, 18 roadway intersections would have to be accommodated. For this study it was estimated that all 18 would require swing gate structures.

3.3.2.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report

therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.2.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.2.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.2.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Maintenance costs are included in this report.

3.3.2.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.2.7, Cost Summary. Construction costs for the various options are included in Table 3.3.2-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.2-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.2.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.2.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Pearlington. The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.2.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.2.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.2.6.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

3.3.2.6.3.1 Pumping Facilities. Flow and Pump Sizes. Option B.

Design hydraulic heads derived for the 6 pumping facilities included in the Pearlington Ring Levee system for the elevation 30 protection level were constant at approximately 25 feet and the corresponding flows required varied from 47,127 to 594,701 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 475 horsepower pumps, to one having eight 60-inch diameter pumps each running at 1000 horsepower.

3.3.2.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.2.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.2.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.2.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.2.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.2.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.2.7 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.2-1 and 3.3.2-2 below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.2-1.
Pearlington Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$104,800,000
Option B – Elevation 30 ft NAVD88	$120,200,000

Table 3.3.2-2.
Pearlington Ring Levee O & M Cost Summary

Option	O&M Cost
Option A – Elevation 20 ft NAVD88	$1,320,000
Option B – Elevation 30 ft NAVD88	$1,526,000

3.3.2.8 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.3 Hancock County, Bay St. Louis Ring Levee

3.3.3.1 General

Bay St. Louis was an extremely hard hit area during the 2005 hurricane season. Water reached a depth of 10-20 ft over the coastal community. An earthen ring levee was evaluated for protection of this area. The levee was evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this protection option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented elsewhere in this report.

3.3.3.2 Location

The location of the ring levee at Bay St. Louis is shown below in Figures 3.3.3-1 and in Figure 3.3.3-2.

Figure 3.3.3-1. Vicinity Map, Bay St. Louis

Figure 3.3.3-2. Bay St. Louis Ring Levee

3.3.3.3 Existing Conditions

Drainage at Bay St. Louis and Waveland is to the Mississippi Sound to the south and to tributaries of St. Louis Bay to the north. The Shoreline Park subdivision area to the north of Bay St. Louis is very low at elevations of 4-6 ft NAVD88 and subject to frequent flooding from storm surge. The 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), 20-ft(peach), and 24-ft(dark pink) ground contour lines are shown below in Figure 3.3.3-3.

Impacts from hurricanes can be devastating. Damage from Hurricane Katrina in August, 2005 in the Bay St. Louis area are shown below in Figure 3.3.3-4 and 3.3.3-5.

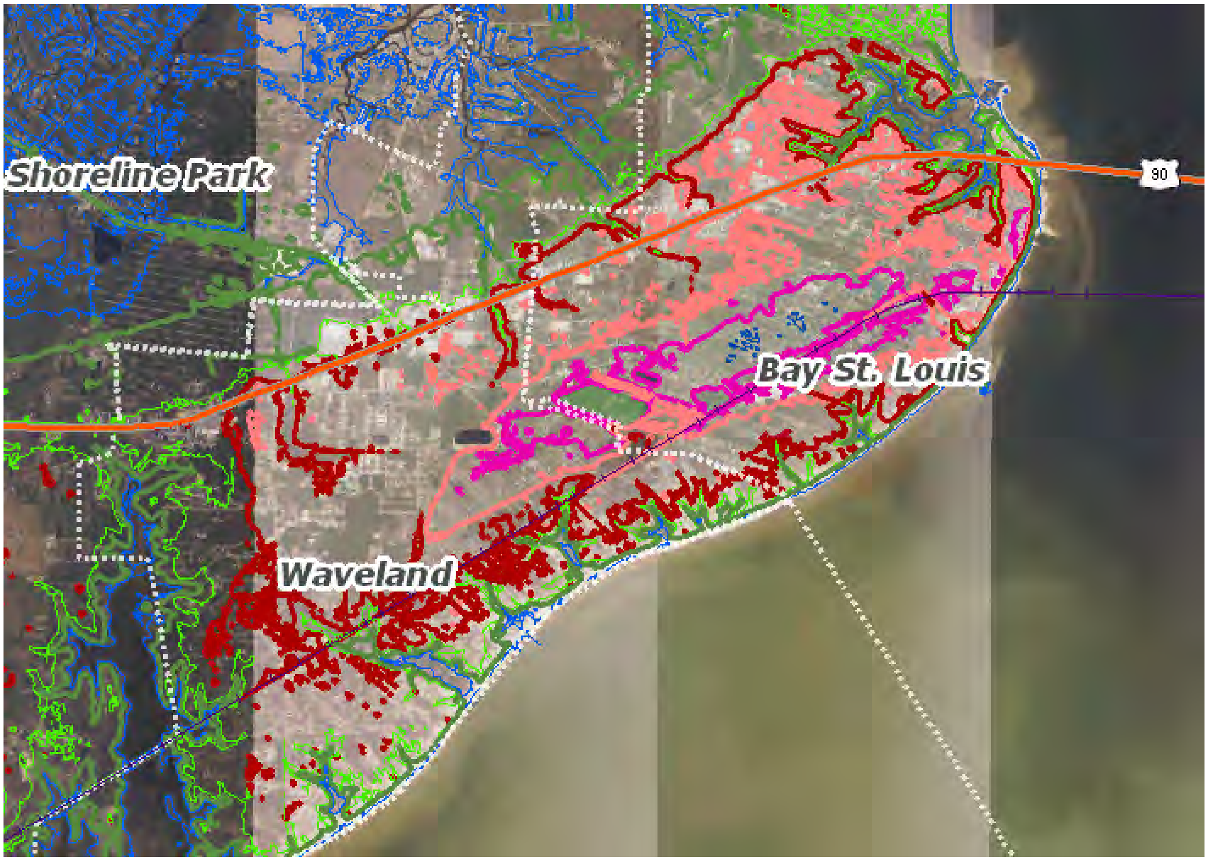


Figure 3.3.3-3. Bay St. Louis Ground Contours and City Limits

Source: http://ngs.woc.noaa.gov/storms/katrina/24614515.jpg

Figure 3.3.3-4. Hurricane Katrina Damage, Bay St. Louis, MS

Source: http://www.pbase.com/dbphotos/image/48766824

Figure 3.3.3-5. Hurricane Katrina Damage, Bay St. Louis, MS

3.3.3.4 Coastal and Hydraulic Data

Historic coastal data are shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), 20-ft(peach), and 24-ft(dark pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.3-6. The data indicates the water was as high as 22-28 ft NAVD88 near the site, totally inundating most of the area.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown elsewhere in this report. Points near Bay St. Louis at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.3-7.

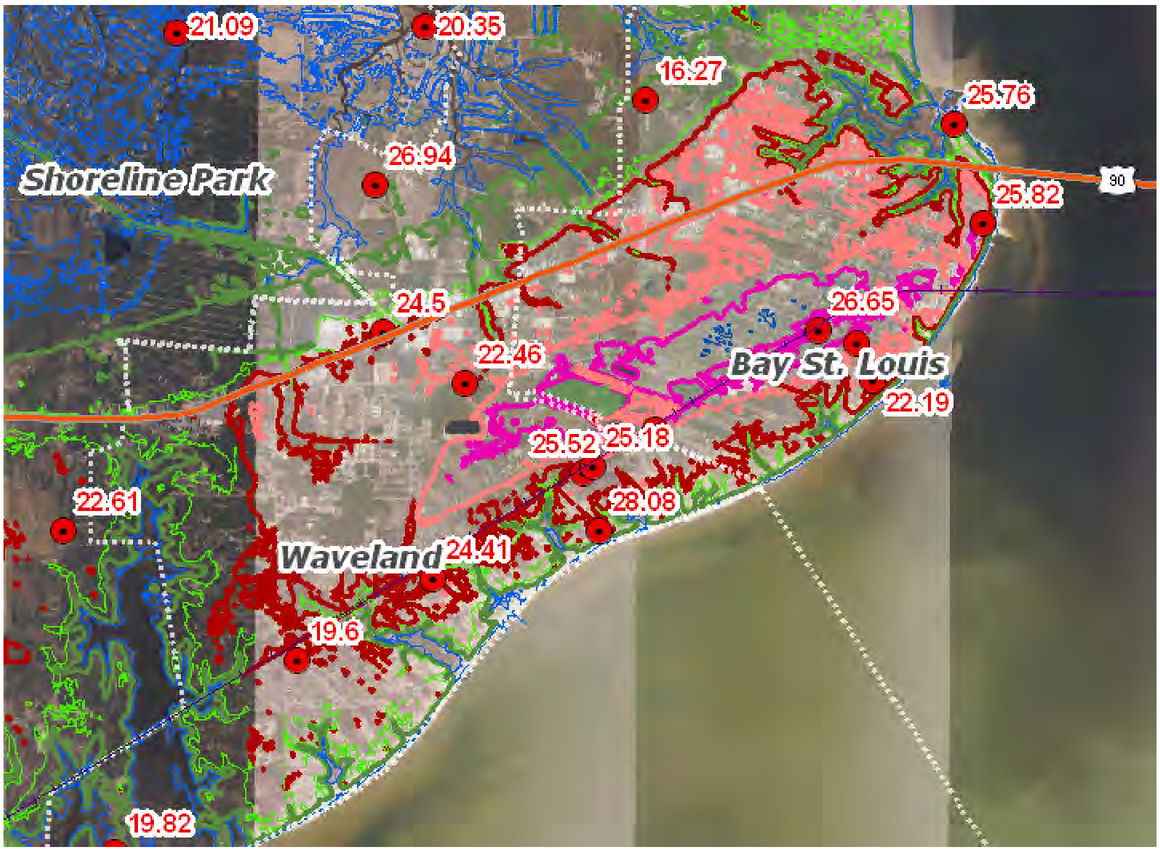


Figure 3.3.3-6. Ground Contours and Katrina High Water, Bay St. Louis

Figure 3.3.3-7. Hydrodynamic Modeling Save Point near Bay St. Louis

Existing Condition Stage –Frequency data for Save Point 62, at Highway 90 in Bay St. Louis, is shown below in Figure 3.3.3-8. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

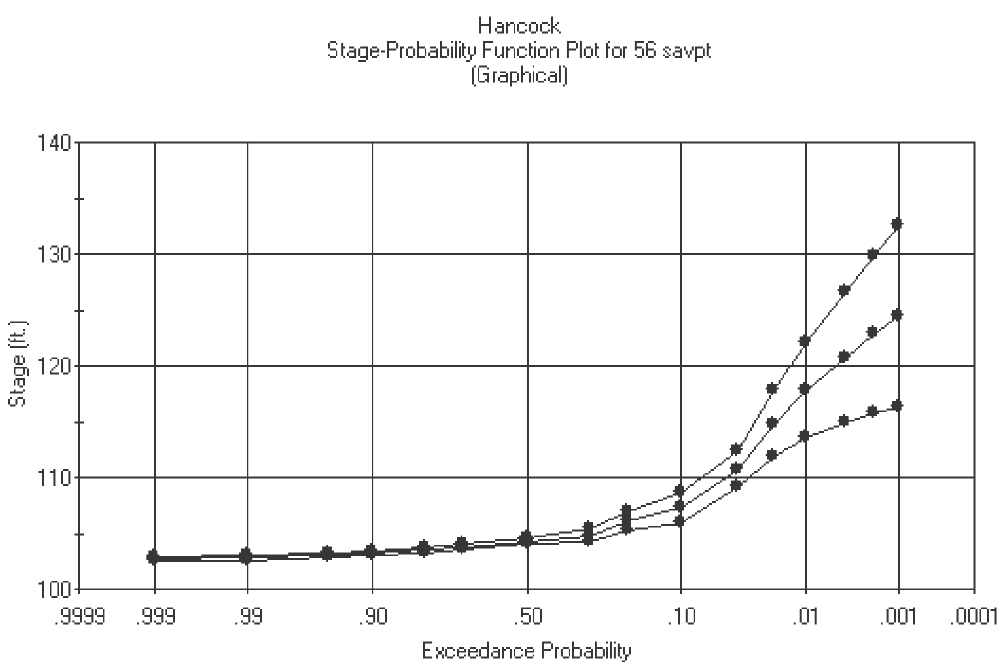


Figure 3.3.3-8. Existing Conditions at Save Point 56, near Bay St. Louis, MS

It should be noted that the frequency curve shown above reflects only that flooding resulting from storm surge in the gulf. Riverine flooding is not incorporated into this curve.

3.3.3.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 3591 acres around the most densely populated areas of Bay St. Louis as shown on the following Figure 3.3.3-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal.

Figure 3.3.3-9. Pump/Culvert/Sub-basin Site Locations

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in Figure 3.3.3-10.

Overtopping failures are caused by the high velocity of flow on the back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.3-11 was caused by approximately 1-2 ft of overtopping crest depth.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.3-10. North Sea, Germany, March 1976

Source: ERDC, Steven Hughes

Figure 3.3.3-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.3-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

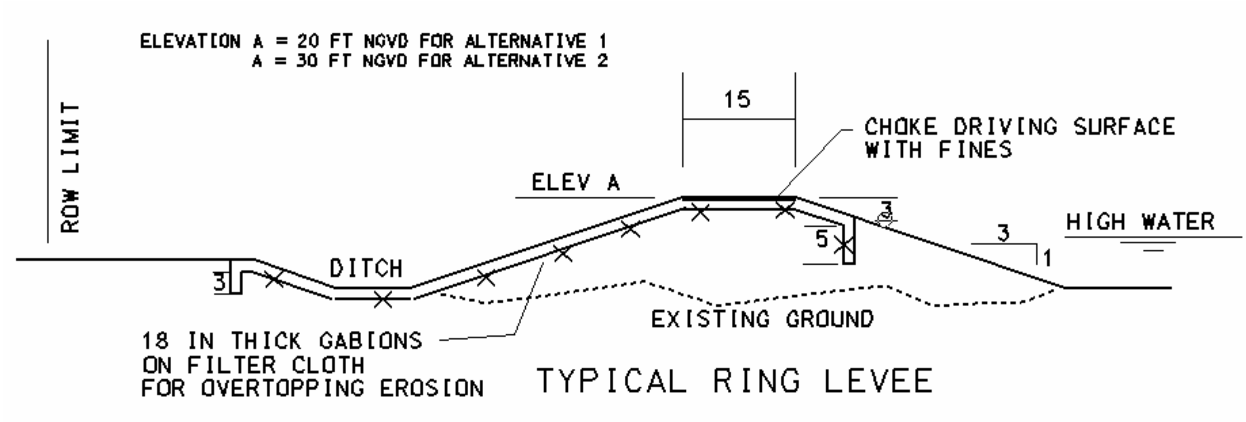


Figure 3.3.3-12. Typical Section at Ring Levee

3.3.3.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.3-13.

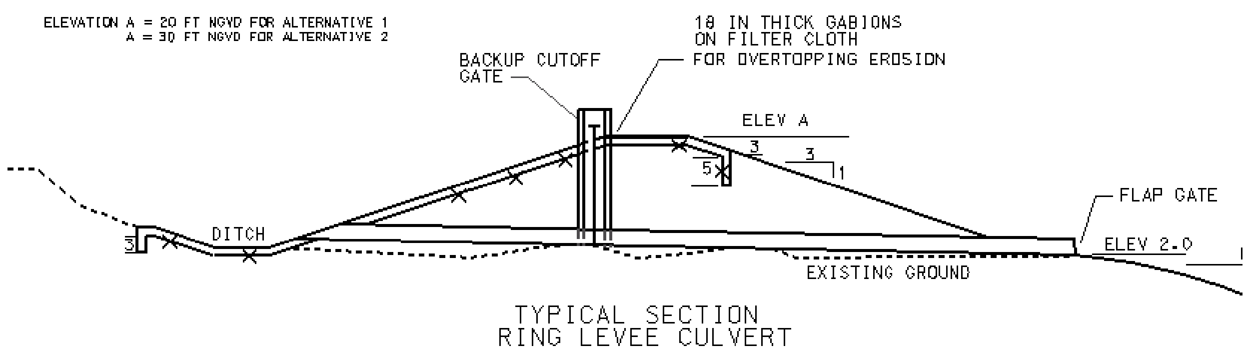


Figure 3.3.3-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown in Figure 3.3.3-9 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types and their hydrologic soil grouping and sub-basins are shown in Figure 3.3.3-14.

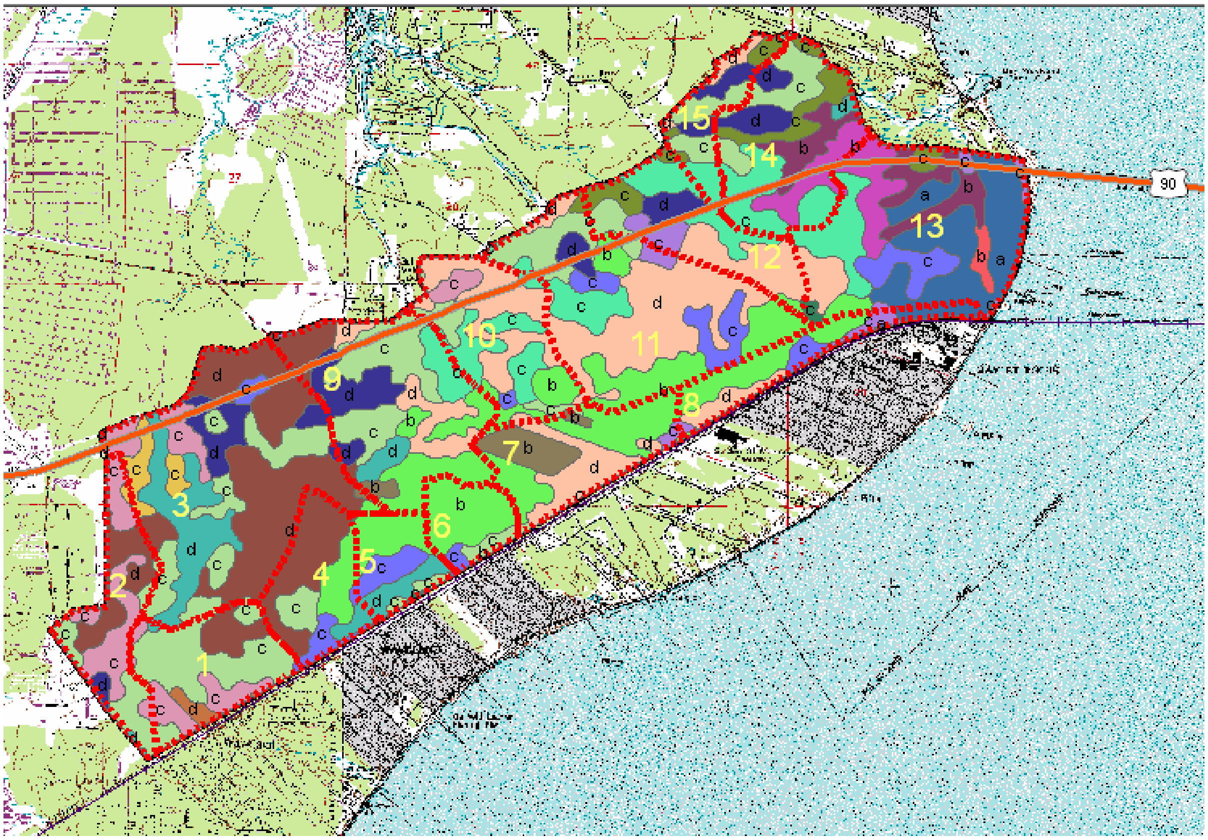


Figure 3.3.3-14. Bay St. Louis Hydrologic Soil Groups

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft above the upstream invert with a tailwater elevation of 2.0 ft above the downstream invert assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes

(And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.3.5.2 Geotechnical Data

Geology: Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.3.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts and pumping facilities. The sites are shown above.

3.3.3.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.3.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.3.5.3.3 Pumping Stations Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.3.5.3.4 Pumping Stations Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.3.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 12 pumping facilities included in the Bay St. Louis Ring Levee system for the elevation 20 protection level varied from approximately 10 feet to 15 feet and the corresponding flows required varied from 56,695 to 390,483 gallons per minute. The plants thus derived varied in size from a plant having two 36-inch diameter, 125 horsepower pumps, to one having eight 42-inch diameter pumps each running at 290 horsepower.

3.3.3.5.3.6 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.3.5.3.7 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bay St. Louis area to elevation 20, 21 roadway intersections would have to be accommodated. For this study it was estimated that of this number, 4 would require swing gate structures, with the rest requiring roller gates of various heights.

3.3.3.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.3.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.3.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.3.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.3.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.3.7.Cost Summary. Construction costs for the various options are included in Table 3.3.3-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.3-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.3.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.3.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Bay St. Louis. The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.3.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.3.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.3.6.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

Pumping Facilities. Flow and Pump Sizes. Option B. Design hydraulic heads derived for the 12 pumping facilities included in the Bay St. Louis Ring Levee system for the elevation 30 protection level varied from approximately 20 feet to 30 feet, and the corresponding flows required varied from 56,695 to 390,483gallons per minute. The plants thus derived varied in size from a plant having 2 36-inch diameter, 250 horsepower pumps, to one having eight 42-inch diameter pumps, each running at 475 horsepower.

3.3.3.6.3.1 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bay St. Louis area to elevation 30, 69 roadway intersections would have to be accommodated. For this study it was estimated that of this number, 62 would require swing gate structures, with the remaining 7 requiring roller gates of various heights.

3.3.3.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.3.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.3.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.3.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.3.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.3.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.3.7 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.3-1 and 3.3.3-2 below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.3-1.
Bay St Louis Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$283,000,000
Option B – Elevation 30 ft NAVD88	$382,900,000

Table 3.3.3-2.
Bay St Louis Ring Levee O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$2,002,000
Option B – Elevation 30 ft NAVD88	$2,803,000

3.3.3.8 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.4 Hancock County, Elevated Roadway

3.3.4.1 General

Residential and business areas along the coast in Hancock County are susceptible to storm surge damage. A damage reduction option is to raise the beach front road in Hancock County to elevation 11ft NAVD88 was evaluated. The levee alignment is shown in red below. Additional options not evaluated in detail are described elsewhere in this report. The option consists of more than one element and function. This option also contains a provision for a levee at elevation 16 ft NAVD88, shown in blue below. The elevation 16 ft NAVD88 levee functions in coordination with the Harrison County Elevated Hwy 90 Roadway also at elevation 16 ft NAVD and the St. Louis Bay closure structure.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.4.2 Location

The location of project in Hancock County is shown below in Figure 3.3.4-1.

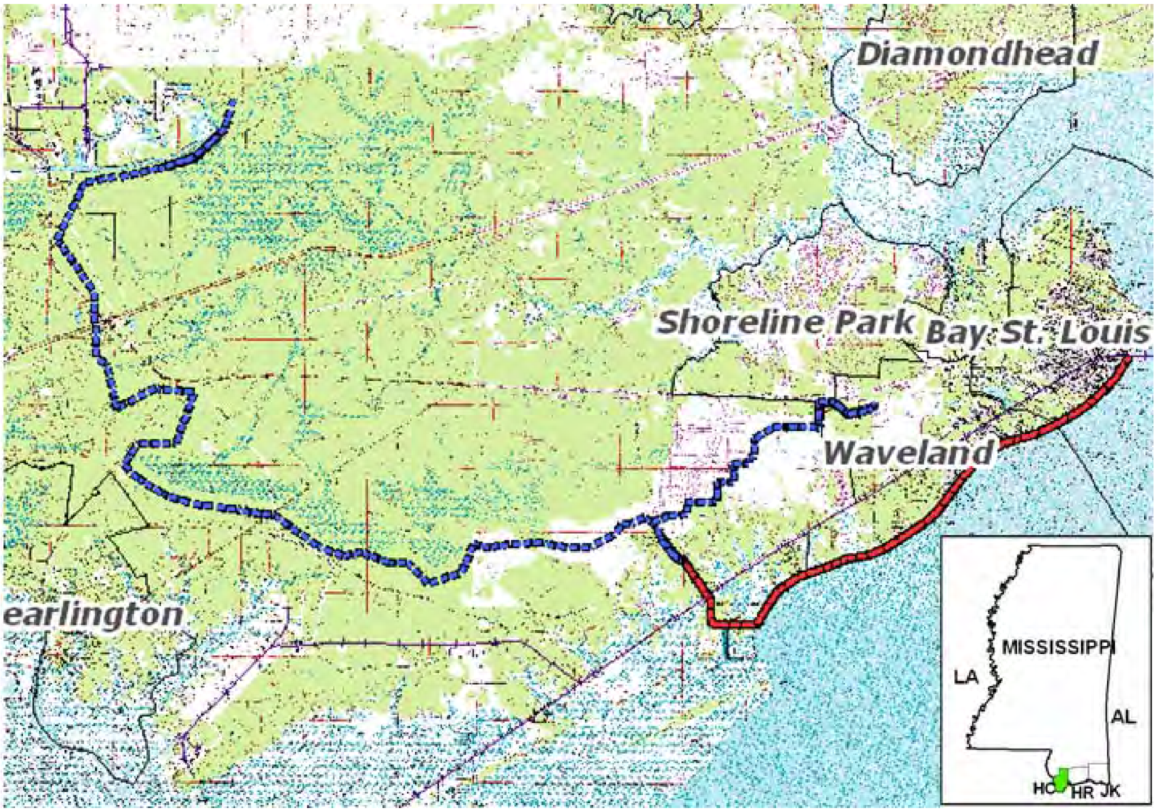


Figure 3.3.4-1. Vicinity Map near Waveland

3.3.4.3 Existing Conditions

The beach front road in Hancock County joins the communities of Bay St. Louis and Waveland at the mouth of St. Louis Bay. The 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown below in Figure 3.3.4-2.

Drainage at Bay St. Louis and Waveland is to the Mississippi Sound to the south and to tributaries of St. Louis Bay to the north. The Shoreline Park subdivision area to the north of Bay St. Louis is very low at elevations of 4-6 ft NAVD88 and subject to frequent flooding from storm surge.

Impacts from hurricanes can be devastating. Damage from Hurricane Katrina in August, 2005 in the Waveland area are shown below in Figures 3.3.4-3 and 3.3.4-4.

3.3.4.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.4-5. The data indicates the Katrina high water was as high as 28 ft NAVD88 near the Mississippi Sound, totally inundating the area.

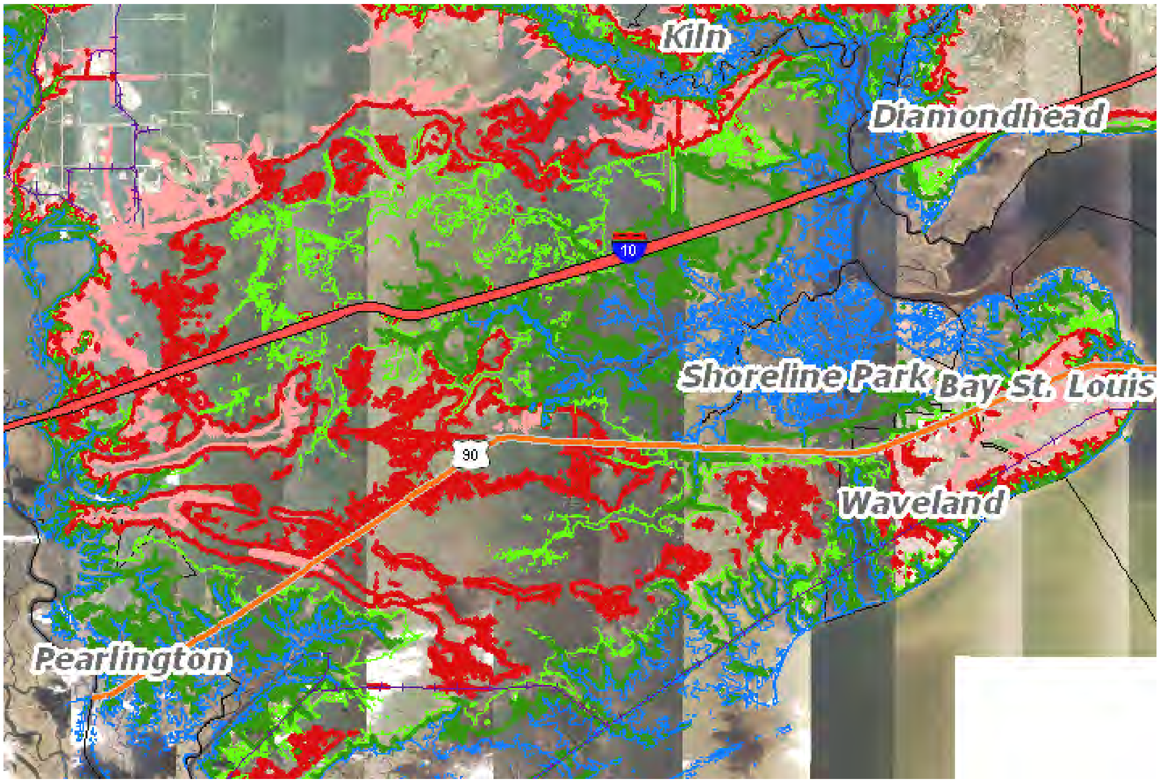
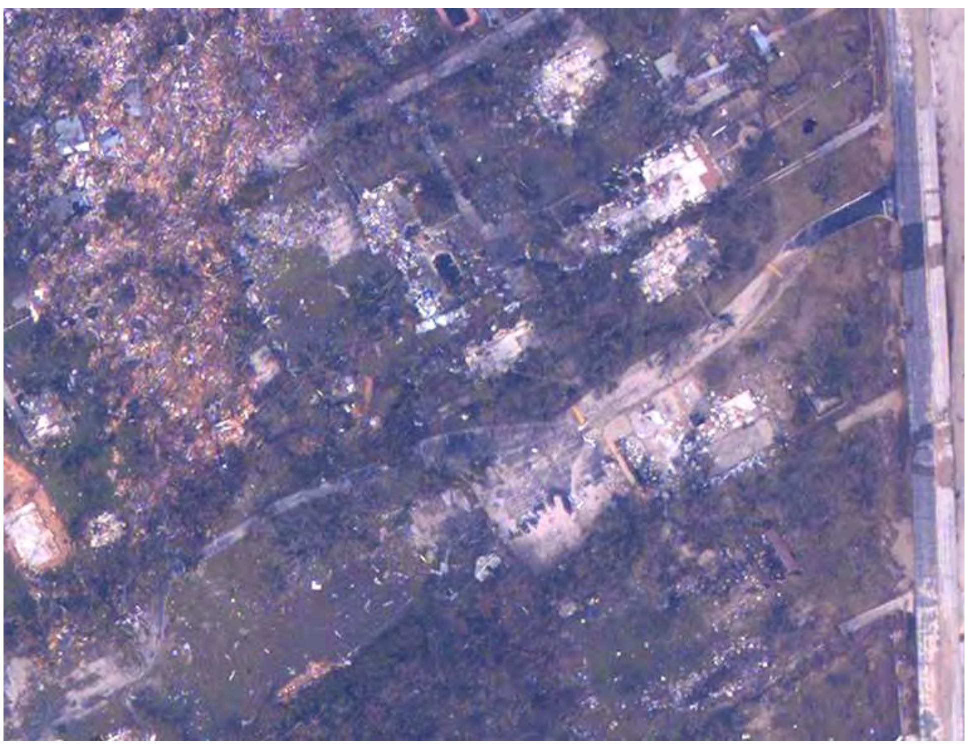


Figure 3.3.4-2. Existing Conditions near Waveland



Source: http://ngs.woc.noaa.gov/storms/katrina/24334552.jpg

Figure 3.3.4-3. Hurricane Katrina Damage near Waveland

Source:G.J. Charlet III, http://www.flickr.com/photo_zoom.gne?id=46937047&size=m

Figure 3.3.4-4. Hurricane Katrina Damage near Waveland

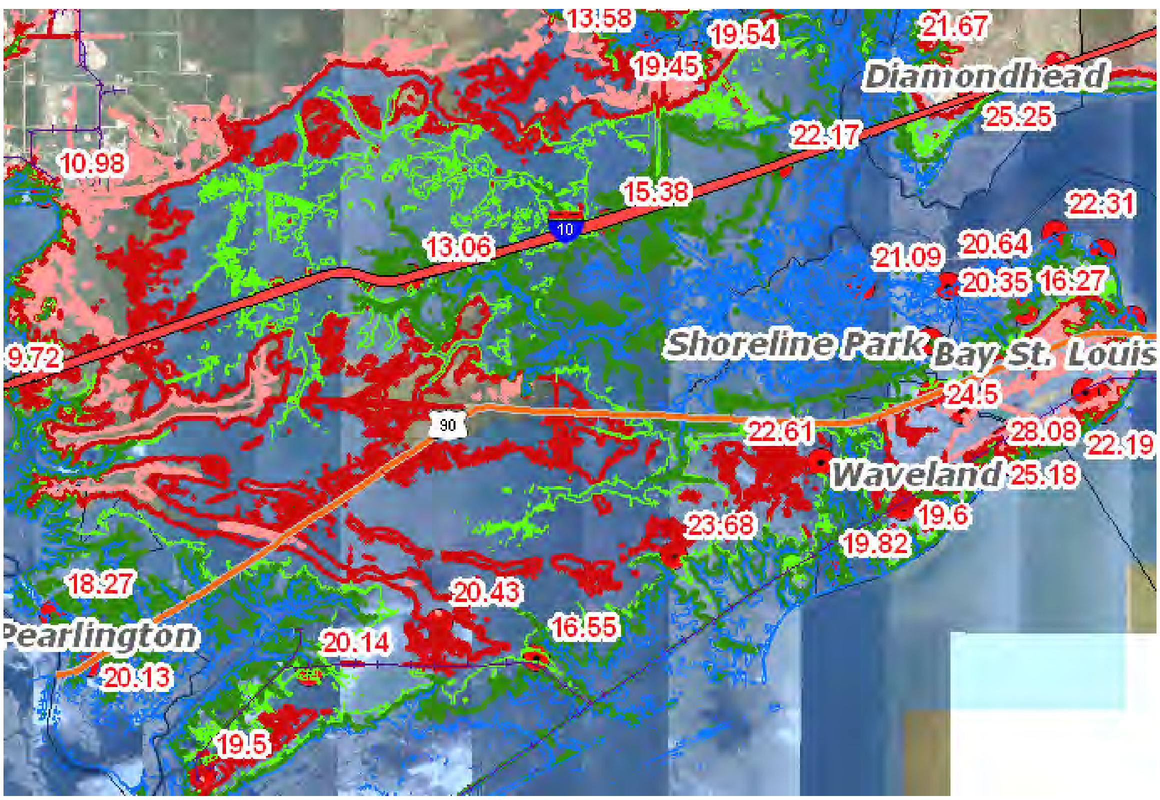


Figure 3.3.4-5. Ground Contours and Katrina High Water, Coastal Hancock Co.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Ocean Springs at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.4-6.

Existing Condition Stage –Frequency data for Save Point 56, just off the coast of Waveland, is shown below as an example in Figure 3.3.4-7. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

3.3.4.5 Option – Elevate Roadway to 11 ft NAVD88

This option consists of raising the beach front road to elevation 11 ft NAVD88 in the Bay St. Louis/Waveland area as shown on the following Figures 3.3.4-8 and 3.3.4-9, along with the internal sub-basins and levee culvert/pump locations. There is one culvert but no pumps associated with the Elevation 16 ft NAVD88 levee. This levee runs mostly along the ridge line so the drainage is away from the levee. A small boat access structure is also shown at the mouth of one basin. Rising sector gates will be provided at this gate allowing shallow draft traffic most of the time. The gate will be closed prior to hurricane storm surge. A drawing of a typical boat access gate is shown in Figure 3.3.11-15.

Figure 3.3.4-6. Hydrodynamic Modeling Save Points near Waveland

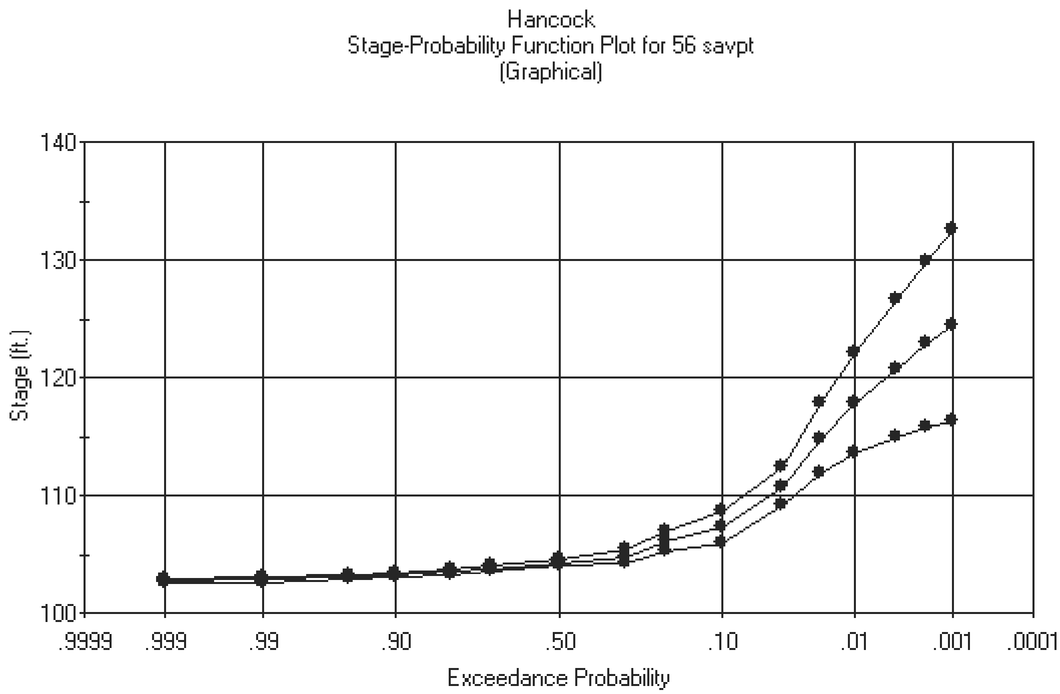


Figure 3.3.4-7. Existing Conditions at Save Point 56, near Waveland

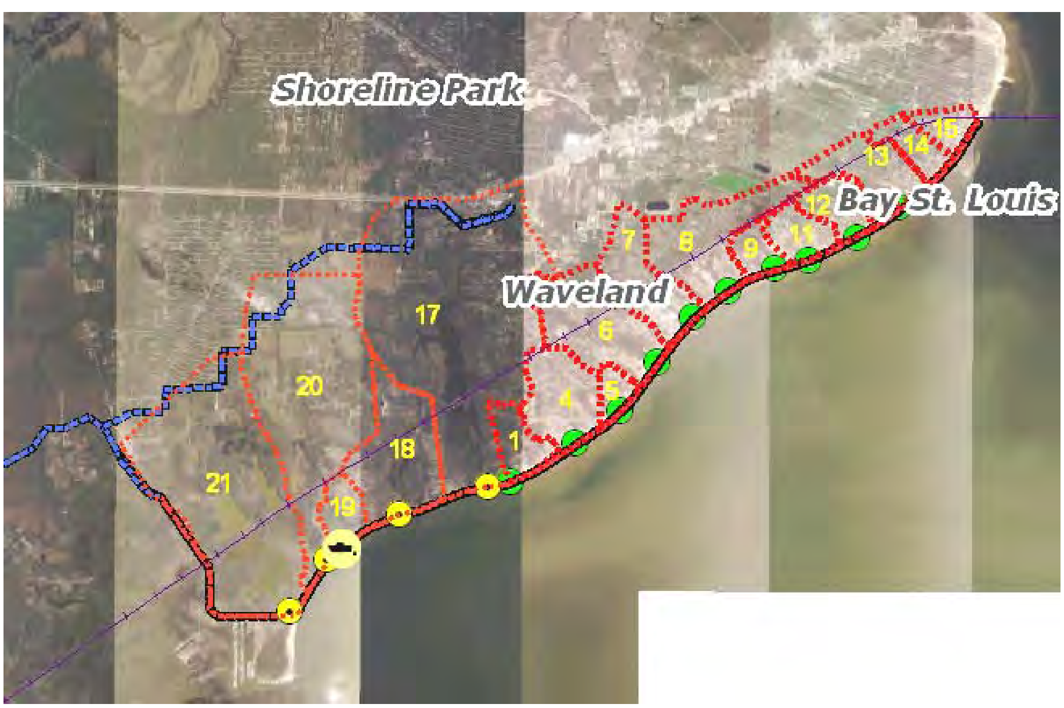


Figure 3.3.4-8. Pump/Culvert/Boat Access Site Locations and Sub-basins

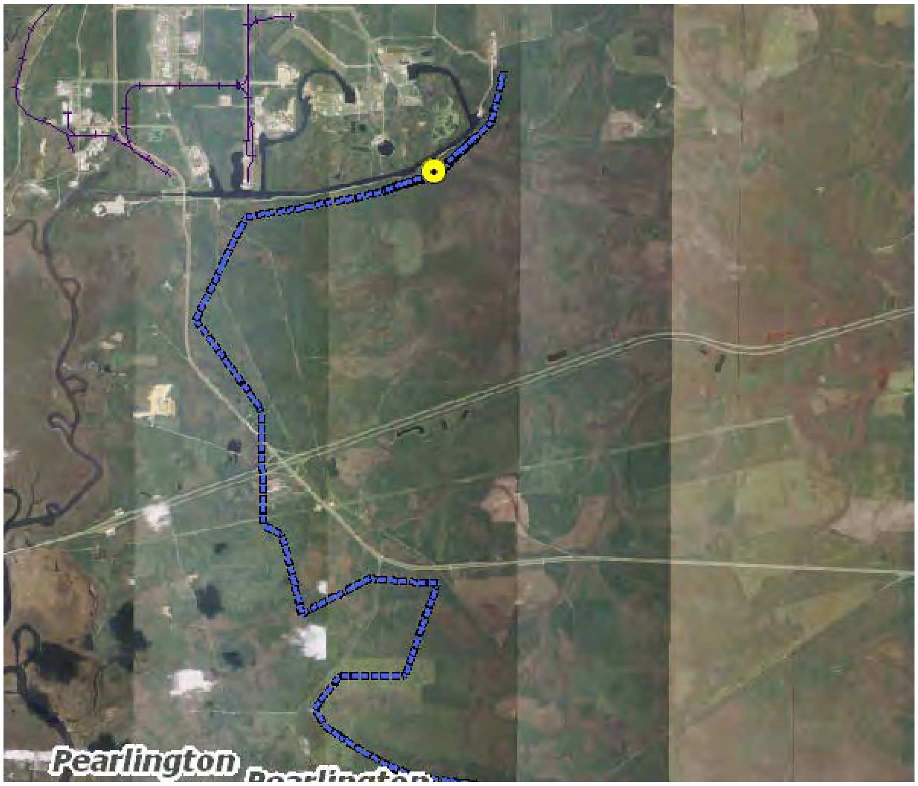


Figure 3.3.4-9. Culvert Site Location

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown on Figure 3.3.4-10.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.4-10. North Sea, Germany, March 1976

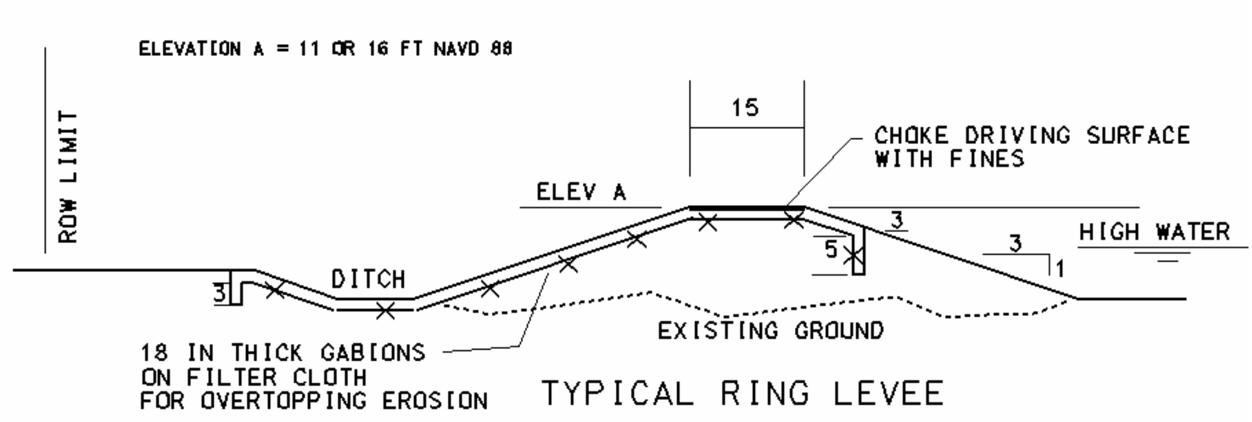
Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.4-11 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.3.4-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown above on Figure 3.3.4-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.



Source: ERDC, Steven Hughes

Figure 3.3.4-12. Typical Section at Ring Levee

3.3.4.5.1 Interior Drainage

Drainage on the interior of the raised roadway would be collected at the highway and channeled to culverts placed at locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.4-13.

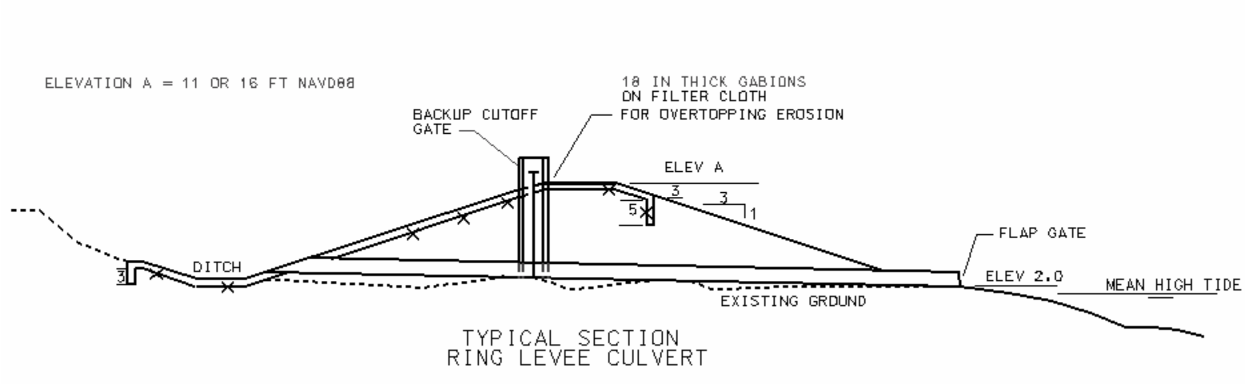


Figure 3.3.4-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the drainage basin into major sub-basins as shown above and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater/tailwater elevation difference was maintained at 3.0 ft or less. Drainage ditches along the toe of the highway will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.4.5.2 Geotechnical Data

Geology: The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The Line 3 defense elevates the roadway and accompanying seawall to elevation by extending the seaway at its present slope to grade, creating the roadway subgrade then sloping the backside to one vertical to three horizontal side slopes with a twenty five foot toe width for access and drainage. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The embankment will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface on the back side will be armored by the placement of 12 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the road. The armoring will be anchored on the back face by trenching and extend across the toe easement. All non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate ramping over the embankment where the surface elevation is near that of the crest elevation. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the roadway and the potential for erosion and instability must be considered. Final designs may require the installation of a cutoff wall within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.4.5.3 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 12 pumping facilities included in the Hancock County Raised Roadway at the elevation 11 protection level was constant at 7 feet, and the corresponding flows required varied from 78,994 to 263,913 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 150 horsepower pumps, to one having four 60-inch diameter pumps each running at 750 horsepower.

3.3.4.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.4.5.5 Construction Procedures and Water Control Plan

Construction would be done by heavy construction equipment after removal of structures and relocation of utilities. Water control will be addressed by constructing drainage facilities prior to construction of the levee.

3.3.4.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Boat access gates and power plants would require this level of security.

3.3.4.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Maintenance costs are included in this report.

3.3.4.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.4.6, Cost Summary. Construction costs for the various options are included in Table 3.3.4-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.4-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and

coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.4.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.4.6 Hancock County. Elevated Roadway. Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.4-1 and 3.3.4-2 below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.4-1.
Hancock Co Elevated Roadway Construction Cost Summary

Option	Total project cost
Option - Elevated Roadway	$328,000,000

Table 3.3.4-2.
Hancock Co Elevated Roadway O & M Cost Summary

Option	O&M Cost
Option A – Elevated Roadway	$3,831,000

3.3.4.7 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

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Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.5 Harrison County, Elevated Roadway

3.3.5.1 General

Residential and business areas along the coast in Harrison County are susceptible to storm surge damage. A damage reduction option is to raise Highway 90 to elevation 16ft NAVD88 was evaluated. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.5.2 Location

The location of Hwy 90 in Harrison County is shown below in Figure 3.3.5-1 extending from Biloxi Bay to Pass Christian.

Figure 3.3.5-1. Vicinity Map, Harrison County

3.3.5.3 Existing Conditions

In Harrison County, ground elevations over most of the residential and business areas vary between elevation 8-12 ft NAVD88 on the coast and rising within 1000 ft to elevation 30-36 along a ridge parallel to the coast line, then decreasing to the north. The 4-ft (blue), 8-ft (green), 20-ft (pink), 30-ft (dark blue) and 34-ft (gold) ground contours shown the pattern at the coastline for the county below in Figure 3.3.5-2.

A close-up near Keesler Air Force Base is shown below. The 4-ft(blue), 8-ft(dark green), 12-ft(light green), 16-ft(brown), 20-ft(pink), 24-ft(light purple), 28-ft (teal), and 32-ft (gold) ground contour lines are shown below in Figure 3.3.5-3.

The area is drained by natural and some improved channels. Above the ridge water drains to the north, thence to either the Back Bay of Biloxi on the east side of the county, or to the west to the St Louis Bay. South of the ridge, the water drains to Mississippi Sound.

Drainage from ordinary rainfall is hindered on occasions when either of the rivers in the area or the gulf is high, but impacts from hurricanes are devastating.

Damage from Hurricane Katrina in August, 2005 in the Pascagoula area are shown below in Figures 3.3.5-4 and 3.3.5-5. Many homes are still un-repaired, pending settlement of insurance claims.

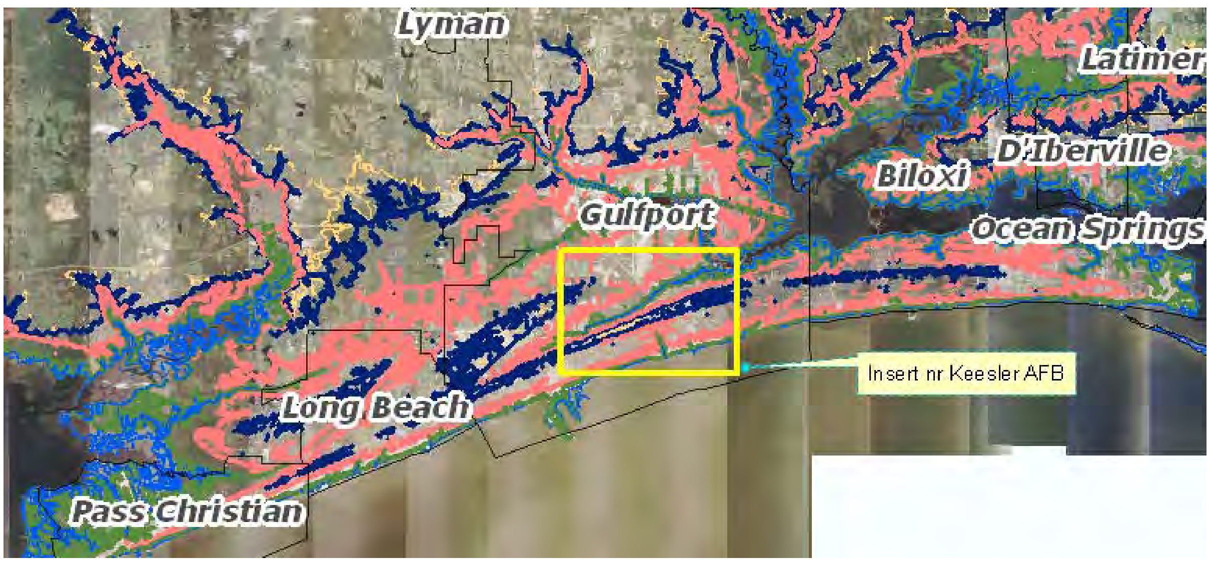


Figure 3.3.5-2. Existing Conditions, Harrison County

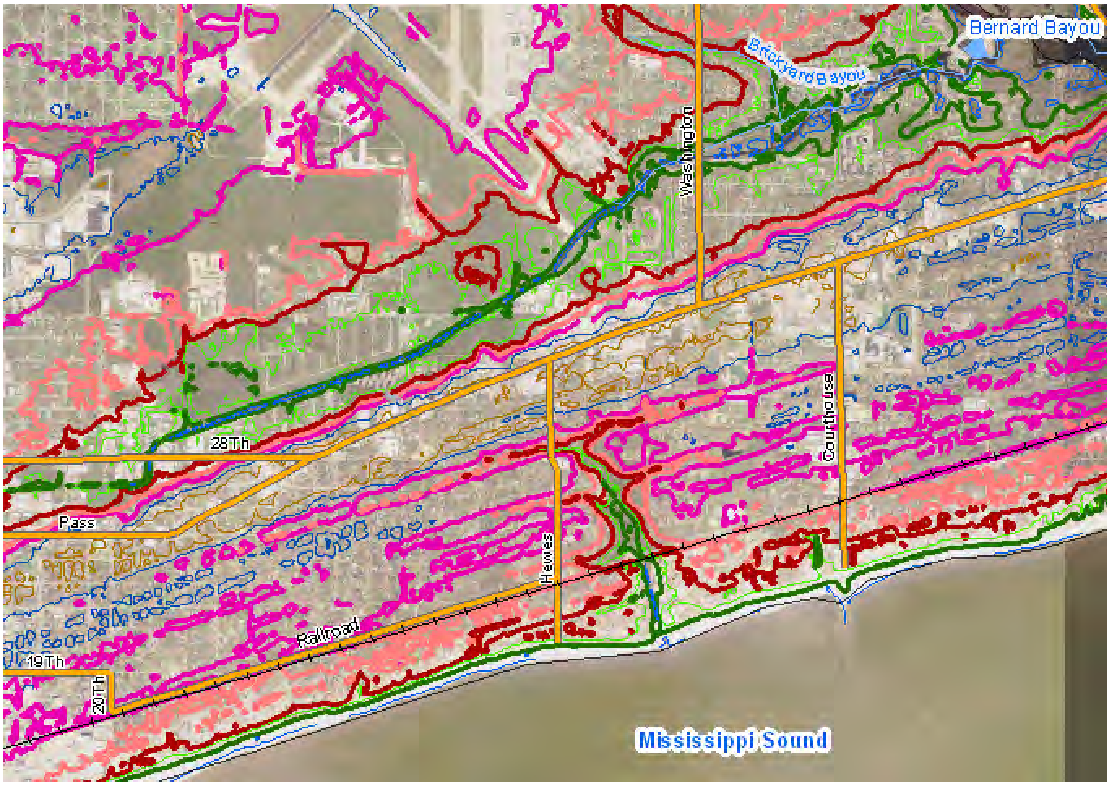


Figure 3.3.5-3. Existing Condition near Keesler AFB

Source : http://ngs.woc.noaa.gov/storms/katrina/24330924.jpg

Figure 3.3.5-4. Hurricane Katrina Damage, Harrison County

Source: danakay, http://www.flickr.com/photo_zoom.gne?id=45235550&size=m

Figure 3.3.5-5. Hurricane Katrina Damage, Harrison County

3.3.5.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the Katrina inundation limits are shown below in Figure 3.3.5-6 and 3.3.5-7. The data indicates the Katrina high water was as high as 21 ft NAVD88 Biloxi, and 28 ft NAVD88 at Pass Christian.

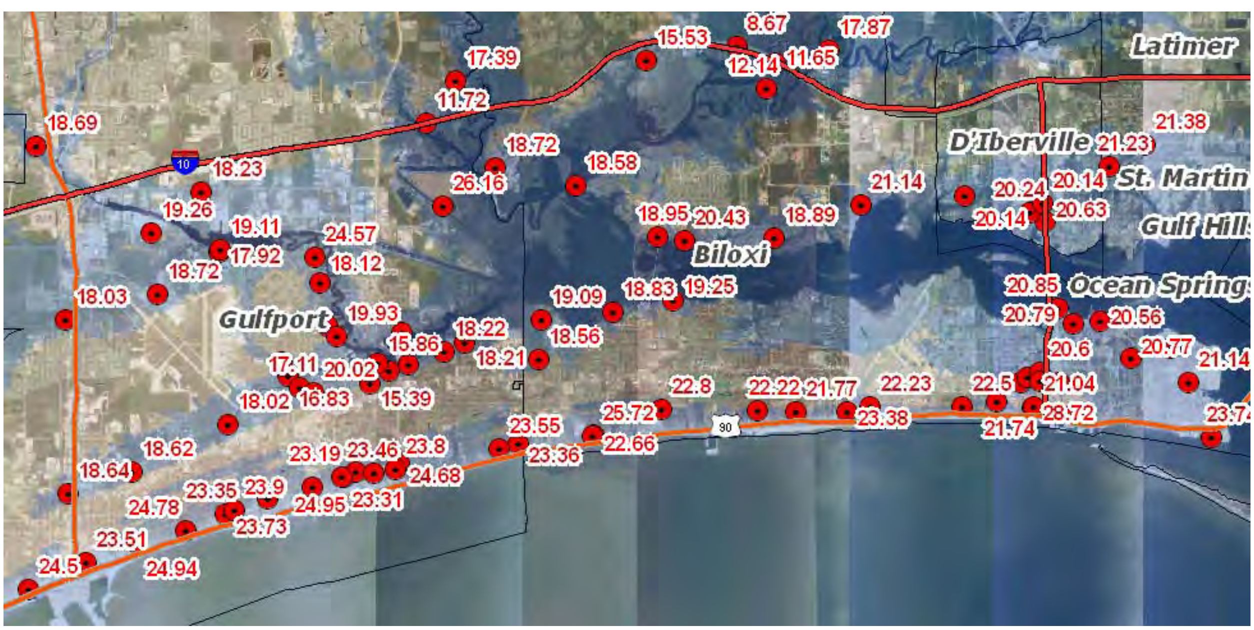


Figure 3.3.5-6. Katrina High Water Elevations, Harrison County

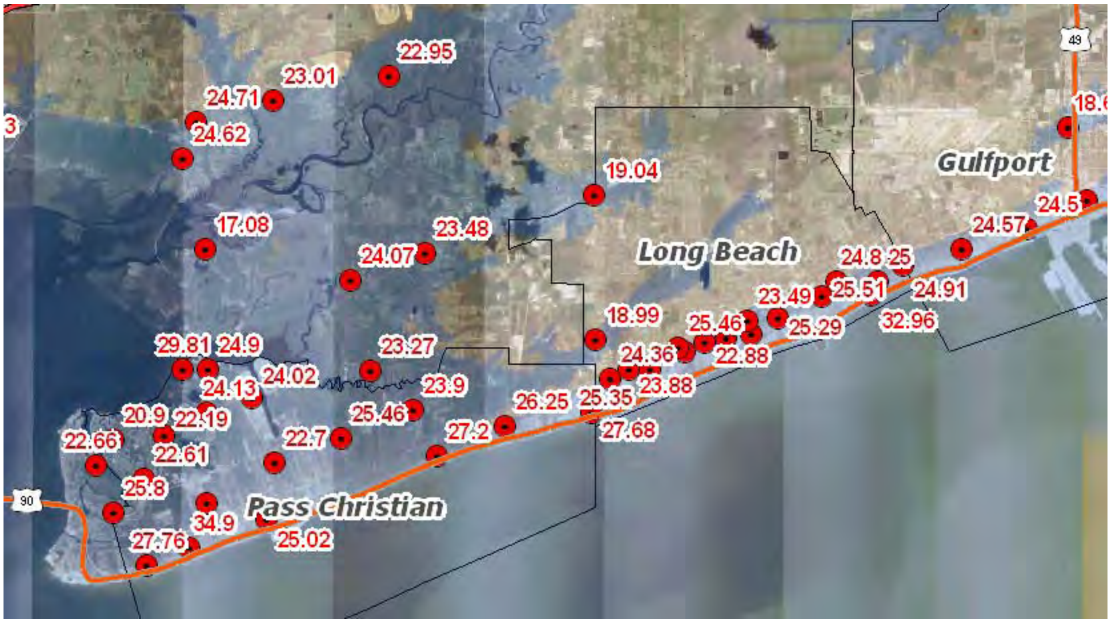


Figure 3.3.5-7. Katrina High Water Elevations, Harrison County

A closer view at the intersection of Hwy 90 and US Hwy 49 in Gulfport of existing flooding potential along Harrison County is shown below in Figure 3.3.5-8. Ground contours shown are 4-ft(blue), 8-ft(dark green), 12-ft(light green), 16-ft(brown), 20-ft(pink), 24-ft(light purple), 28-ft (teal), and 32-ft (gold).

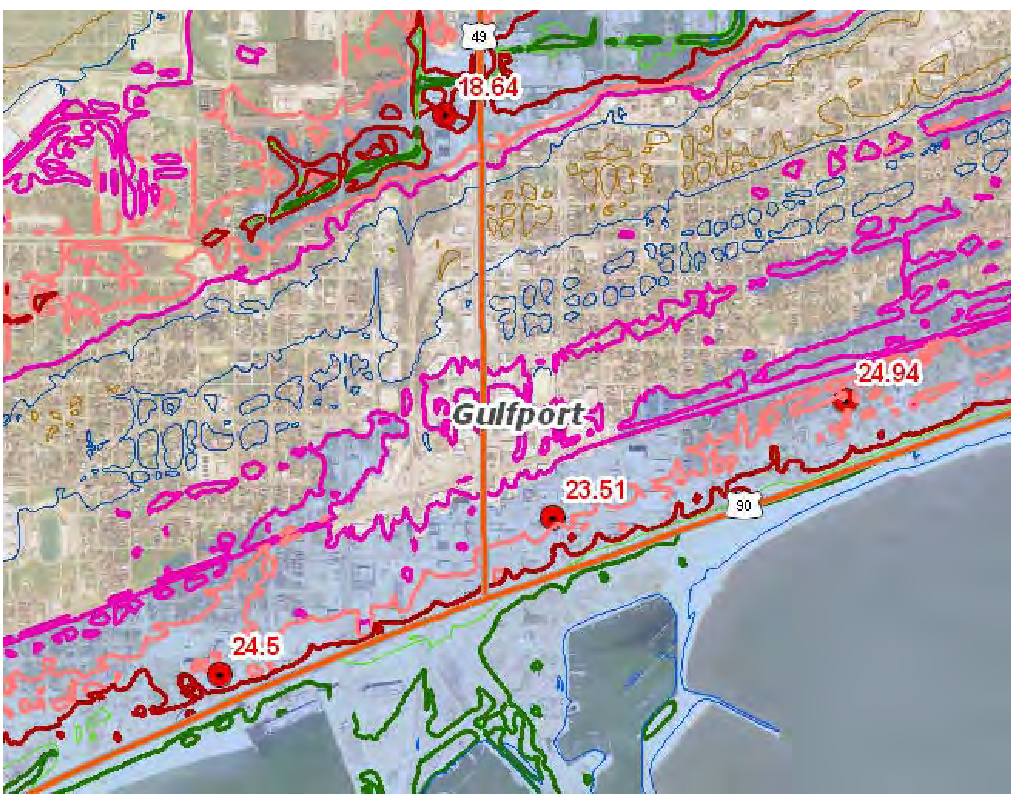


Figure 3.3.5-8. Ground Contours and Katrina High Water Elevations at Hwy 49

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near the coast in Harrison County at which data from hydrodynamic modeling was saved are shown below in Figures 3.3.5-9 and 3.3.5-10.

Existing Condition Stage –Frequency data for Save Point 50, just off the coast of Harrison County, is shown below as an example in Figure 3.3.5-11. The 95% confidence limits, approximately equal to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

3.3.5.5 Option – Elevate US Highway 90 to Elevation 16.0 ft NAVD88

This option consists of raising US Hwy 90 to elevation 16 ft NAVD88 along the coast of Harrison County as shown on the following Figures 3.3.5.12 through 3.3.5.15, along with the internal sub-basins and levee culvert/pump locations.

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown below on Figure 3.3.5-16.

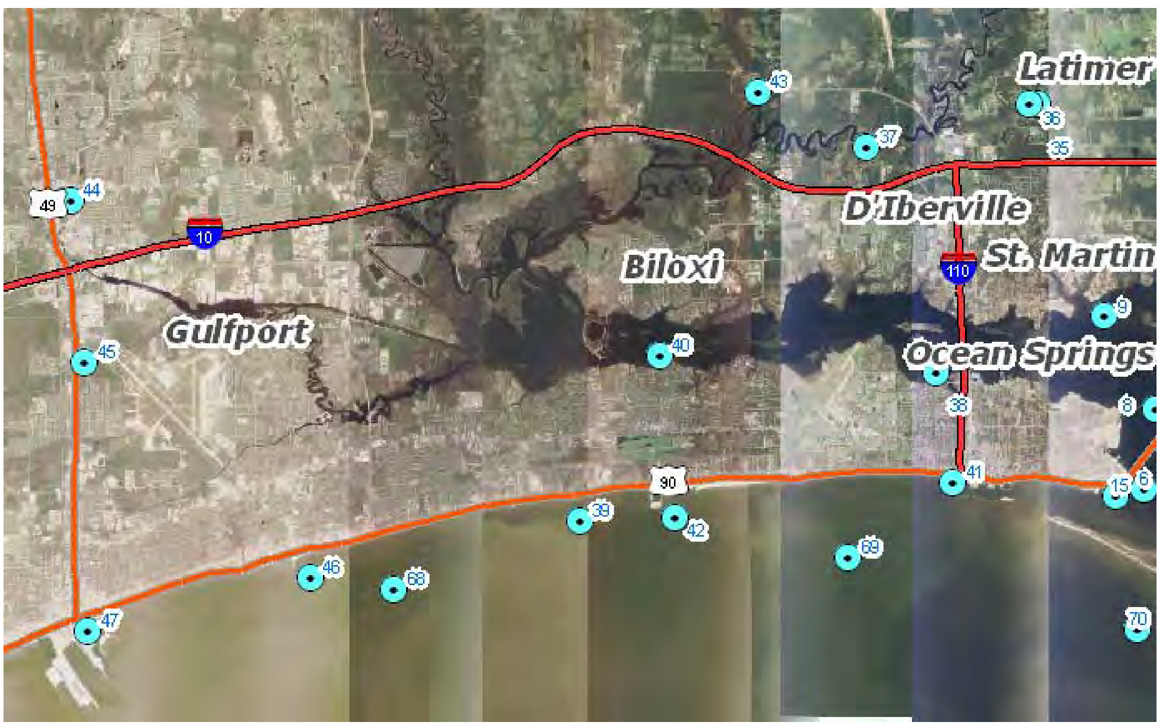


Figure 3.3.5-9. Hydrodynamic Modeling Save Points in Harrison County

Figure 3.3.5-10. Hydrodynamic Modeling Save Points in Harrison County

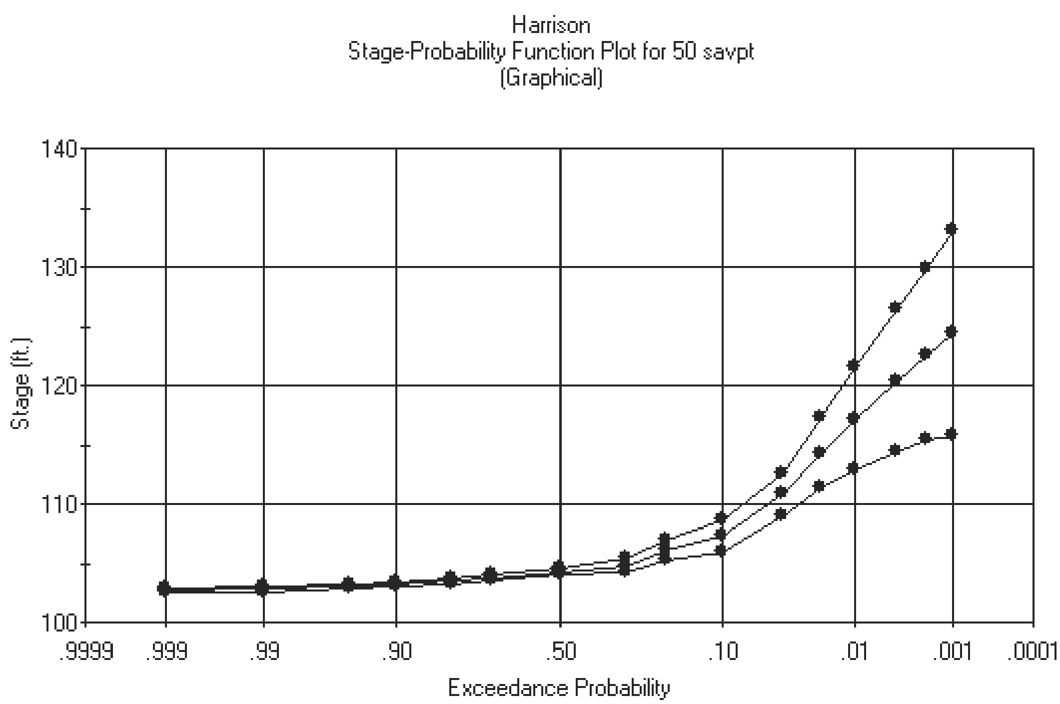


Figure 3.3.5-11. Existing Conditions at Save Point 50, near Pass Christian, MS

Figure 3.3.5-12. Pump/Culvert/Sub-basin Site Locations, Harrison County

Figure 3.3.5-13. Pump/Culvert/Sub-basin Site Locations, Harrison County

Figure 3.3.5-14. Pump/Culvert/Sub-basin Site Locations, Harrison County

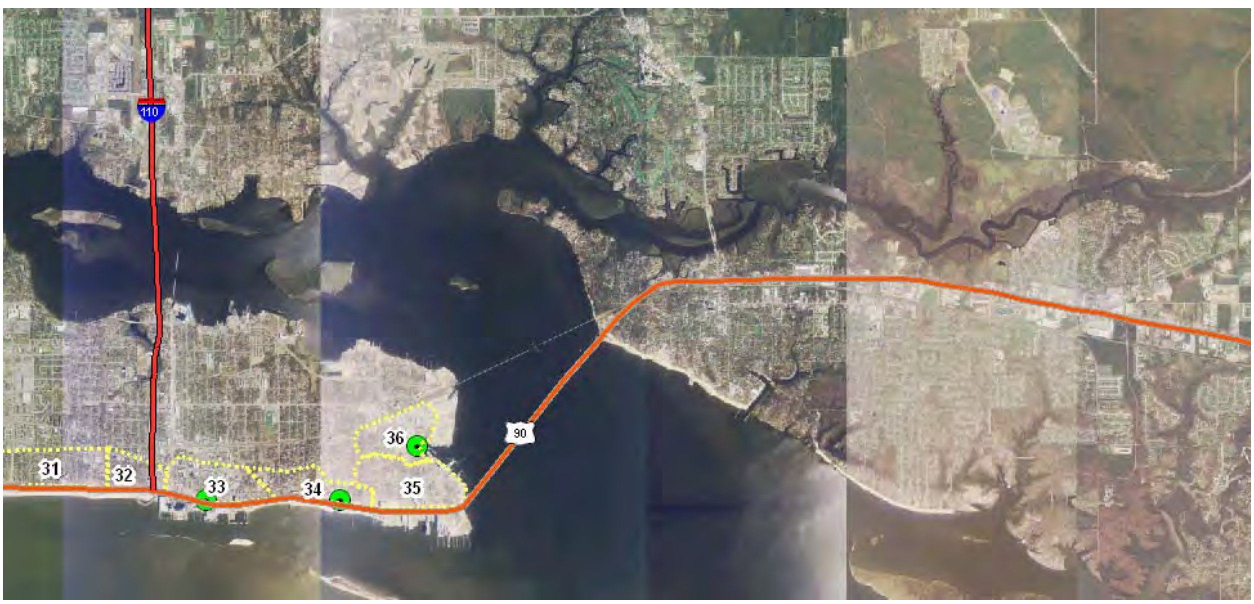


Figure 3.3.5-15. Pump/Culvert/Sub-basin Site Locations, Harrison County

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.5-16. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown on Figure 3.3.5-17, below was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.3.5-17. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown above in Figure 3.3.5-18, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

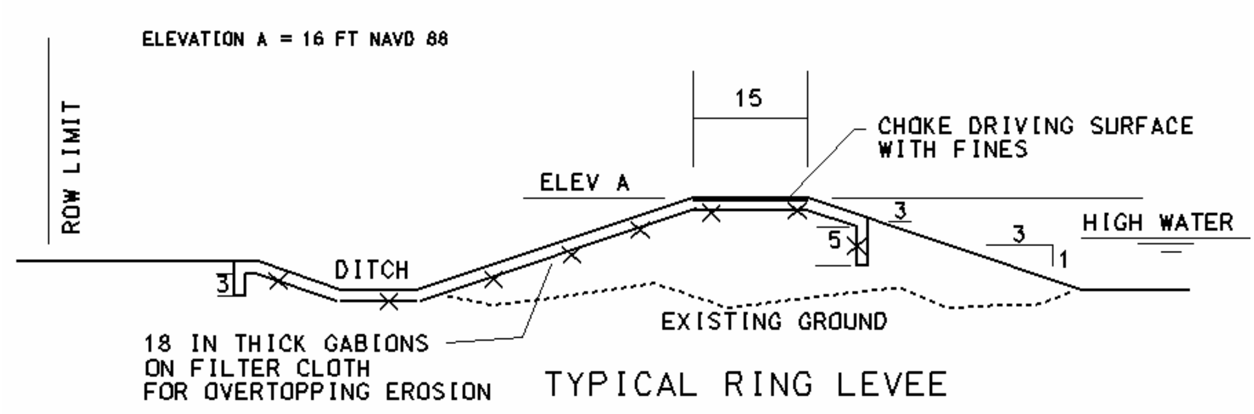


Figure 3.3.5-18. Typical Section at Ring Levee

3.3.5.5.1 Interior Drainage

Drainage on the interior of the raised highway would be collected at the highway and channeled to culverts placed at locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.5-19.

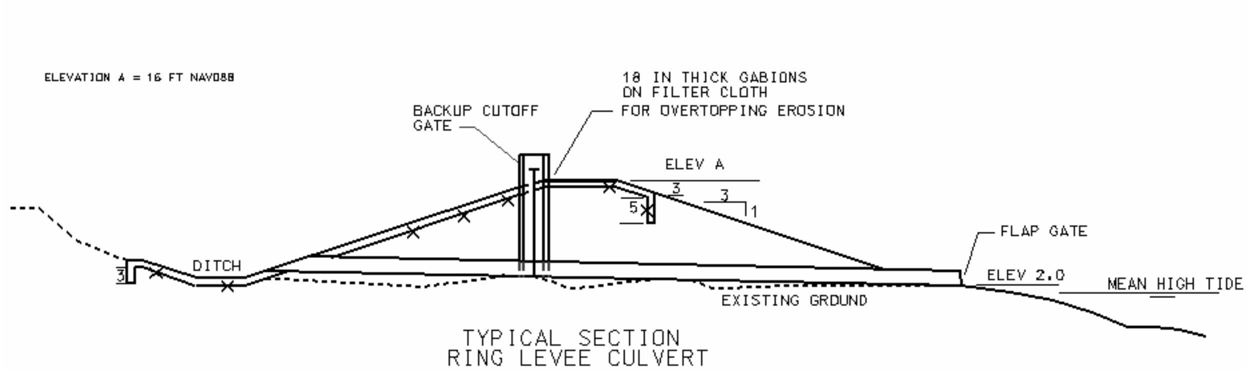


Figure 3.3.5-19. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the drainage basin into major sub-basins as shown above and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater/tailwater elevation difference was maintained at 3.0 ft or less. Drainage ditches along the toe of the highway will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.5.5.2 Geotechnical Data

Geology. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The Line 3 defense elevates the roadway and accompanying seawall to elevation by extending the seaway at its present slope to grade, creating the roadway subgrade then sloping the backside to one vertical to three horizontal side slopes with a twenty five foot toe width for access and drainage. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The embankment will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface on the back side will be armored by the placement of 12 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the road. The armoring will be anchored on the back face by trenching and extend across the toe easement. All non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate ramping over the embankment where the surface elevation is near that of the crest elevation. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the roadway and the potential for erosion and instability must be considered. Final designs may require the installation of a cutoff wall within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.5.5.3 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 15 pumping facilities included in the Harrison County Raised Roadway at the elevation 16 protection level was constant at 7 feet, and the corresponding flows required varied from 39,945 to 515,258 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 475 horsepower pumps, to one having six 60-inch diameter pumps each running at 1000 horsepower.

3.3.5.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.5.5.5 Construction Procedures and Water Control Plan

Construction would be done by heavy construction equipment after removal of structures and relocation of utilities. Water control will be addressed by constructing drainage facilities prior to construction of the levee.

3.3.5.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would

possess the highest threat level of all the critical assets. Boat access gates and power plants would require this level of security.

3.3.5.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Maintenance costs are included in this report.

3.3.5.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.5.6, Cost Summary. Construction costs for the various options are included in Table 3.3.5-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.5-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.5.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.5.6 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.5-1 and 3.3.5-2 below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.5-1.
Harrison Co Elevated Roadway Construction Cost Summary

Option	Total project cost
Option - Elevated Roadway	$1,989,200,000

Table 3.3.5-2.
Harrison Co Elevated Roadway O & M Cost Summary

Option	O&M Cost
Option A – Elevated Roadway	$19,586,000

3.3.5.7 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.6 Forrest Heights Levee, City of Gulfport, Harrison County

3.3.6.1 General

The culturally historical Forest Heights residential community in the City of Gulfport, Harrison County, Mississippi, has frequently been inundated by flood waters due to storm surges from the Mississippi Sound and from inland flooding along the lower Turkey Creek. Water reached a depth of 2-8 ft over the entire community during Hurricane Katrina inundation. The Forest Heights levee is proposed to be constructed as a pilot project for the MsCIP comprehensive plan. The levee will address the combination of storm surge protection, inland flooding protection, and evacuation. The levee is intended to be constructed to a height such that the levee might be certified under the National Flood Insurance Program. A preliminary engineering analysis suggests a levee built to approximately elevation 21 feet NAVD ’88 would satisfy or exceed certification elevation criteria.

Engineering performance and economic evaluations of protection options were done using the Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA. HEC-FDA modeling was done using variations in with-project conditions compared to the future without-project conditions for the Turkey Creek study. Details regarding the methodology are presented in the Economic Appendix. Additional evaluation to determine the precise levee height will be performed during final engineering and design based upon analyzing the risk and uncertainty associated with the coincident occurrence of inland flooding and storm surge impacts.

3.3.6.2 Location

The Forrest Heights community is located in an area known as North Gulfport within the City of Gulfport on the Mississippi Gulf Coast. The location of the levee at Forrest Heights is shown below in Figures 3.3.6-1 and 3.3.6-2. The community lies along the lower Turkey Creek floodplain, which has a tendency to frequently exceed its stream channel capacity and flood adjacent low-lying areas.

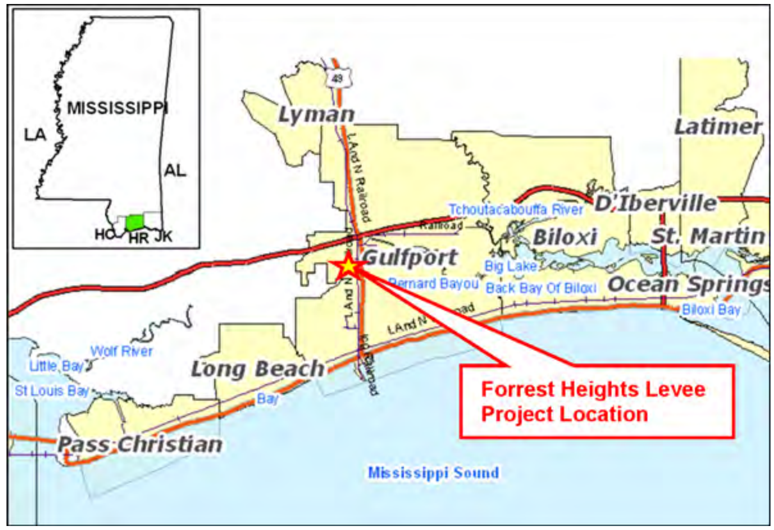


Figure 3.3.6-1. Vicinity Map

Figure 3.3.6-2. Forrest Heights Ring Levee Location

3.3.2.3 Existing Conditions

The community of Forrest Heights lies on the bank of Turkey Creek about 2.6 miles from the mouth at Bernard Bayou. Ground elevations over most of the residential area are between elevations 10-14 ft NAVD88. Drainage is mostly along streets and through natural drainage ways to the Turkey Creek. Impacts from flooding and hurricanes have been devastating. Hurricane Katrina in August, 2005 resulted in significant flood damages to residences in the Forrest Heights community. A levee with top width of 6 ft was constructed around the community to elevation 16.5 ft NGVD with sideslopes of 1 vertical to 1.5 horizontal in 1969, prior to Hurricane Camile. It has not had adequate maintenance and is a state of disrepair. It is scheduled to be restored to as-built condition by January of 2009. However, the restored levee will not be sufficient to meet the present day standard for certification according to the existing FEMA flood profiles in the vicinity. It is assumed that the as-built condition of this restored levee will be the existing condition for this report.

3.3.6.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), 20-ft(orange), and 20-ft(pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.6-3. The data indicates the water was as high as 18-20 ft NAVD88 near the site, totally inundating the entire area.

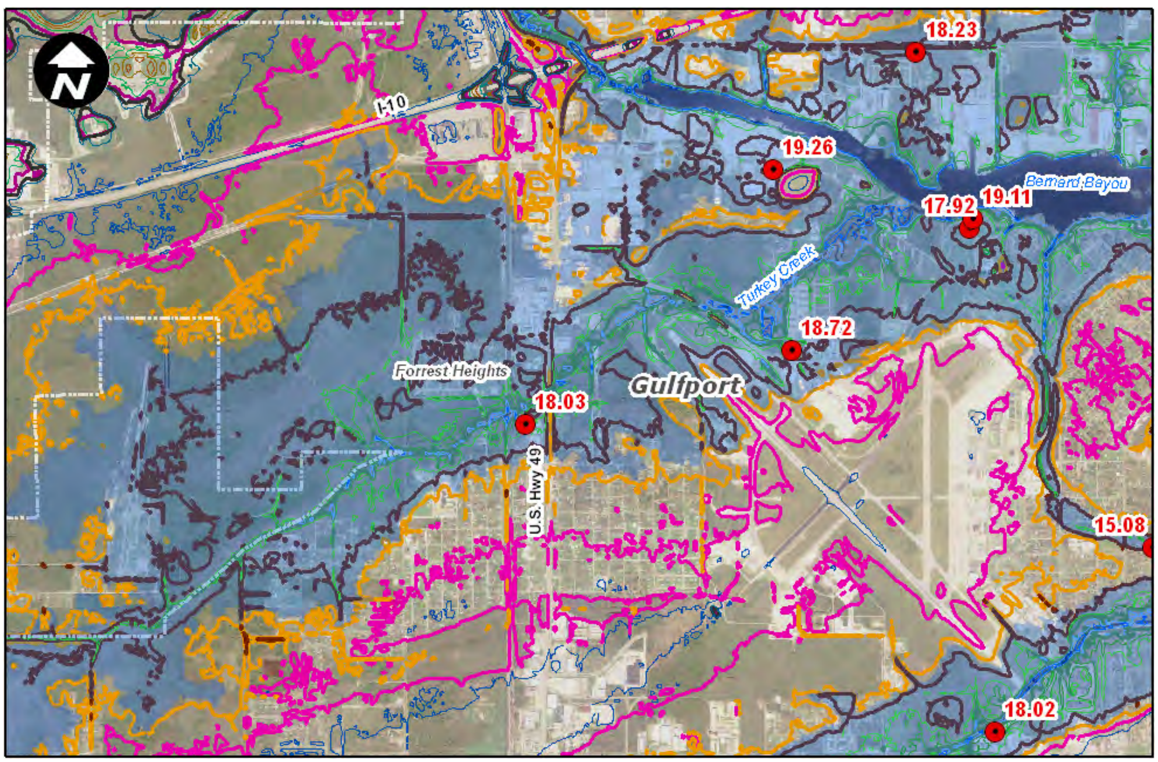


Figure 3.3.6-3. Hurricane Katrina Inundation and High Water, Forrest Heights

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical coastal tide gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area. An expanded description of the procedure is presented in Section 2.13 of the

Engineering Appendix and in the Economic Appendix. Points near Forrest Heights at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.6-4, and the stage frequency curve for that location is shown in Figure 3.3.6-5. Hydrodynamic output stage-frequency pairs, with uncertainty, are displayed in Table 3.3.6-1.

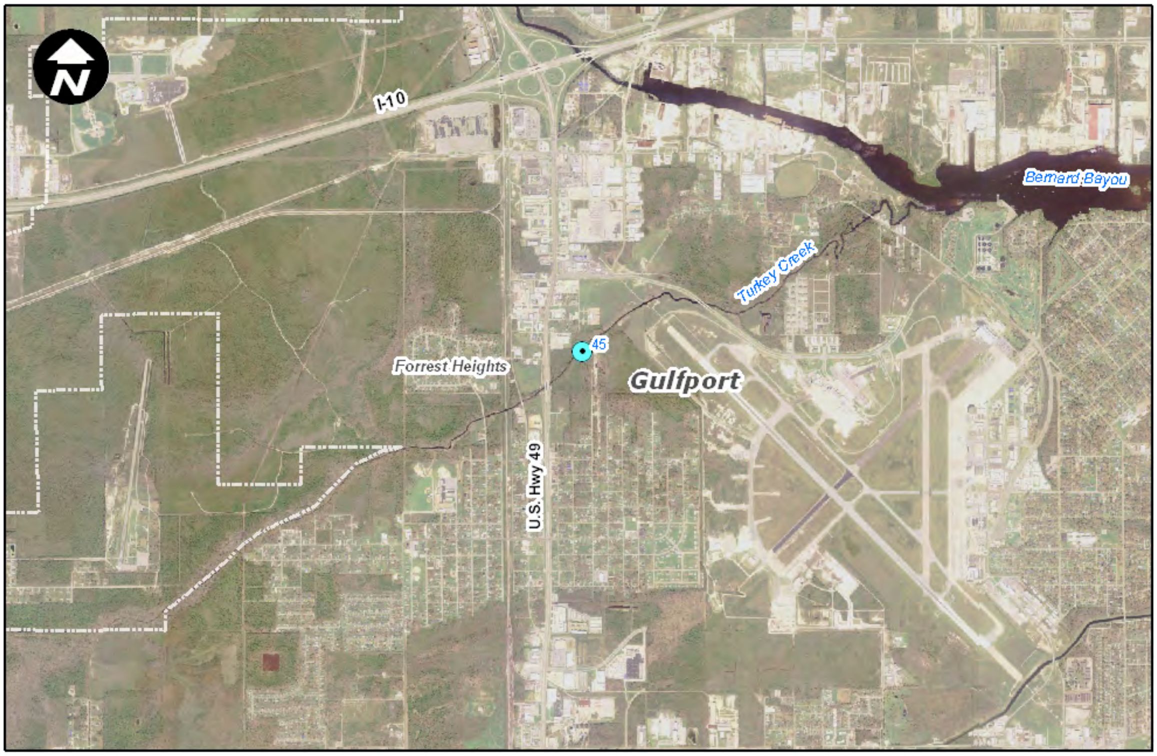


Figure 3.3.6-4. Hydrodynamic Modeling Save Point near Forrest Heights

It should be noted that the frequency curve reflects only that flooding resulting from storm surge in the gulf. The Forrest Heights community is also subject to riverine flooding by Turkey Creek. The preliminary FEMA Harrison County Flood Insurance Study (FIS) dated November 2007 provides computed Turkey Creek flood profiles which appear to have been adjusted for the effects of coincident surge in Back Bay of Biloxi. Table 3.3.6-2 shows relevant discharge and stage information from the FIS for Turkey Creek at Ohio Avenue, the southern entrance to the Forrest Heights community. In comparison to the preliminary FEMA Flood Insurance Study dated November 2007, which is based on contemporary (post-Katrina) FEMA contractor hydrodynamic modeling, the ERDC frequency curve, which is based on surge alone, suggests a lower stage associated with the annual one in one hundred chance (0.01 exceedance probability) event.

Figure 3.3.6-6 shows a portion of the preliminary Harrison County Flood Insurance Rate Map in the vicinity of Forrest Heights. Low-lying peripheral areas of the neighborhood are shown in a shaded blue field as being in the 1% annual chance (‘100-yr’) regulatory floodplain, with the remainder of the community occupying a shaded Zone X field, being areas subject to shallow flooding at annual probabilities of occurrence between 0.02 (2%) and 0.01 (1%).

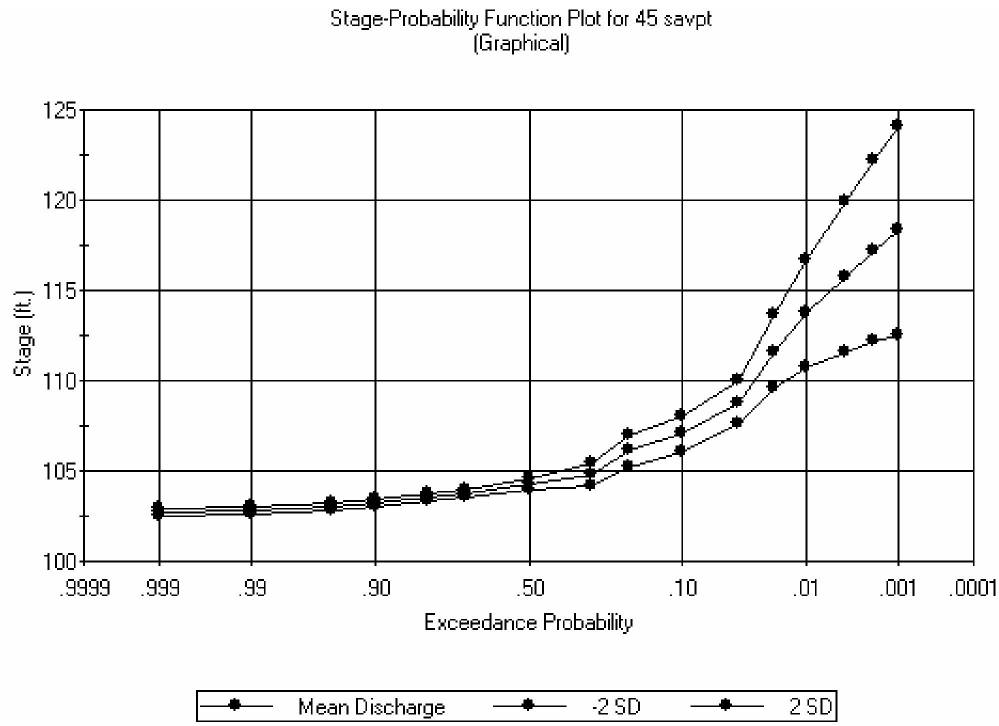


Figure 3.3.6-5. Surge-only Stage Frequency Curve, Vicinity of Forrest Heights

Table 3.3.6-1.
Surge Stage-Probability and Uncertainty

Annual Probability	Stage (Ft. NAVD88)	Standard Deviation (Feet)
0.04	8.8	0.6
0.02	11.6	1
0.01	13.7	1.5
0.002	17.2	2.5
0.001	18.3	2.9

Table 3.3.6-2.
Turkey Creek Flood Stages at Ohio Avenue, Harrison County FIS

Exceedance Probability	Discharge (cfs)	Stage (ft. NAVD '88)
0.1	2600	12
0.02	3650	14.2
0.01	5500	15.5
0.002	7950	18.3

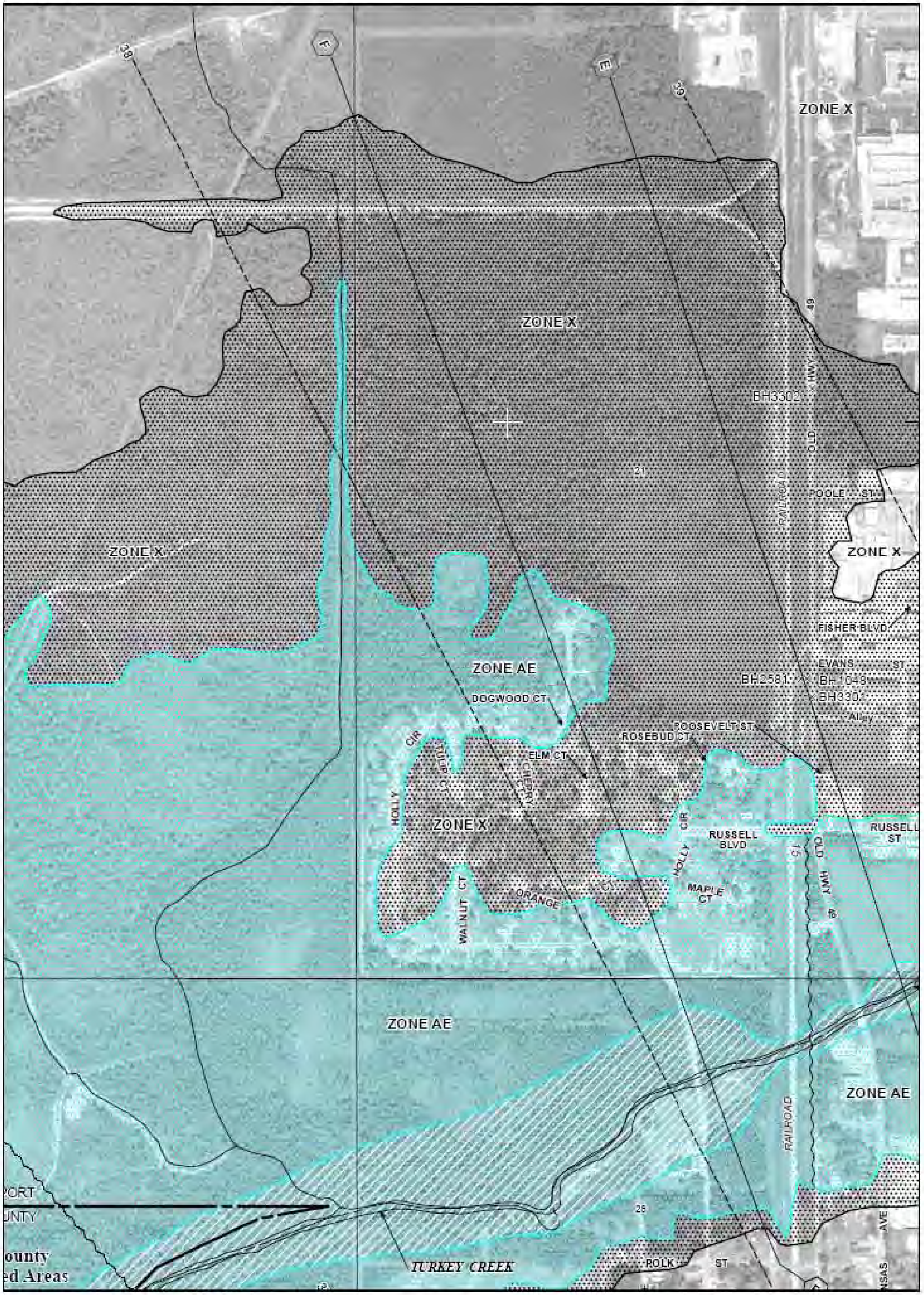


Figure 3.3.6-6. Preliminary FEMA Flood Insurance Rate Map, Vicinity of Forrest Heights

Hydraulic data was developed for use in the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) program. The HEC-FDA program uses risk-based analysis methods for evaluating flood damage and flood damage reduction alternatives. The program relies on hydrologic, hydraulic, and economic data input. Uncertainties in these data are input and used by the model for computing annual damages. Version 1.2.3b dated August 2007 was used. As described in chapter 2 of this appendix, this is a customized version of the current official release version 1.2 dated March 2000. This section describes the model's hydrologic and hydraulic input as applied to the Forrest Heights community. The Economic appendix describes the economic input and results. The Main Report describes how the model output was examined and used in the plan formulation process.

Forrest Heights is subject to both riverine and surge flooding. For this reason, a discharge-frequency curve and a stage-discharge relationship (also known as a 'rating curve') were developed for input into the HEC-FDA model. The discharge-frequency curve was computed in FDA using synthetic statistics using the 0.5-, 0.1-, and 0.01 annual exceedance probability discharges from the preliminary Harrison County FIS (see Table 3.3.6-2). The version of FDA used extends the stage frequency curve to the 0.999 and 0.0001 annual exceedance values. Uncertainty about the discharge-frequency curve was computed by the FDA program assuming an equivalent period of record. Sensitivity analysis of discharge uncertainty with respect to the equivalent period of record was conducted. Interpretation of the standard error and apparent period of record of the underlying hydrologic information used to develop the FIS discharge values versus discharge uncertainty computed by the FDA program suggested that an equivalent period of record of 20 years provided a reasonable preliminary estimate of uncertainty of discharge in the un-gaged stream. The resultant discharge-frequency curve and curves at the 5% and 95% confidence limits are shown below in Figure 3.3.6-7 and the values are shown in Table 3.3.6-3. These relationships are representative in the vicinity of Ohio Avenue.

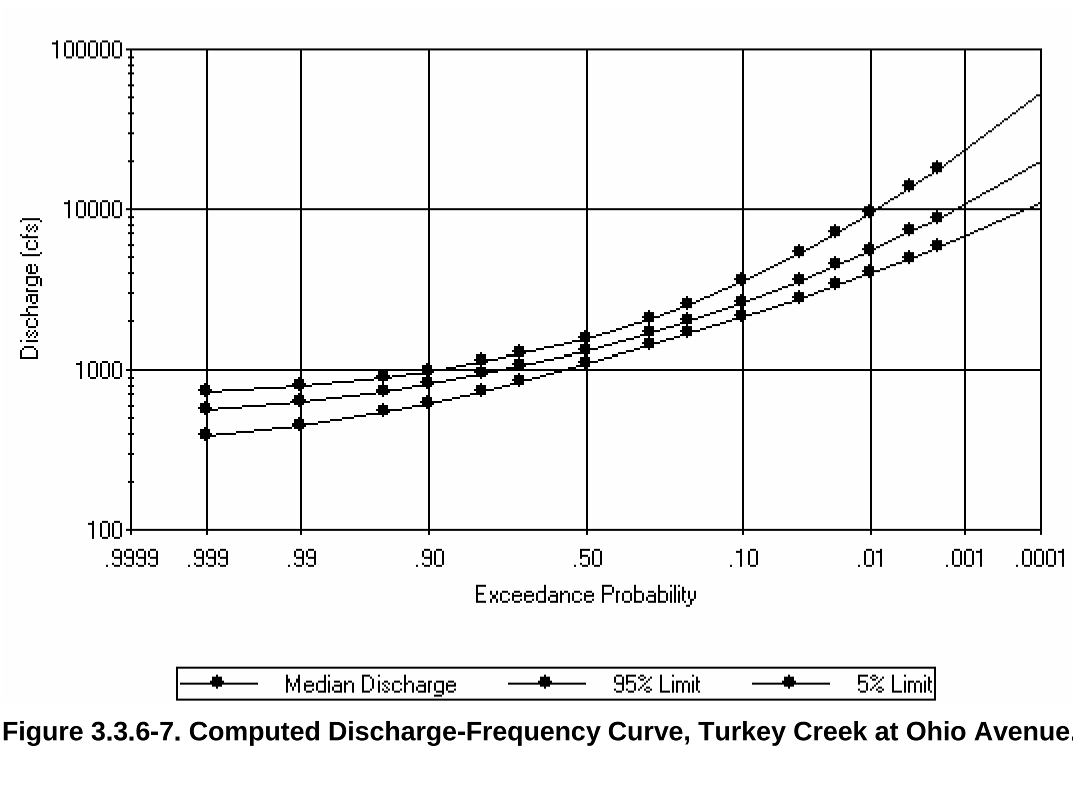


Figure 3.3.6-7. Computed Discharge-Frequency Curve, Turkey Creek at Ohio Avenue.

Table 3.3.6-3.
Discharge-Frequency, Turkey Creek at Ohio Avenue

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves Discharge (cfs)			
		95%	75%	25%	5%
0.9990	563	383	458	660	720
0.9900	634	447	525	733	795
0.9500	735	542	622	839	904
0.9000	811	614	696	918	986
0.8000	932	731	814	1,044	1,118
0.7000	1,045	840	924	1,165	1,245
0.5000	1,300	1,081	1,167	1,443	1,548
0.3000	1,678	1,412	1,511	1,882	2,051
0.2000	1,995	1,669	1,785	2,274	2,522
0.1000	2,601	2,118	2,281	3,066	3,515
0.0400	3,563	2,770	3,027	4,411	5,296
0.0200	4,449	3,330	3,684	5,716	7,104
0.0100	5,500	3,961	4,439	7,334	9,428
0.0040	7,211	4,935	5,625	10,093	13,561
0.0020	8,771	5,778	6,671	12,723	17,655
0.0001	19,704	11,042	13,464	33,224	52,792

The stage-discharge curve was developed by fitting an equation of the form $H = CQ^a$ (H = water surface elevation; Q is discharge; C and a determined by regression) through the Turkey Creek stage at cross section F as shown on the Turkey Creek Flood Profile, Plate 83P, of the preliminary FIS. The profile plate shows this location to have been adjusted for coincident probability of surge. The equation thus developed was used to extend the rating curve through a broader range of discharges than represented on the flood profiles. Uncertainty about the rating curve was assumed to be 1.5 feet at the 10-year and higher discharges based on FIS hydraulic modeling techniques and assuming a poor historic hydrologic data record (Turkey Creek is ungaged). The rating curve is shown in Figure 3.3.6-8.

3.3.6.4.1 Engineering Performance

Project engineering performance was computed using HEC-FDA. Engineering performance was computed for the existing and future without project conditions; and a variety of existing and future with-project conditions. Performance was computed with risk and uncertainty. The base year was assumed to be 2012, and the future year was assumed to be 2061 (50 year period of analysis). Scenarios were also evaluated assuming (a) existing sea level, (b) expected sea level rise, and (c) high sea level rise.

The existing condition assumes that the NRCS has reconstituted their levee around the Forest Heights community to a crest elevation of 16.5 feet. The existing and future hydrologic and hydraulic conditions are presumed to be as represented by the FIS hydrology and flood profiles with uncertainty. Typically, one would consider increasing future flood discharges to account for possible increases in runoff due to development and urbanization. However, in this case, the underlying FIS hydrologic information is dated, being circa 1976, and subsequent studies have suggested that the effective tributary drainage area in this relatively flat and undifferentiated portion of the Turkey Creek watershed is less than the 25 or so square miles attributed to the creek at the location of Forest Heights. The existing hydrology is most likely conservative, and revisions downward for an un-gaged stream seem ill-advised. Additionally, the area in question benefits from an updated and contemporary FIS, where the Turkey Creek profiles have been adjusted for coincident surge elevations, and the floodplain has been re-mapped accordingly. In the end, it seems advisable to rely on the existing FIS profiles and hydrology for conservative results.

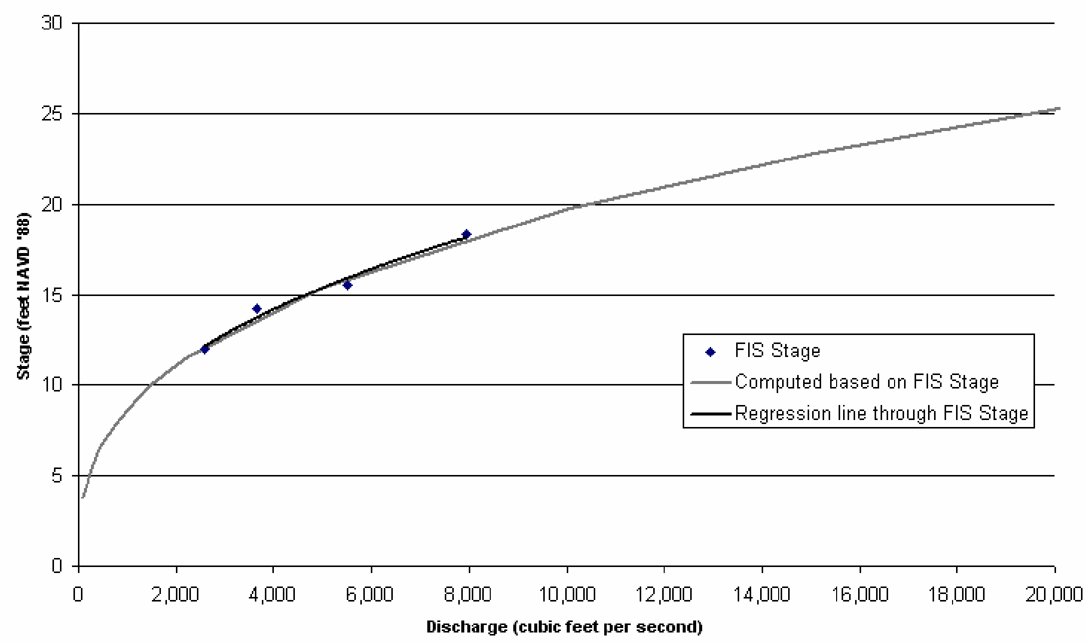
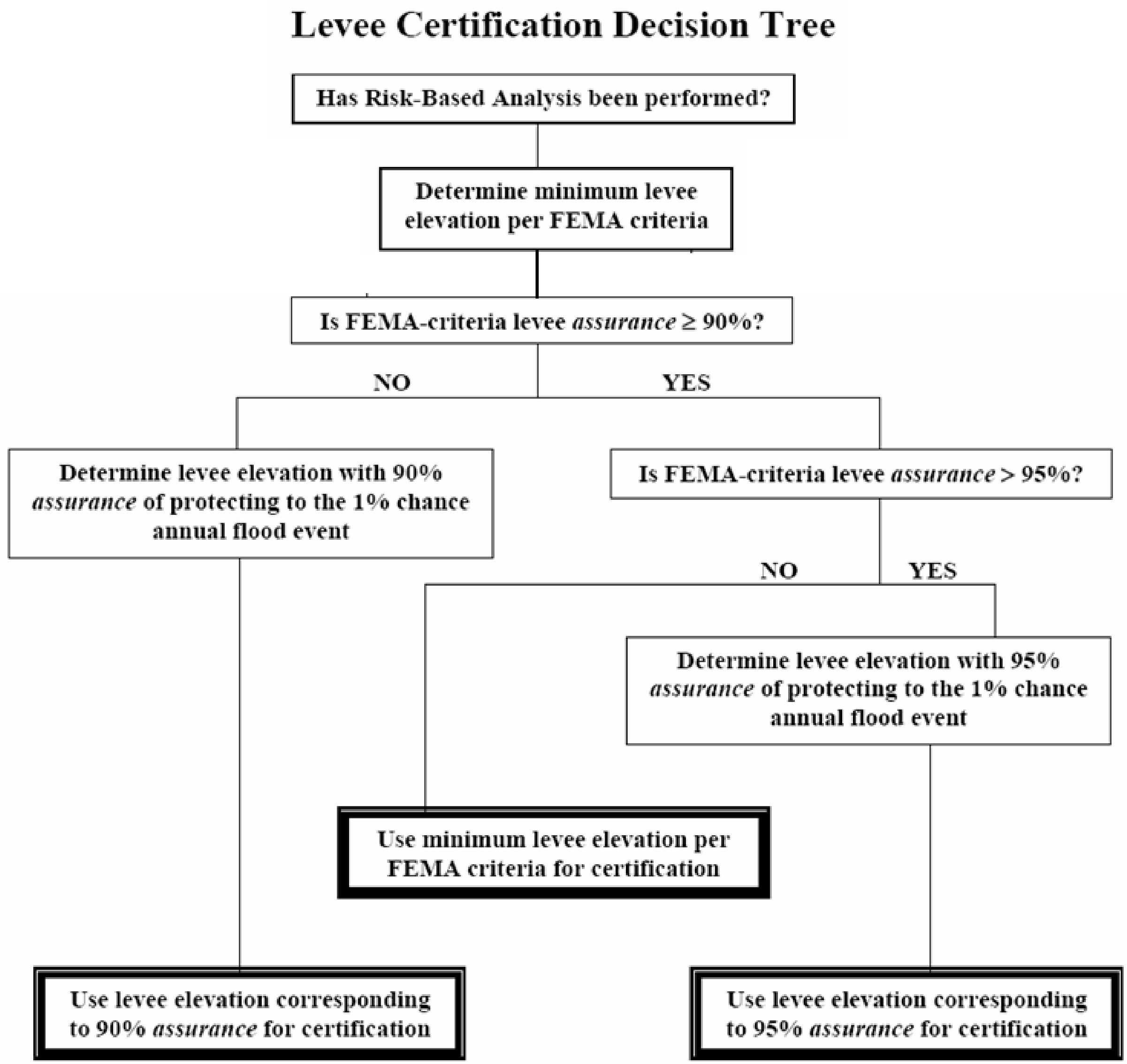


Figure 3.3.6-8. Computed Rating Curve, Turkey Creek at Ohio Avenue.

With-project conditions were evaluated for levees with crest elevations of 17 and 21 feet. The existing with-project condition assumes clearing and snagging of debris in Turkey Creek will counteract any local water surface profile impact due to flow obstruction by the levee. Future with-project conditions assume that the channel maintenance has been neglected, and thus the rating curve at Ohio Avenue is shifted upwards by 0.3 feet,

Performance was also evaluated assuming a levee built to the local Base Flood Elevation (BFE, the regulatory one in one hundred annual chance ('100-year') water surface elevation plus three feet. Historically, FEMA required levees to be built to the BFE plus three feet for certification. This condition no longer in and of itself satisfies certification criteria, which now requires that risk and uncertainty also be considered, as illustrated in Figure 3.3.6-9. This condition was evaluated for the purposes of levee certification. Assuming the BFE is defined by the FIS water surface elevation at Ohio Avenue as described on the FIS Turkey Creek Flood Profile, this elevation is 15.5 feet plus 3 feet, or elevation 18.5 feet.

Forest Heights occupies a small fringe of the floodplain, and the FDA simulations assume that when the levee is overtopped, the interior floods to the exterior flood elevation.



FEMA Criteria = 1% chance median annual flood event plus three feet of freeboard
ASSURANCE = percent chance non-exceedance given the 1% annual chance event occurs.

Figure 3.3.6-9. USACE Levee Certification Decision Tree, circa 2007

3.3.6.4.2 Performance Results

Engineering performance results as computed by HEC-FDA are shown in Figure 3.3.6-10. Base year 2012 results are the same regardless of the sea level scenario and are thus only reported once. Note that 'without project' implies that an NRCS levee built to elevation 16.5 feet (NGVD).

In this, and similar tables, the median target stage describes the probability each year of the water surface elevation exceeding the levee crest elevation according to the best estimates of the discharge frequency curve and rating curve (i.e. uncertainty is not accounted for). The expected annual exceedance probability takes discharge-frequency and stage-frequency into account when estimated the annual probability of exceeding the levee crest.

Long term risk describes the probability that the water surface would exceed the levee crest elevation in the specified time period. For example, according to these calculations, there is a 32.3 percent change that the NRCS levee (aka ‘without project) elevation would be equaled or exceeded in a 30 year period. The expected probability is used in estimating long term risk.

The conditional non-exceedance probability describes the probability, given the occurrence of some event, that the levee crest elevation would not be exceeded. For example, given the occurrence of the 1% annual water surface elevation, there is about a 55% chance that the levee would not be overtopped. Discharge and stage uncertainty is accounted for in this computation.

Figure 3.3.6-10 shows that the FEMA criteria levee at elevation 18.5 feet provides an assurance at 78.9%, which is less than 90%. With respect to the levee certification decision tree shown in Figure 3.3.6-9, and according to these analyses, the minimum certifiable levee elevation is that elevation corresponding to 90% assurance (i.e. conditional non-exceedance probability). The el. 21 feet levee is the only levee evaluated that exceeds the required assurance (92.9% base year, 91.9% future year). Sensitivity analysis shows that the 90% assurance levee crest elevation is approximately 20.2 feet NAVD ’88.

Note that crest elevation is not the sole determinant for levee certification; amongst other things, the levee must be properly constructed of sound material; the levee must be properly maintained by the owner; and interior flooding must be properly accounted for. Levee certification may be reconsidered over time as physical conditions change. For example, Figure 3.3.6-10 shows that, all else remaining the same, if sea level rises one foot in 50 years, a levee built to elevation 21 feet would provide an assurance of 87.4 percent, and the probability that it would be overtopped in a 50 year time frame would change from 11.6% to 16.9%. The point here is that, as the environment changes, the benefits of investments change.

Engineering analyses for sufficiently demonstrating a certifiable levee will be carried forward in the planning, engineering, and design phase of this project.

3.3.6.5 Option A - Elevation 17 ft NAVD88

This option consists of an earthen dike around the Forrest Heights community as shown on the following Figure 3.3.6-11, along with the levee culvert/interior pump/detention location. The earth dike will be trapezoidal in shape with a 12-foot top width with one foot vertical to three foot horizontal slopes on both sides (1H:3V). For this option the two existing roadway entrances will be ramped over the restored levee. The total length of the levee will be approximately 7900 feet.

Levees reduce the storage capacity and overbank flow conveyance of the adjacent floodplain. The reductions in overbank flow area could induce higher water levels upstream. An HECRAS model was used to evaluate the potential for induced damages and solutions. The modeling indicates that selective clearing and snagging would prevent increases in water surface elevations upstream that would occur due the placement of the levees in the floodplain.

The selective clearing and snagging would extend for approximately 4.5 miles from the mouth of Turkey Creek at Bernard Bayou to the upstream limits as shown in Figure 3.3.6-12. Selective clearing and snagging would remove obstructions such as debris dams and excessive sedimentation that hinders the flow through the Turkey Creek channel. While the selective clearing and snagging component of the plan does not eliminate flooding along Turkey Creek, the plan does reduce flood damages along the creek and at the upper end of the canals at 28th Street. The main purpose of the selective clearing and snagging is to make sure that induced damages do not occur due to the construction of the levee.

Note: FEMA Base Flood Elevation at FIS x-section F, Turkey Ck. at Ohio Avenue, is 15.5 feet. Levee height then for "FEMA plue 3 ft" plan is 15.5 ft. + 3 ft. = 18.5 feet NAVD '88. FIS: Harrison County FIS, Preliminary, dated 15 November 2007.

Forrest Heights Project Performance
by Plans and Damage Reaches by Analysis Year 2012
(Stages in ft.)

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Plan Name	Stream Name	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Without	Harrison Stream	12	Forest Heights Reach	levee	0.0064	0.0155	0.1447	0.3235	0.5424	0.9901	0.8957	0.7398	0.5493	0.3239	0.199
17-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0047	0.0123	0.1160	0.2654	0.4603	0.9950	0.9281	0.7978	0.6227	0.3894	0.251
21-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0006	0.0022	0.0218	0.0536	0.1043	1.0000	0.9976	0.9792	0.9290	0.8024	0.671
FEMA plus 3 ft	Harrison Stream	12	Forest Heights Reach	levee	0.0022	0.0063	0.0610	0.1456	0.2699	0.9995	0.9784	0.9114	0.7890	0.5725	0.411

Forest Heights Project Performance
by Plans and Damage Reaches by Analysis Year 2061
(Stages in ft.)

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Plan Name	Stream Name	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Without	Harrison Stream	12	Forest Heights Reach	levee	0.0064	0.0155	0.1447	0.3235	0.5424	0.9901	0.8957	0.7398	0.5493	0.3239	0.1998
17-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0057	0.0139	0.1305	0.2951	0.5030	0.9934	0.9127	0.7661	0.5819	0.3503	0.2202
21-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0007	0.0025	0.0244	0.0599	0.1163	1.0000	0.9968	0.9752	0.9191	0.7627	0.6454
FEMA plus 3 ft	Harrison Stream	12	Forest Heights Reach	levee	0.0025	0.0071	0.0689	0.1636	0.3004	0.9992	0.9724	0.8954	0.7611	0.5386	0.3789

Forest Heights Project Performance
by Plans and Damage Reaches by Analysis Year 2061
(Stages in ft.)

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

Expected Sea Level Rise, 1.0 feet

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Plan Name	Stream Name	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Without	Harrison Stream	12	Forest Heights Reach	levee	0.0126	0.0257	0.2289	0.4779	0.7274	0.9640	0.7926	0.5888	0.3905	0.1996	0.1118
17-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0110	0.0223	0.2019	0.4309	0.6762	0.9763	0.8255	0.6280	0.4266	0.2238	0.1278
21-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0012	0.0037	0.0365	0.0887	0.1694	0.9999	0.9922	0.9558	0.8744	0.7025	0.5488

Forest Heights Project Performance
by Plans and Damage Reaches by Analysis Year 2061
(Stages in ft.)

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

High Sea Level Rise, 1.5 feet

					Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Plan Name	Stream Name	Damage Reach Name	Damage Reach Description	Target Stage	Median	Expected	10	30	50	10%	4%	2%	1%	.4%	.2%
Without	Harrison Stream	12	Forest Heights Reach	levee	0.0167	0.0331	0.2857	0.5688	0.8141	0.9378	0.7225	0.5042	0.3139	0.1482	0.0791
17-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0150	0.0282	0.2489	0.5110	0.7609	0.9584	0.7646	0.5480	0.3490	0.1698	0.0919
21-FT Levee	Harrison Stream	12	Forest Heights Reach	levee	0.0016	0.0046	0.0449	0.1084	0.2051	0.9998	0.9882	0.9411	0.8449	0.6540	0.4961

Figure 3.3.6-10. Engineering Performance

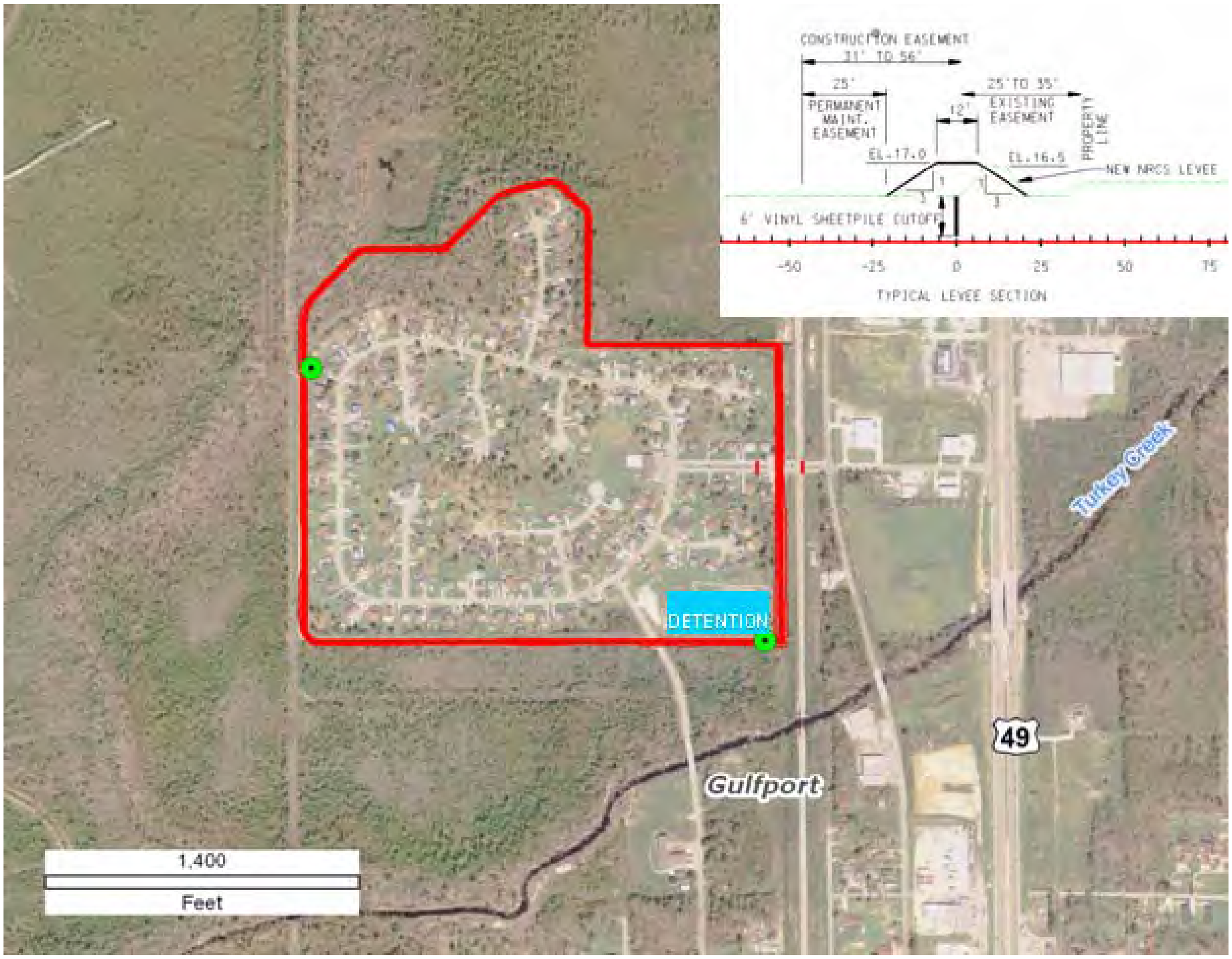


Figure 3.3.6-11. 17-ft Elevation Levee Alignment with Culvert and Pump/Detention Basin Locations

The selective clearing and snagging work will follow Stream Obstruction Removal Guidelines established by the American Fisheries Society. Only debris, snags and sediment that obstruct the flow will be removed. Material to be removed includes: 1) fine sediment accumulations that obstruct flows and alter flow patterns; 2) Debris blockages that currently or in the near future cause obstructed flow and altered flow patterns; and 3) Rooted trees that obstruct flow or need to be cleared for equipment access. Access areas that are cleared will be reestablished at the conclusion of the selective clearing and snagging activities. Some access points, however, may remain for the non-Federal sponsor to use for maintenance activity of the completed project. The existing bank alignment along the entire reach will not be changed, including the downstream reaches of Turkey Creek along the meander bends. Specific reaches to be cleared and snagged will be identified by an interdisciplinary team prior to construction.

Damage and failure by overtopping of levees could be caused by storm surges greater than the levee crest. Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.6-13 was caused by approximately 1-2 ft of overtopping crest depth.

An overtopping reach of the levee with a revetment at the detention/culvert location would be included in the levee design to prevent overtopping failure. The levee would be protected by gabions on filter cloth as shown in Figure 3.3.6-14, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

Figure 3.3.6-12. Channel Clearing and Snagging Limits

Source: ERDC, Steven Hughes

Figure 3.3.6-13. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

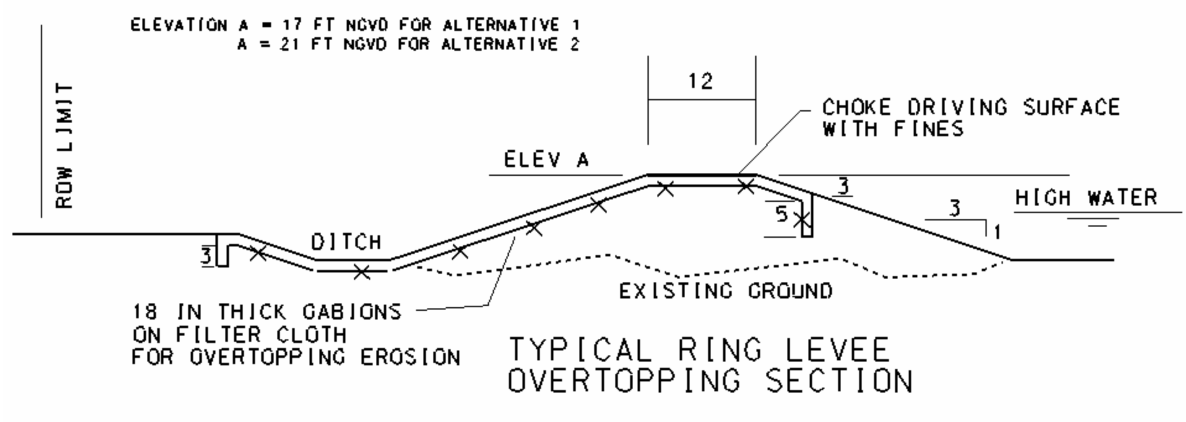


Figure 3.3.6-14. Typical Levee Overtopping Section

3.3.6.5.1 Interior Drainage

Drainage at the site is impacted by hurricanes in the gulf and by adjacent flooding from Turkey Creek. Backwater from each of these sources prevents water from running off. The existing NRCS levee at elevation 16.5 NAVD88 protects the neighborhood to some degree from these sources, but does not eliminate the flooding during times when the water outside the levee is up and there is rainfall inside the levee. This is the present condition at the site. Construction of the Corps levee will follow the footprint of the NRCS levee and provide additional protection from flooding from hurricanes and Turkey Creek. The interior flooding will be improved by adding a detention basin and pumping facility.

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown in Figure 3.3.6-9. The culverts would have flap gates on the outside ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.6-15.

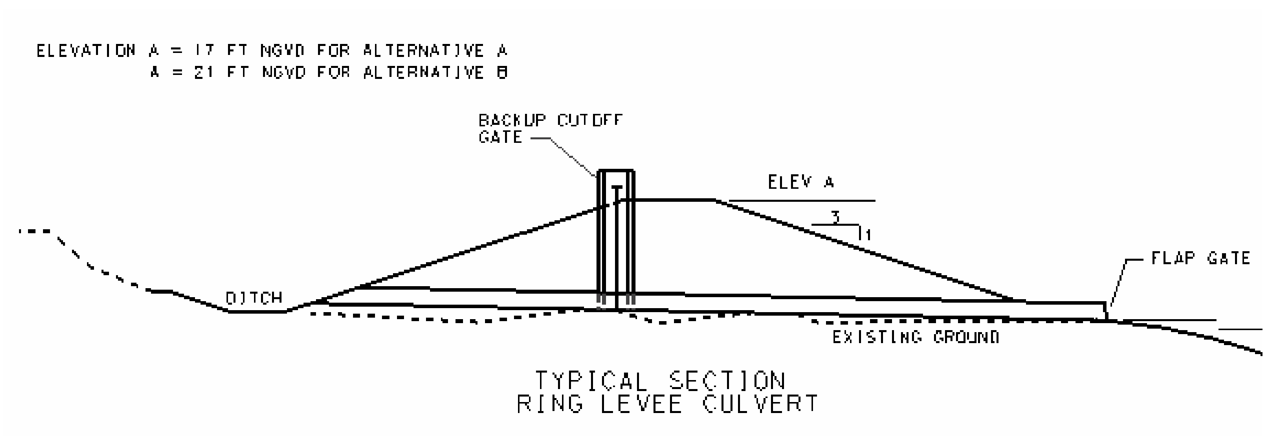


Figure 3.3.6-15. Typical Section at Culvert

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins shown below in Figure 3.3.6-16 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The curve number was determined from previous studies done for Turkey Creek.

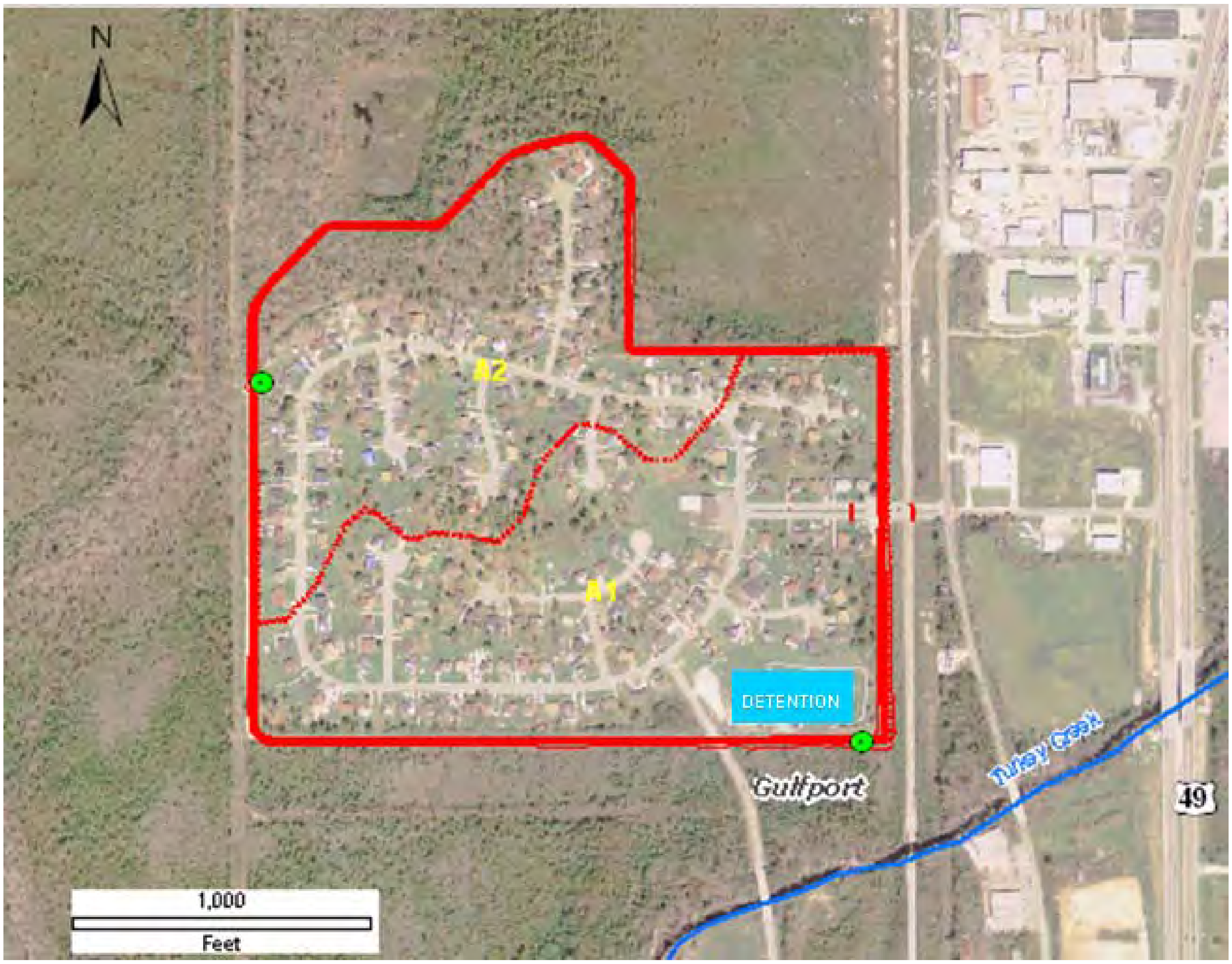


Figure 3.3.6-16 17-ft Elevation Levee Sub-basins

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 10 ft NAVD88 with a tailwater elevation of 6 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Turkey Creek or Mississippi Sound, pumping would be required to evacuate rainfall. Pump size was determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events or high water in Turkey Creek, when the culvert gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. A detention basin was added to help reduce the size of required pumps. The detention basin would have an area of approximately 3 acres but would not be excavated. The area is the lowest site in the subdivision and is presently is used for recreation facilities such as baseball and tennis. Detailed modeling of the area was not possible for this report, therefore the exact extent of the detention basin is not precisely defined. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further design during construction will refine the requirement for the appropriate detention area and pump sizes to provide protection from 100-yr rainfall.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pump could also be used to augment the flow capacity of the levee culverts.

3.3.6.5.2 Geotechnical Data

Geology: Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.6.5.3 Structural, Mechanical and Electrical

3.3.6.5.3.1 Culverts

Culverts for the project were assumed to be reinforced concrete box structures fitted with flap gates and sluice gates to provide protection from high water outside the levee. An automated system could be incorporated whereby the gates could be monitored and operated from some central location. Detailed design of these monitoring and operating systems is beyond the scope of this study.

3.3.6.5.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Forrest Heights community 2 roadway intersections would have to be accommodated. For this study it was estimated that for option 1 both roadway entrances could use ramps for crossing the restored levee. For option 2 both roadway entrances would use sliding flood gates.

3.3.6.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.6.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.6.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.6.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All gates will be operated to assure proper working order. Debris and shoaled sediment will be removed from the interior ponding area. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. An operation and maintenance (O&M) manual for the levee will be developed for the non-Federal sponsor. The O&M manual will include guidelines for maintaining the integrity of the levee over the 50-year life of the project. Regular inspections and maintenance of the levees would be performed by the non-federal sponsor and USACE personnel. Maintenance costs are included in this report.

3.3.6.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.6.7., Cost Summary. Construction costs for the various options are included in Table 3.3.6-4 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.6-5. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.6.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.6.6 Option B - Elevation 21 ft NAVD88

This option consists of an earthen levee around northern, western, and southern sides of the Forrest Heights community. Because of the height of the levee, the eastern side will be constructed with a concrete “T”-wall structure. The “T” wall will take less space than an earthen levee and encroach less into property along the alignment. The alignment of the levee is generally the same as Option A, but is shown below in Figure 3.3.6-17. Closure gates across the two access roads to the subdivision will be required. The lengths of the levee culverts will be slightly longer than those used in Option A. Other features and methods of analysis are the same.

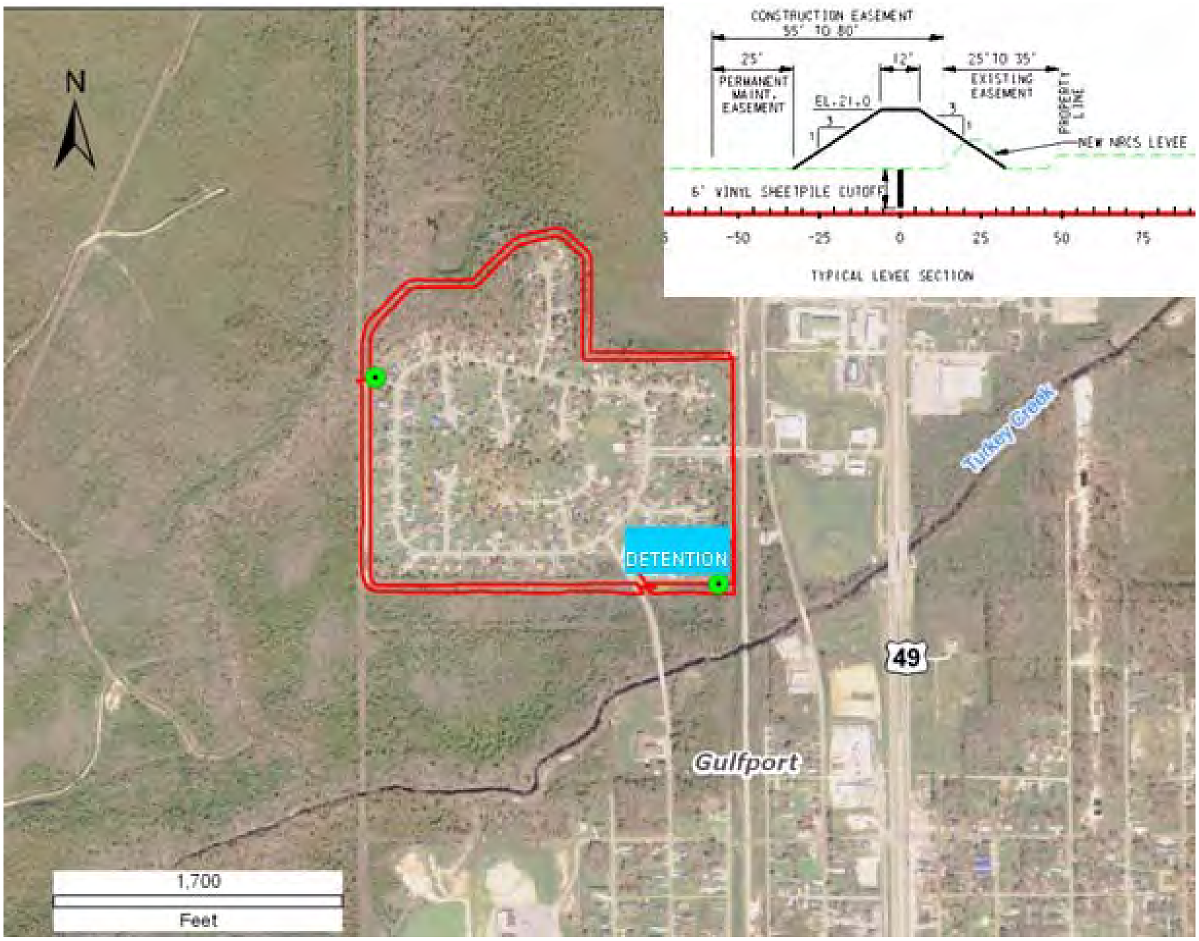


Figure 3.3.6-17. 21-ft Elevation Levee Alignment with Culvert and Detention Basin/Pump Locations

3.3.6.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.6.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.6.6.3 Structural, Mechanical and Electrical

Culvert lengths are not presented but are incorporated into the cost estimate. The “T” wall is shown below in Figures 3.3.6-18.

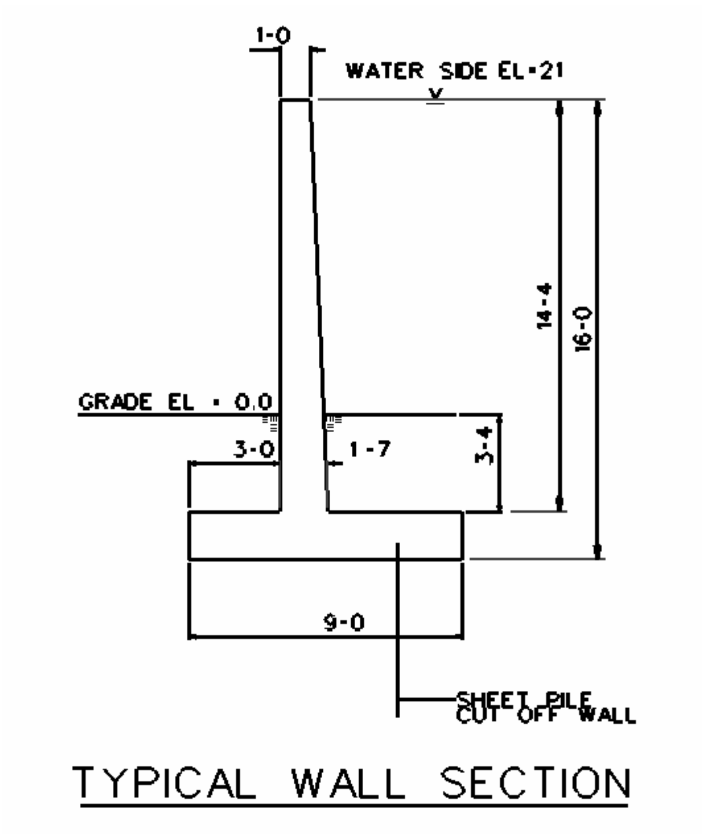


Figure 3.3.6-18. 21-ft Elevation Flood Wall Section

3.3.6.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.6.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.6.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.6.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above, with additional requirements for periodic inspection and operation of the flood gates.

3.3.6.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.6.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.6.7 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.6-4 and 3.3.6-5 below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.6-4.
Construction Cost Summary

Option	Total project cost
Option A – Elevation 17 ft NAVD88	$6,100,000
Option B – Elevation 21 ft NAVD88	$11,400,000

Table 3.3.6-5.
O & M Cost Summary

Option	O&M Cost
Option A – Elevation 17 ft NAVD88	$42,000
Option B – Elevation 21 ft NAVD88	$114,000

3.3.6.8 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Regulation ER 1105-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Regulation ER 1105-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

3.3.7 Jackson County, Ocean Springs Elevated Roadway

3.3.7.1 General

Residential and business areas along the coast in Jackson County are susceptible to storm surge damage. A damage reduction option is to raise the beach front road in Ocean Springs to elevation 11ft NAVD88 was evaluated. This option entails the raising of the Beach Road and the adjoining seawall to Elevation 11.00 from Highway 90 eastward to the Jackson County Marina. The project also provides for all utility infrastructure such as water, sewer, storm drain, gas and electric lines to be removed and reinstalled to meet the new grades. Several options of this measure were considered before selecting the final one for cost and economic comparisons. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center's (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented elsewhere in this report.

3.3.7.2 Location

The location of project in Ocean Springs is shown below in Figure 3.3.7.1.

3.3.7.3 Existing Conditions

The city of Ocean Springs lies at the eastern side of the Back Bay of Biloxi. Ground elevations over most of the residential and business areas vary between elevation 16-24 ft NAVD88, with houses along the coast at between 8-16 ft NAVD88. The 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown below in Figure 3.3.6-2.

Drainage is mostly through natural drainage ways, drowned at the mouth by Mississippi Sound.

Impacts from hurricanes can be devastating. Damage from Hurricane Katrina in August, 2005 in the Ocean Springs area are shown below in Figure 3.3.7-3 and 3.3.7-4.

3.3.7.4 Coastal and Hydraulic Data

Typical coastal data is shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.7-5. The data indicates the Katrina high water was as high as 22.5 ft NAVD88 near the Mississippi Sound.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Ocean Springs at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.7-6.

Existing Condition Stage–Frequency data for Save Point 33, just off the coast of Ocean Springs, is shown below in Figure 3.3.7-7. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

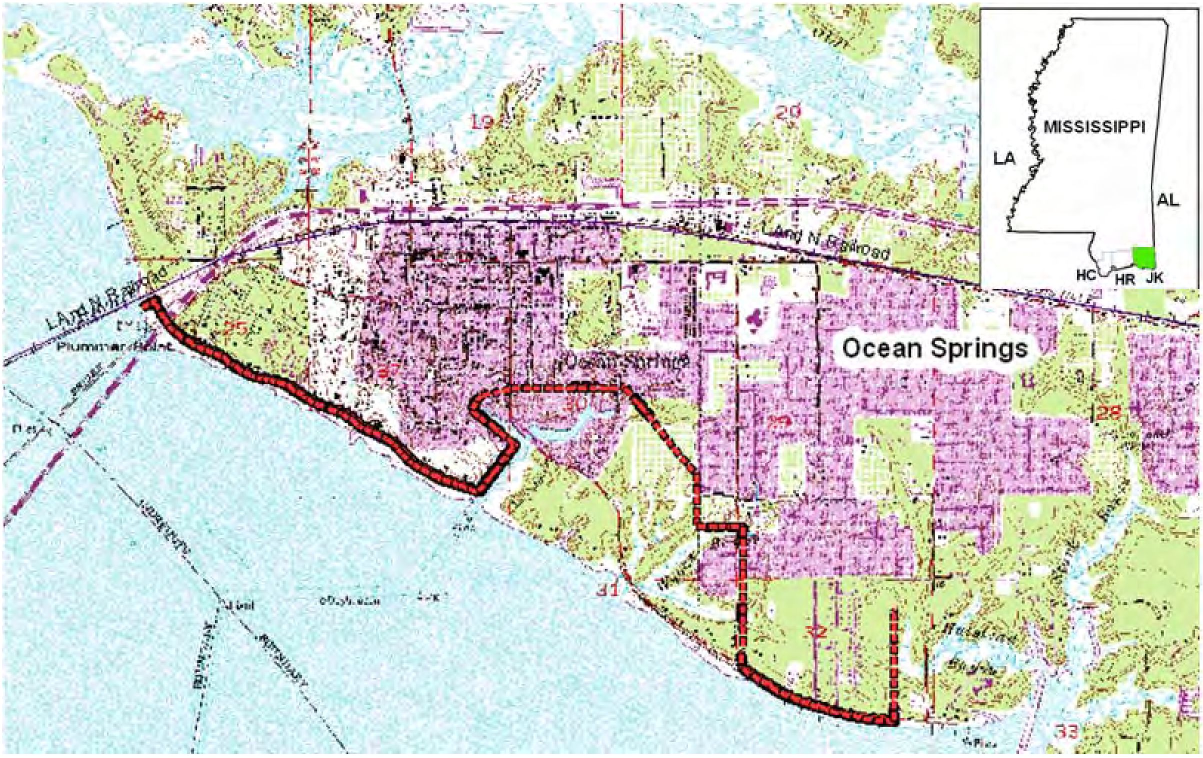


Figure 3.3.7-1. Vicinity Map, Ocean Springs

Figure 3.3.7-2. Existing Conditions

Source: http://ngs.woc.noaa.gov/storms/katrina/24834173.jpg

Figure 3.3.7-3. Hurricane Katrina Damage, Jackson County

Source:B&B Sanders, http://www.flickr.com/photo_zoom.gne?id=355219026

Figure 3.3.7-4. Hurricane Katrina Damage, Jackson County

Figure 3.3.7-5. Ground Contours and Katrina High Water, Ocean Springs

Figure 3.3.7-6. Hydrodynamic Modeling Save Points near Ocean Springs

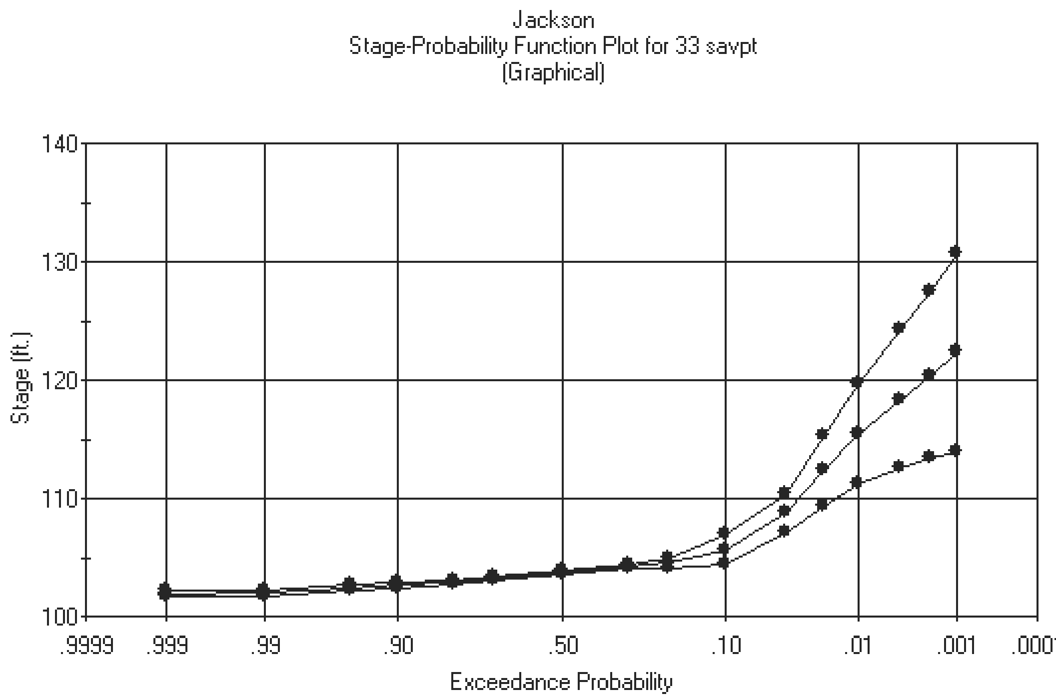


Figure 3.3.7-7. Existing Conditions at Save Point 33, near Ocean Springs

3.3.7.5 Option – Elevate Roadway to 11 ft NAVD88

This option consists of raising the beach front road to elevation 11 ft NAVD88 in Ocean Springs as shown on the following Figure 3.3.7-8, along with the internal sub-basins and levee culvert/pump locations.

Figure 3.3.7-8. Pump/Culvert/Sub-basin Site Location

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in Figure 3.3.7-9.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.7-9. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.7-10 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.3.7-10. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown above in Figure 3.3.7-11, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

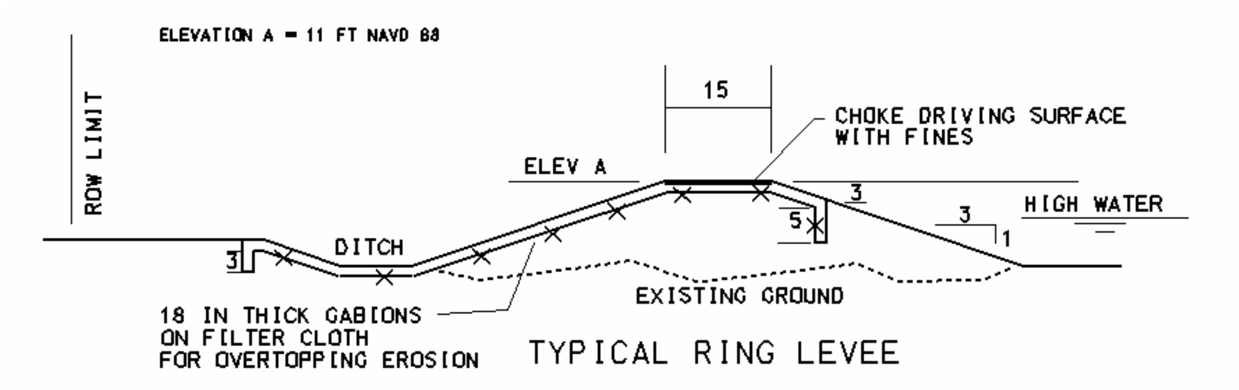


Figure 3.3.7-11. Typical Section at Ring Levee

3.3.7.5.1 Interior Drainage

Drainage on the interior of the raised highway would be collected at the highway and channeled to culverts placed at locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.7-12.

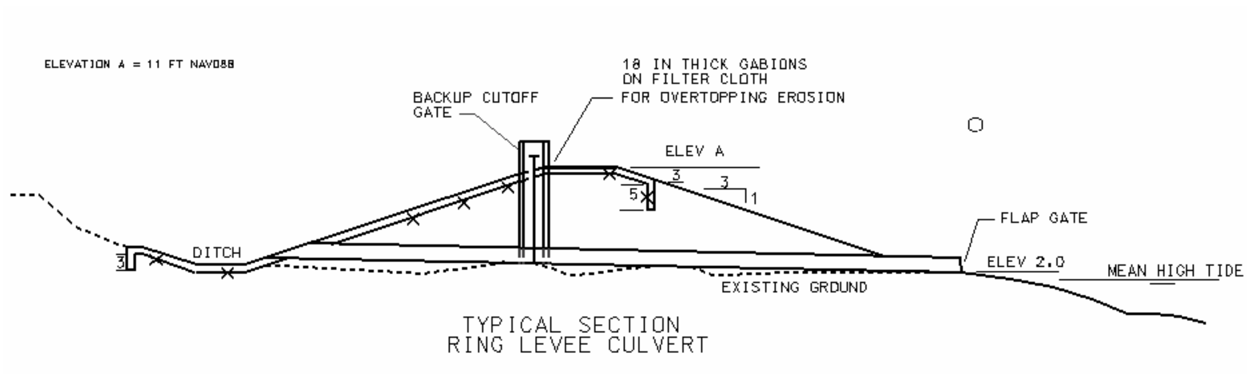


Figure 3.3.7-12. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the drainage basin into major sub-basins as shown above and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater/tailwater elevation difference was maintained at 3.0 ft or less. Drainage ditches along the toe of the highway will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.7.5.2 Geotechnical

Geology: The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The Line 3 defense elevates the roadway and accompanying seawall to elevation by extending the seaway at its present slope to grade, creating the roadway subgrade then sloping the backside to one vertical to three horizontal side slopes with a twenty five foot toe width for access and drainage. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The embankment will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface on the back side will be armored by the placement of 12 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the road. The armoring will be anchored on the back face by trenching and extend across the toe easement. All non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate ramping over the embankment where the surface elevation is near that of the crest elevation. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the roadway and the potential for erosion and instability must be considered. Final designs may require the installation of a cutoff wall within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.7.5.3 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 7 pumping facilities included in the Jackson County Raised Roadway at the elevation 11 protection level were approximately constant at 7 feet, and the corresponding flows required varied from 83,926 to 237,864 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 150 horsepower pumps, to one having six 60-inch diameter pumps each running at 150 horsepower.

3.3.7.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.7.5.5 Construction Procedures and Water Control Plan

Construction would be done by heavy construction equipment after removal of structures and relocation of utilities. Water control will be addressed by constructing drainage facilities prior to construction of the levee.

3.3.7.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Boat access gates and power plants would require this level of security.

3.3.7.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Maintenance costs are included in this report.

3.3.7.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.7.6 Cost Summary. Construction costs for the various options are included in Table 3.3.7-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.7-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.7.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years

3.3.7.6 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.7-1 and 3.3.7-2 below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.7-1.
Jackson Co Ocean Springs Elevated Roadway
Construction Cost Summary

Option	Total project cost
Option - Elevated Roadway	$67,500,000

Table 3.3.7-2.
Jackson Co Ocean Springs Elevated Roadway O & M Cost Summary

Option	O&M Cost
Option A – Elevated Roadway	$287,000

3.3.7.7 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.8 Jackson County, Ocean Springs Ring Levee

3.3.8.1 General

Several high density residential and business areas in Jackson County were identified. They are: Pascagoula/Mosspoint, Gautier, Belle Fontaine, Gulf Park Estates, and Ocean Springs. These are subject to damage from storm surges associated with hurricanes. Earthen ring levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.8.2 Location

The location of the Ocean Springs ring levee in Jackson County is shown below in Figures 3.3.8-1 and 3.3.8-2.

3.3.8.3 Existing Conditions

The city of Ocean Springs lies at the eastern side of the Back Bay of Biloxi. Ground elevations over most of the residential and business areas vary between elevation 16-24 ft NAVD88, with houses along the coast at between 8-16 ft NAVD88. The 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown below in Figure 3.3.8-3.

Drainage is mostly through natural drainage ways, drowned at the mouth by Mississippi Sound.

Impacts from hurricanes can be devastating. Damage from Hurricane Katrina in August, 2005 in the Ocean Springs area are shown below in Figure 3.3.8-4 and 3.3.8-5.

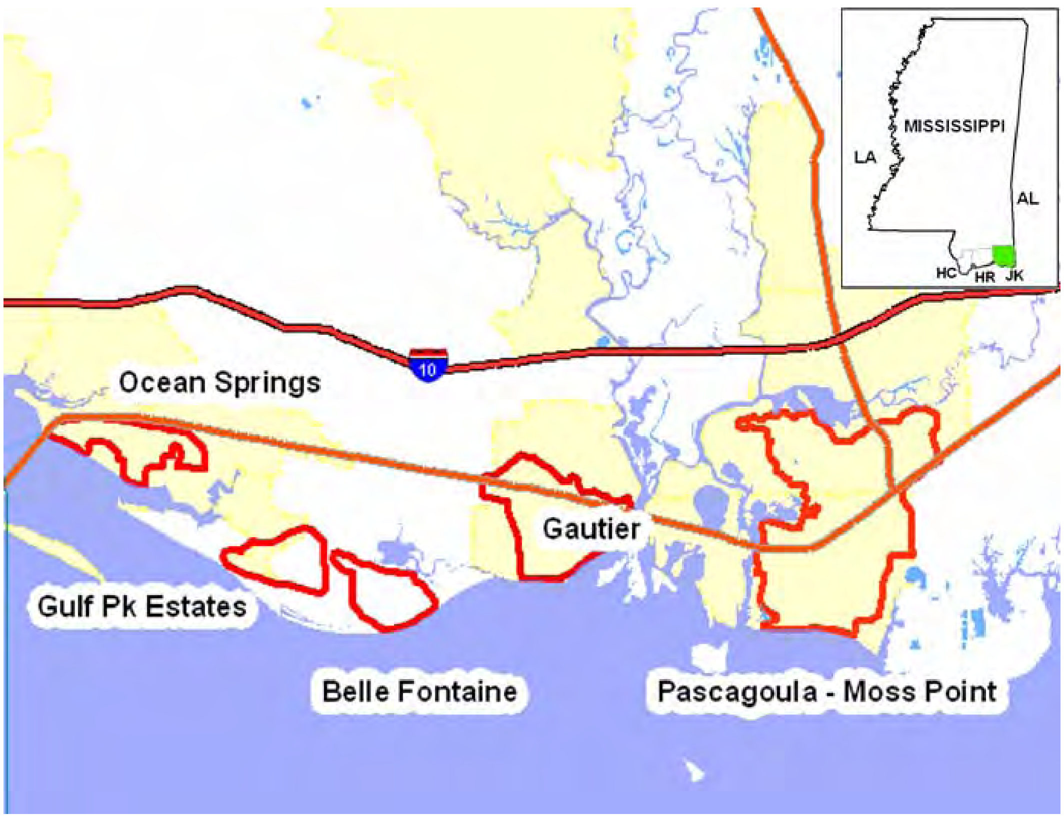


Figure 3.3.8-1. Vicinity Map Ocean Springs, MS

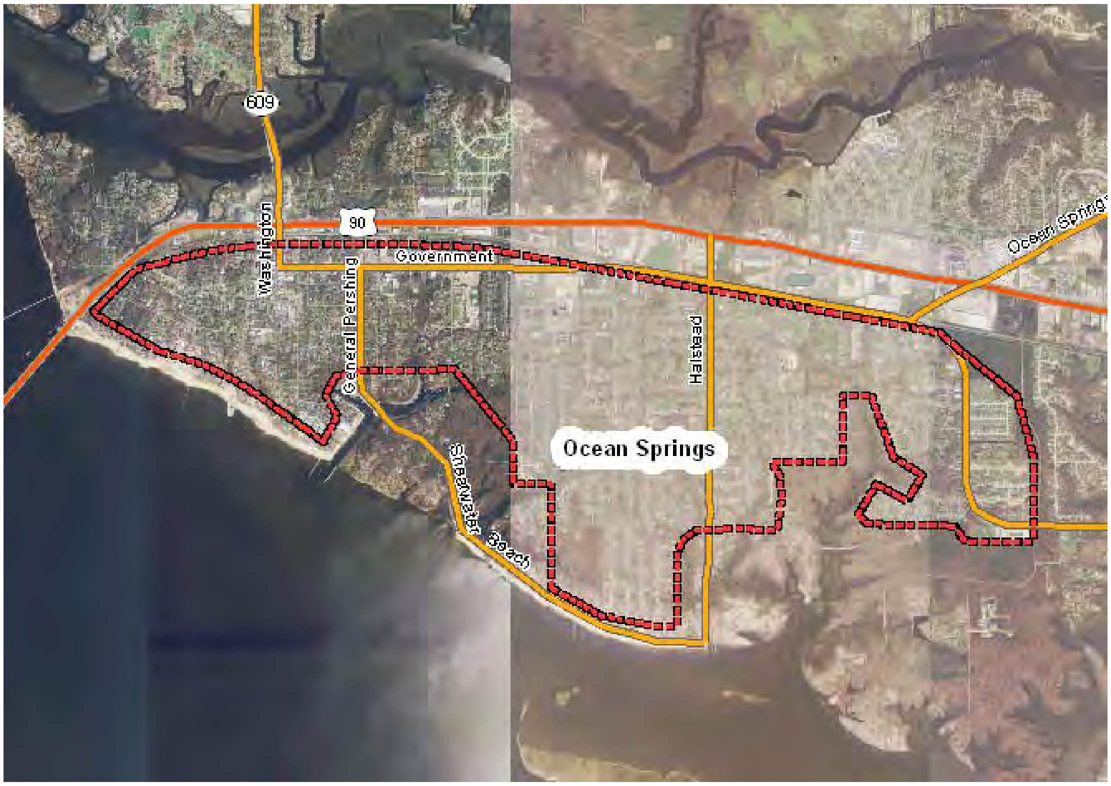


Figure 3.3.8-2. Ocean Springs Ring Levee

Figure 3.3.8-3. Existing Conditions Ocean Springs, MS

Source: http://ngs.woc.noaa.gov/storms/katrina/24834173.jpg

Figure 3.3.8-4. Hurricane Katrina Damage Ocean Springs, MS

Source: B&B Sanders, http://www.flickr.com/photo_zoom.gne?id=355219026

Figure 3.3.8-5. Hurricane Katrina Damage Ocean Springs, MS

3.3.8.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(dark green), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and Hurricane Katrina inundation limits are shown below in Figure 3.3.8-6. The data indicates the Katrina high water was as high as 22.5 ft NAVD88 near the Mississippi Sound.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Ocean Springs at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.8-7.

Existing Condition Stage –Frequency data for Save Point 33, just off the coast of Ocean Springs, is shown below in Figure 3.3.8-8. The 95% confidence limits, approximately equal to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

3.3.8.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 1752 acres around the most densely populated areas of Ocean Springs as shown on the following Figure 3.3.8-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal.

Figure 3.3.8-6. Ground Contours and Katrina High Water

Figure 3.3.8-7. Hydrodynamic Modeling Save Points near Ocean Springs

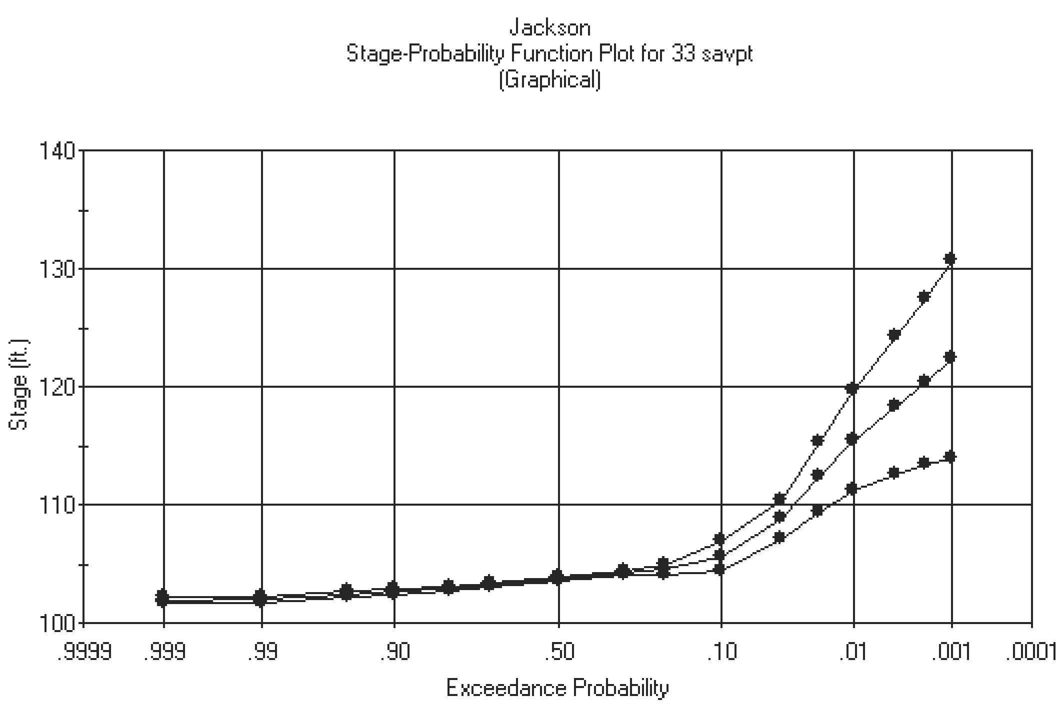


Figure 3.3.8-8. Existing Conditions at Save Point 33, near Ocean Springs

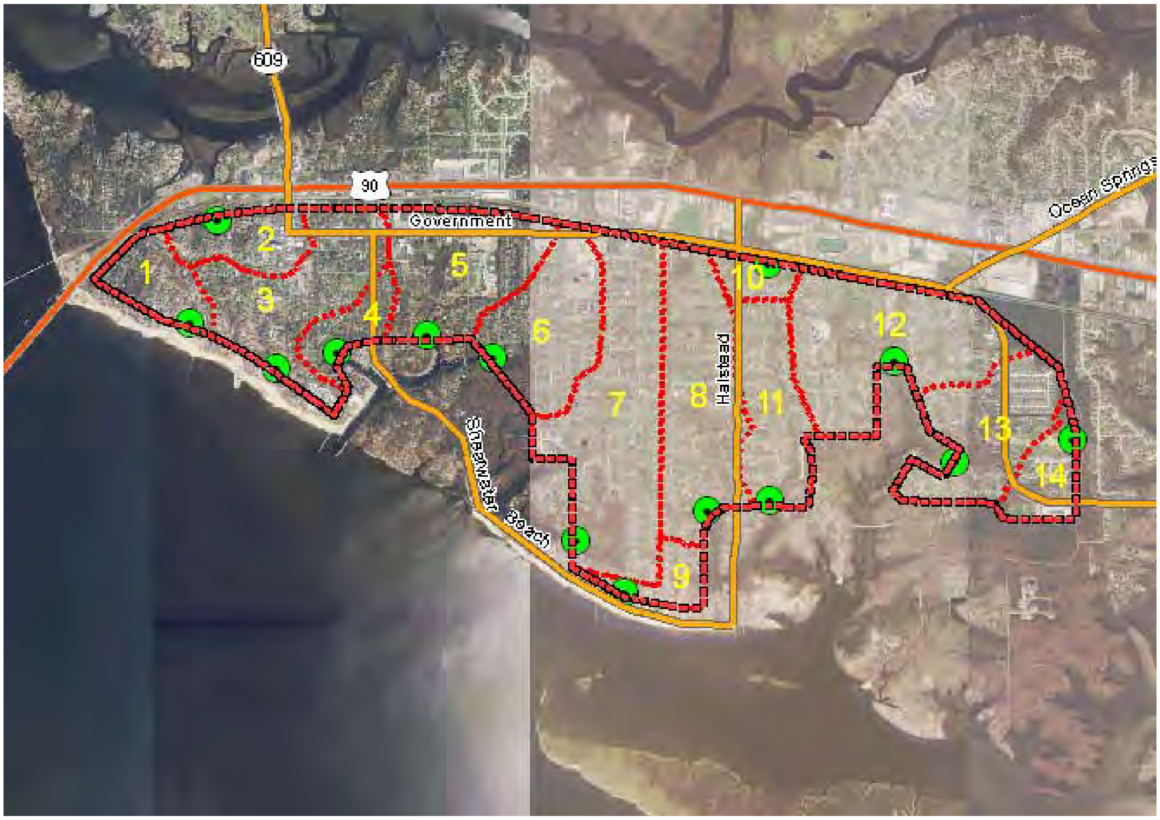


Figure 3.3.8-9. Pump/Culvert/Sub-basin Locations

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in Figure 3.3.8-10.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.8-10. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.8-11 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.3.8-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.8-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

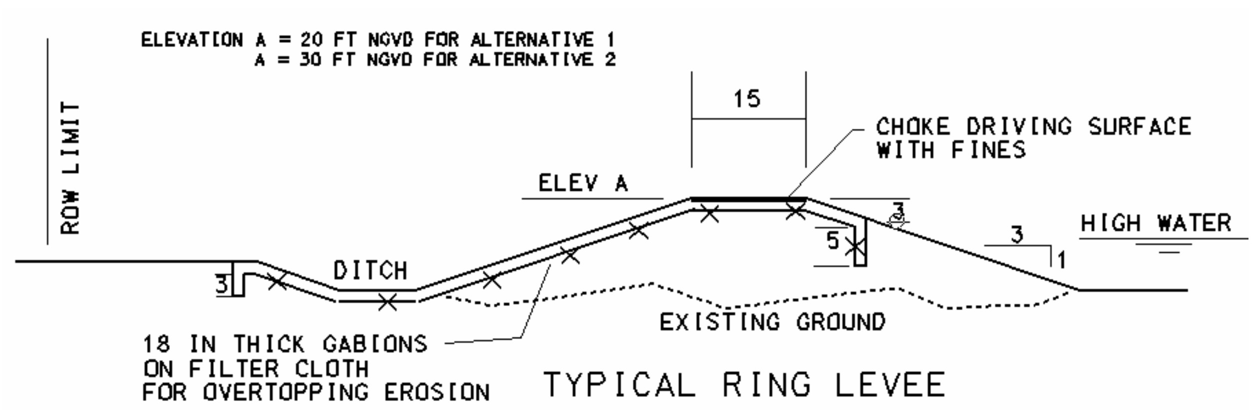


Figure 3.3.8-12. Typical Section at Ring Levee

3.3.8.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above in Figure 3.3.8-9. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.8-13.

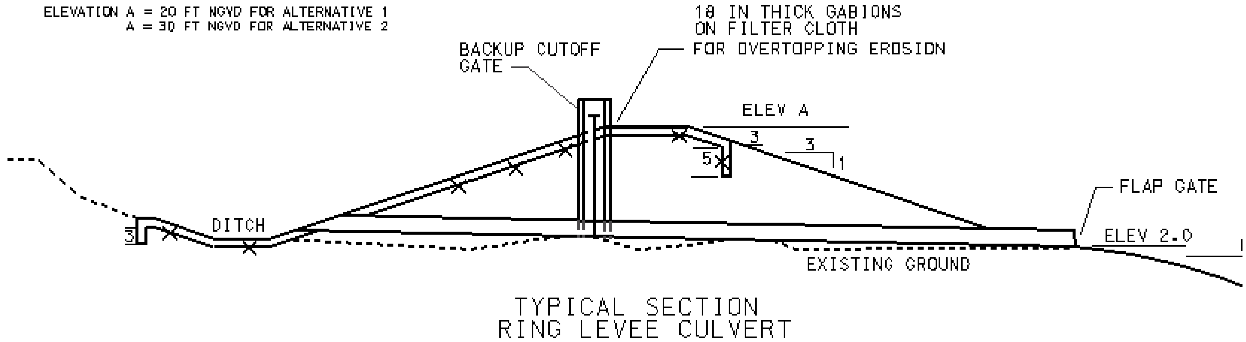


Figure 3.3.8-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown above in Figure 3.3.8-9 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types, hydrologic soil groups, and sub-basins is shown below in Figure 3.3.8-14.

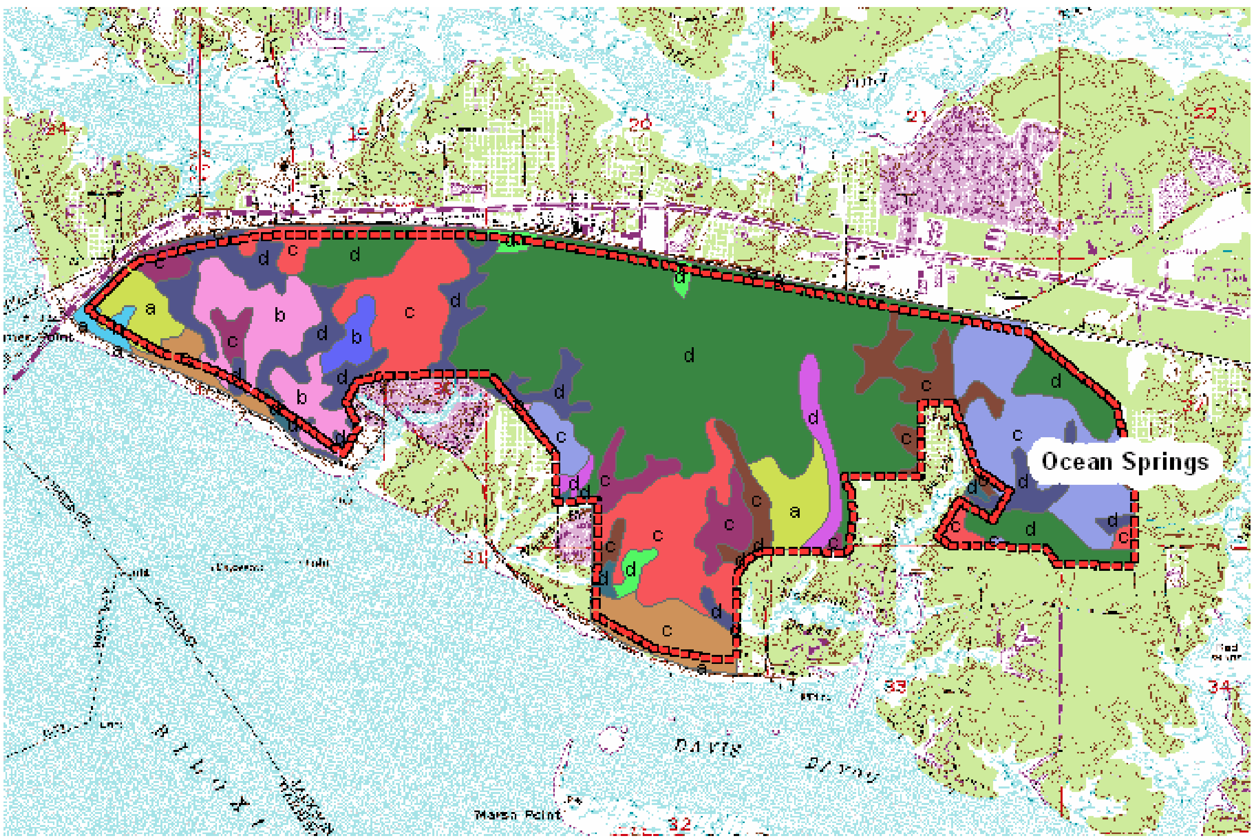


Figure 3.3.8-14. Ocean Springs Hydrologic Soil Groups

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.8.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.8.5.3 Jackson County Ring Levee. Ocean Springs. Option A - Elevation 20 ft NAVD88. Structural, Mechanical and Electrical

3.3.8.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.8.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.8.5.3.3 Pumping Stations Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.8.5.3.4 Pumping Stations Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.8.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 14 pumping facilities included in the Ocean Springs Ring Levee system for the elevation 20 protection level varied from approximately 10 to 15 feet and the corresponding flows required varied from 70,915 to 401,703 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 150 horsepower pumps, to one having four 60-inch diameter pumps each running at 560 horsepower.

3.3.8.5.3.6 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.8.5.3.7 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.8.5.3.8 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Ocean Springs area to elevation 20, 24 roadway intersections would have to be accommodated. For this study it was estimated that 6 roller gate structures and 18 swing gate structures would be required.

3.3.8.5.4 Jackson County Ring Levee. Ocean Springs. Option A - Elevation 20 ft NAVD88. HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.8.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.8.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.8.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.8.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.8.7, Cost Summary. Construction costs for the various options are included in Table 3.3.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.8-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and

coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.8.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.8.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Ocean Springs. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.8.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.8.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.8.6.3 Jackson County Ring Levee. Ocean Springs. Option B - Elevation 30 ft NAVD88. Structural, Mechanical and Electrical

These data are the same as that presented for Option A and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

Pumping Facilities. Flow and Pump Sizes. Option B. Design hydraulic heads derived for the 14 pumping facilities included in the Ocean Springs Ring Levee system for the elevation 30 protection level varied from approximately 15 to 25 feet and the corresponding flows required varied from 70,915 to 401,703 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 290 horsepower pumps, to one having four 60-inch diameter pumps each running at 1000 horsepower

Levee and Roadway/Railway Intersections. Option B. With the installation of a ring levee around the Ocean Springs area to elevation 30, 76 roadway intersections would have to be accommodated. For this study it was estimated that 6 roller gate structures and 70 swing gate structures would be required.

3.3.8.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.8.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.8.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.8.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.8.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.8.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.8.7 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.8-1 and 3.3.8-2, below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.8-1.
Jackson Co Ocean Springs Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$152,100,000
Option B – Elevation 30 ft NAVD88	$327,000,000

Table 3.3.8-2.
Jackson Co Ocean Springs Ring Levee O & M Cost Summary

Option	O&M Cost
Option A – Elevation 20 ft NAVD88	$1,414,000
Option B – Elevation 30 ft NAVD88	$2,532,000

3.3.8.8 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

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Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.9 Jackson County, Gulf Park Estates Ring Levee

3.3.9.1 General

Several high density residential and business areas in Jackson County were identified. They are : Pascagoula/Mosspoint, Gulf Park Estates, Belle Fontaine, Gulf Park Estates, and Ocean Springs. These are subject to damage from storm surges associated with hurricanes. Earthen ring levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.9.2 Location

The location of the Gulf Park Estate ring levee in Jackson County is shown below in Figure 3.3.9-1 and 3.3.9-2.

3.3.9.3 Existing Conditions

Gulf Park Estates Subdivision is located adjacent to and east of Ocean Springs. The area of study for the ring levee is bounded by Simmons Bayou on the north and the Mississippi Sound on the south. Ground elevations over most of the residential areas vary between elevation 10-20 ft NAVD88. The 4-ft(blue), 8-ft(dark green), 12-ft(light green), 16-ft(brown), and 20-ft(pink) ground contour lines and potential levee location (red) are shown below in Figure 3.3.9-3.

Drainage of the residential area is mostly to the north to Simmons Bayou. Only a small part of the area drains to Mississippi Sound.

Impacts from hurricanes are devastating to the area. Recent damage from Hurricane Katrina in August, 2005 the Gulf Park Estates area are shown below in Figures 3.3.9-4 and 3.3.9-5. Many homes are still un-repaired, pending settlement of insurance claims.

Figure 3.3.9-1. Vicinity Map Gulf Park Estates

Figure 3.3.9-2. Gulf Park Estates Ring Levee

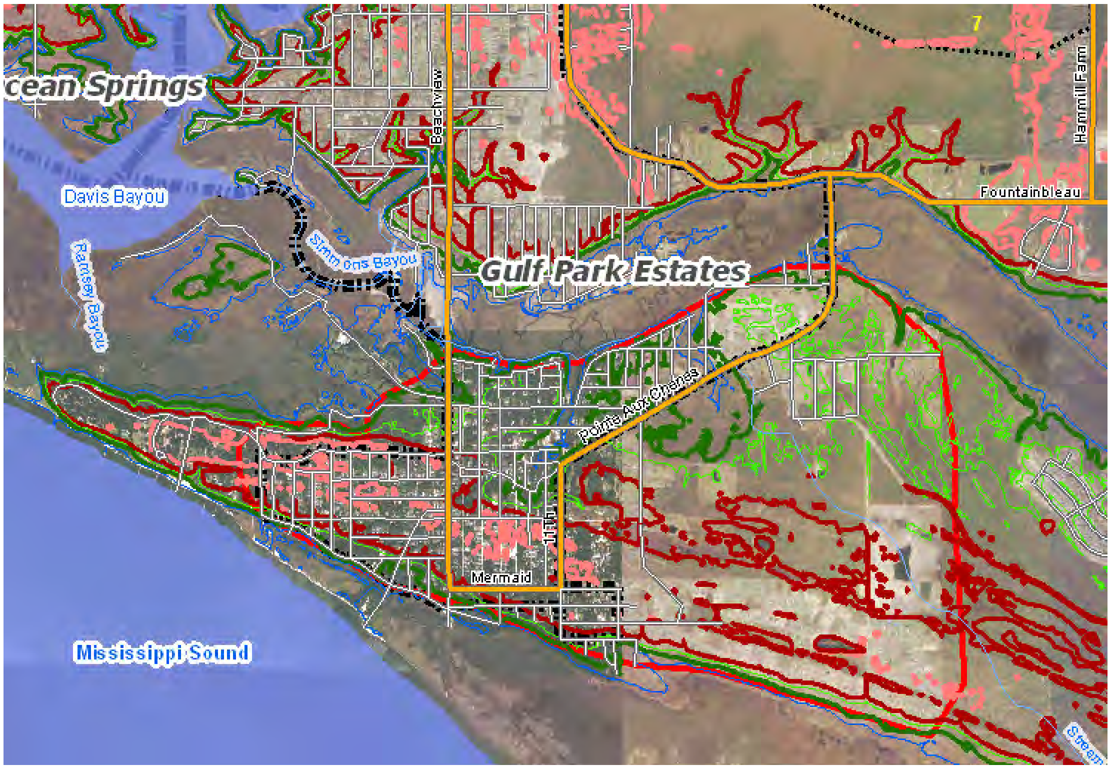

Figure 3.3.9-3. Existing Conditions Gulf Park Estates

Source: http://ngs.woc.noaa.gov/storms/katrina/24333182.jpg

Figure 3.3.9-4. Hurricane Katrina Damage Gulf Park Estates

SourceSpartan1's Photos: http://www.flickr.com/photo_zoom.gne?id=362158993&size=m&context=photostream

Figure 3.3.9-5. Hurricane Katrina Damage Gulf Park Estates, MS

3.3.8.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines major streets are shown below in Figure 3.3.9-6. The data indicates the Katrina high water was as high as 21 ft NAVD88 at the Mississippi Sound.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Gulf Park Estates at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.9-7.

Existing Condition Stage –Frequency data for Save Point 1, just off the coast of Gulf Park Estates, is shown below in Figure 3.3.9-8. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

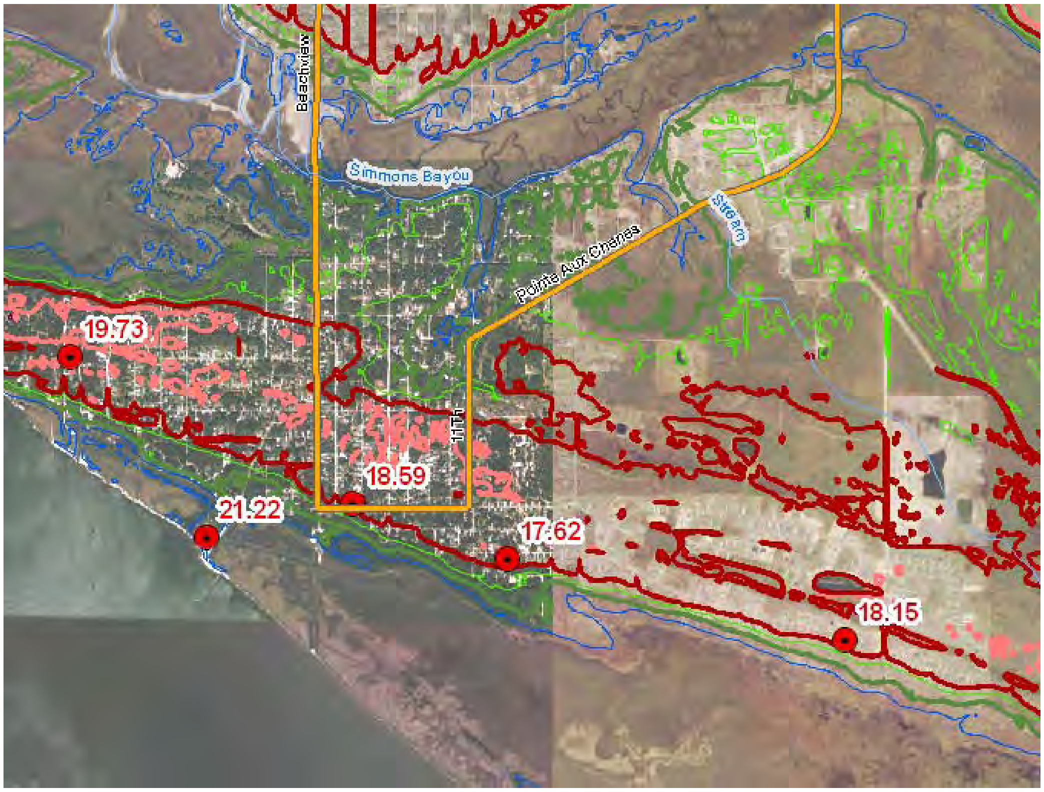

Figure 3.3.9-6. Ground Contours and Katrina High Water

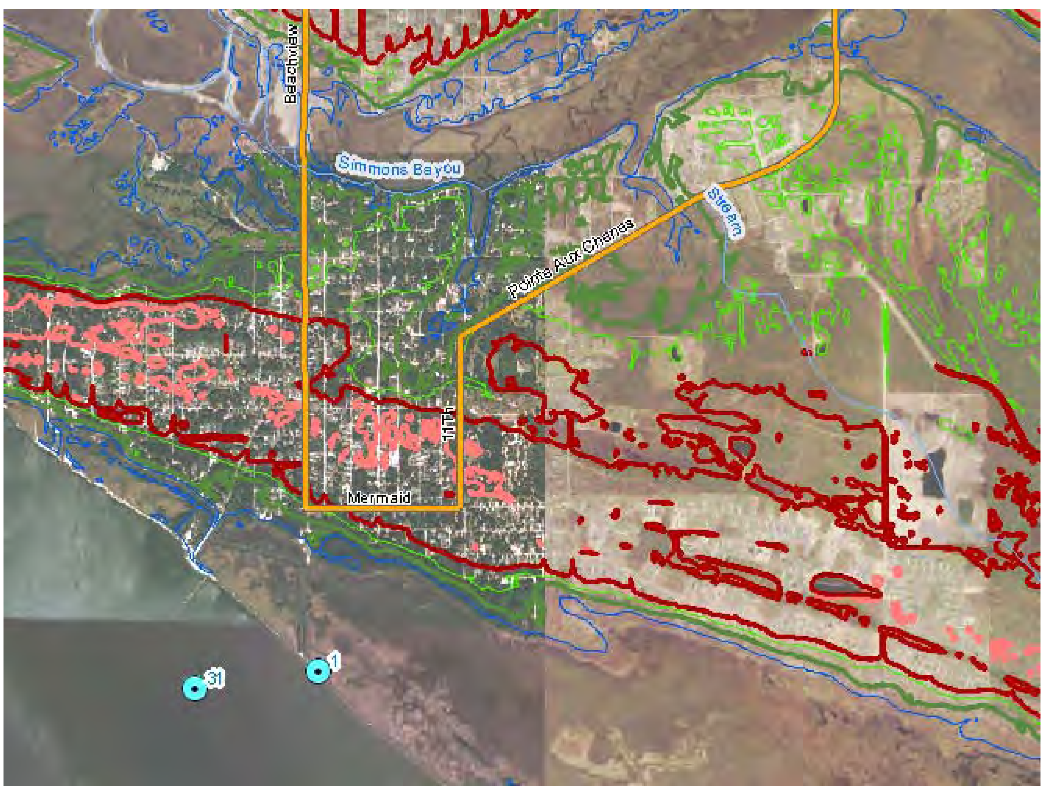

Figure 3.3.9-7. Hydrodynamic Modeling Save Points near Gulf Park Estates

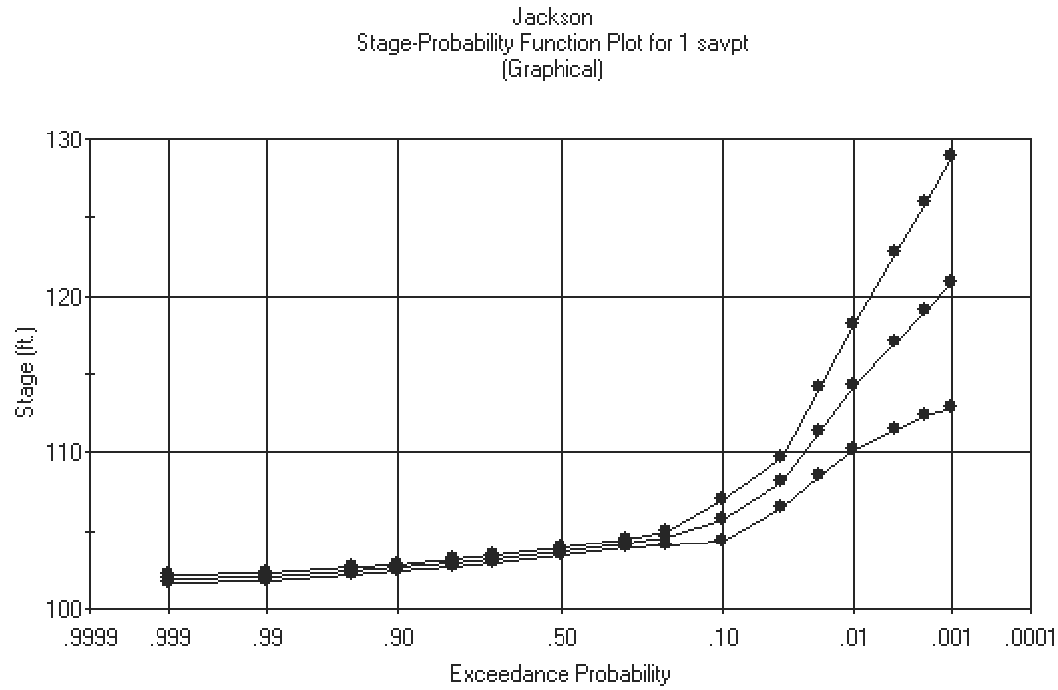

Figure 3.3.9-8. Existing Conditions at Save Point 1, near Gulf Park Estates

3.3.9.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 1473 acres around the most densely populated areas of Gulf Park Estates as shown on the following Figure 3.3.9-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal.

Damage and failure by overtopping of levees could be caused by storm surges greater than the levee crest as shown on Figure 3.3.9-10.

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.9-11 was caused by approximately 1-2 ft of overtopping crest depth.

Figure 3.3.9-9. Pump/Culvert/Sub-basin Locations

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.9-10. North Sea, Germany, March 1976

Source: ERDC, Steven Hughes

Figure 3.3.9-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.9-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

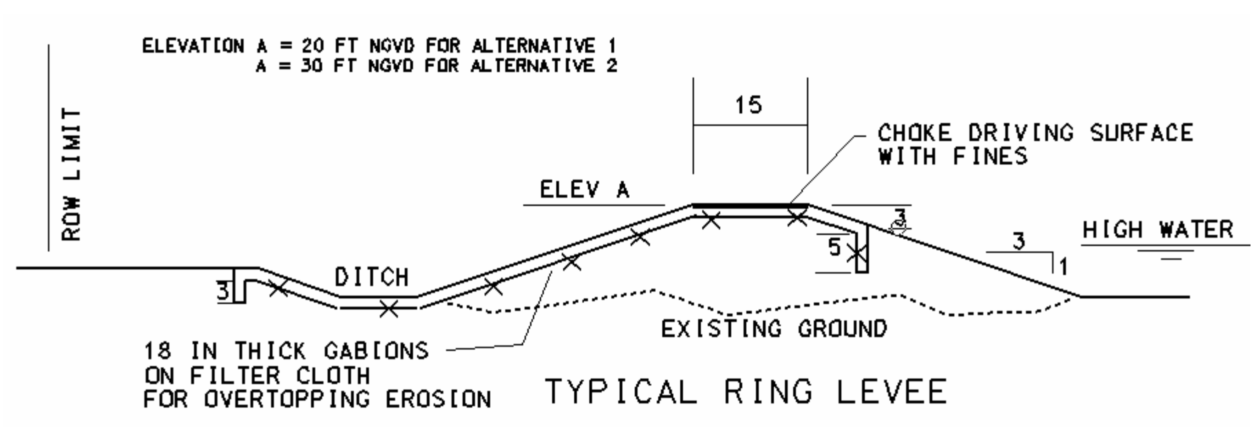

Figure 3.3.9-12. Typical Section at Ring Levee

3.3.9.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above in Figure 3.3.9-9. The culverts would have tidal gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at the upstream end at every culvert in the levee for manual control in the event the tidal gate malfunctions. A typical section is shown is shown below in Figure 3.3.9-13.

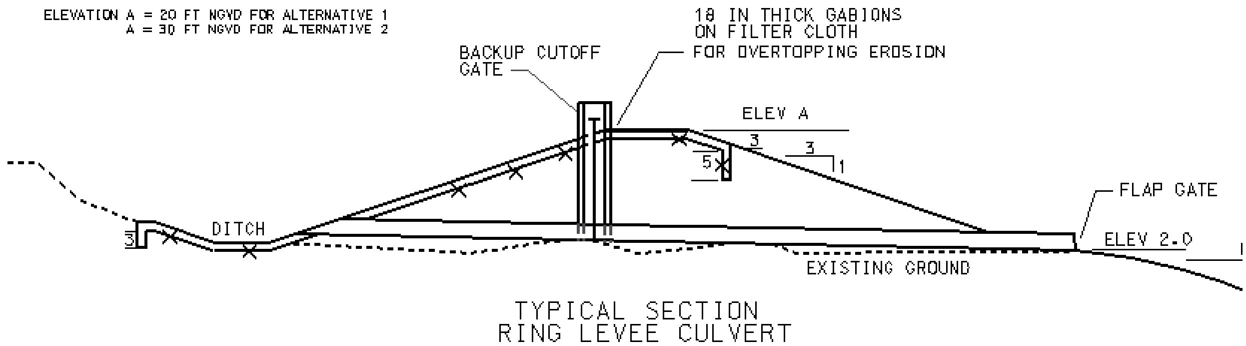

Figure 3.3.9-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown in Figure 3.3.9-9 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types, hydrologic soil groups, and sub-basins is shown in Figure 3.3.9-14.

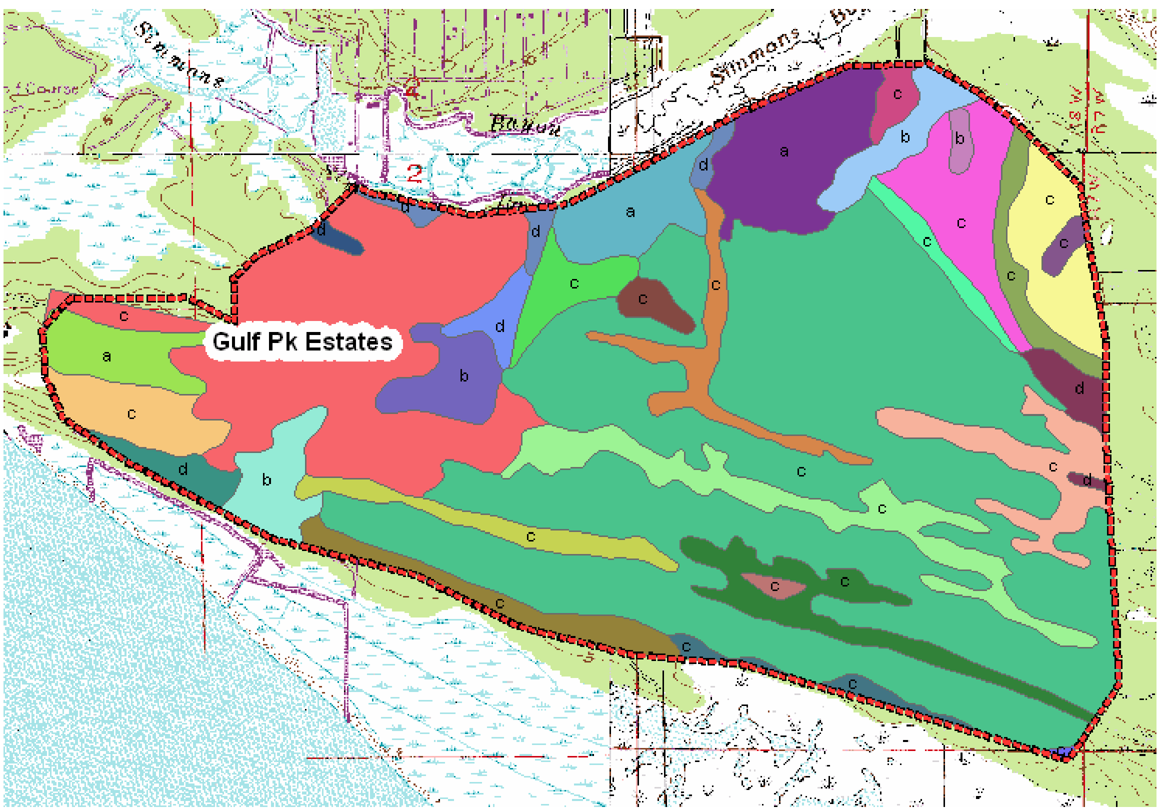

Figure 3.3.9-14. Gulf Park Estates Hydrologic Soil Groups

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission.

Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.9.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.9.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts and pumping facilities. The sites are shown above in Figure 3.3.9-9.

3.3.9.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.9.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.9.5.3.3 Pumping Stations Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.9.5.3.4 Pumping Stations Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.9.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 8 pumping facilities included in the Gulf Park Estates Ring Levee system for the elevation 20 protection level varied from approximately 10 to 15 feet and the corresponding flows required varied from 32,316 to 333,481 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 154 horsepower pump, to one having four 60-inch diameter pumps each running at 560 horsepower.

3.3.9.5.3.6 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.9.5.3.7 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.9.5.3.8 Levee and Roadway/Railway Intersections

With the installation of a ring levee around Gulf Park Estates to elevation 20, 20 roadway intersections would have to be accommodated. For this study it was estimated that 2 roller gate structures and 18 swing gate structures would be required.

3.3.9.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.9.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.9.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.9.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.9.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.9.10., Cost Summary. Construction costs for the various options are included in Table 3.3.9-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.9-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.9.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.9.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Gulf Park Estates. The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.9.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.9.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.9.6.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

3.3.9.6.3.1 Pumping Facilities. Flow and Pump Sizes

Design hydraulic heads derived for the 8 pumping facilities included in the Gulf Park Estates Ring Levee system for the elevation 30 protection level varied from approximately 20 to 25 feet and the corresponding flows required varied from 32,315 to 333,482 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 300 horsepower pump, to one having four 60-inch diameter pumps each running at 1000 horsepower.

3.3.9.6.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around Gulf Park Estates to elevation 30, 13 roadway intersections would have to be accommodated. For this study it was estimated that all 13 would require swing gate structures.

3.3.9.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.9.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.9.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.9.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.9.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.9.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.9.7 Option C – Alternate Alignment, Elevation 20 ft NAVD88

This option consists of an earthen levee at elevation 20 ft NAVD88 enclosing an area of 1355 acres around the most populated areas of Gulf Park Estates in an alignment slightly different from the alignment for Options A and B. The alignment of the levee is shown in Figure 3.3.9-15 below, which also shows the variation in the drainage sub-basins and the locations of the pumps and culverts.

Figure 3.3.9-15. Alternative Alignment Pump/Culvert/Sub-basin Locations

3.3.9.7.1 Interior Drainage

Interior drainage flows are similar to those computed for Option A, above. However, the appropriate ditches, culverts and pumps were re-sized by either adjusting the previously computed flows by the ratio of the change in areas of the sub-basins to get the revised flows, or by computing flows by TR-55 methods.

3.3.9.7.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option C are the same as for Option A, above.

3.3.9.7.3 Structural, Mechanical and Electrical

The primary difference between the description of this option and preceding description of Option A is a slight alteration in the routing of the levee resulting in slight alteration to the required pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option C is presented below.

3.3.9.7.3.1 Pumping Facilities Flow and Pump Sizes

Design hydraulic heads derived for the 9 pumping facilities included in the Gulf Park Estates Ring Levee system for the optional alignment at elevation 20 protection level varied from approximately 5 to 20 feet and the corresponding flows required varied from 31,544 to 333,387 gallons per minute. The plants thus derived varied in size from a plant having two 26-inch diameter, 150 horsepower pumps, to one having eight 42-inch diameter pumps each running at 300 horsepower.

3.3.9.7.3.2 Levee and Roadway/Railway Intersections.

With the installation of a ring levee around Gulf Park Estates to elevation 20, 18 roadway gates for intersections would have to be accommodated. For this study it was estimated that 14 would require swing gate structures with the remaining 4 requiring roller gates of varying heights.

3.3.9.7.4 HTRW

The HTRW paragraphs for Option C are the same as for Option A, above.

3.3.9.7.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option C are the same as for Option A, above.

3.3.9.7.6 Project Security

The Project Security paragraphs for Option C are the same as for Option A, above.

3.3.9.7.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option C are the same as for Option A, above.

3.3.9.7.8 Cost Estimate

The Cost Estimate paragraphs for Option C are the same as for Option A, above.

3.3.9.7.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option C are the same as for Option A, above.

3.3.9.8 Option D – Alternate Alignment, Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Gulf Park Estates. The alignment of the levee is the same as Option C, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option C is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.9.8.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option C, above, except that the culvert lengths through the levees would be longer.

3.3.9.8.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option D are the same as for Option A, above.

3.3.9.8.3 Structural, Mechanical and Electrical

The primary difference between the description of this option and preceding description of Option A, besides the height of the levee, is a slight variation in the levee alignment, resulting in changes to the pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option D is presented below.

3.3.9.8.3.1 Pumping Facilities. Flow and Pump Sizes.

Design hydraulic heads derived for the 8 pumping facilities included in the Gulf Park Estates Ring Levee system for the elevation 30 protection level varied from approximately 15 to 30 feet and the corresponding flows required varied from 31,544 to 333,387 gallons per minute. The plants thus derived varied in size from a plant having two 26-inch diameter, 200 horsepower pump, to one having eight 42-inch diameter pumps each running at 500 horsepower.

3.3.9.8.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around Gulf Park Estates to elevation 30, 15 roadway intersections would have to be accommodated. For this study it was estimated that all 15 would require 30 swing gate structures.

3.3.9.8.4 HTRW

The HTRW paragraphs for Option D are the same as for Option A, above.

3.3.9.8.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option D are the same as for Option A, above.

3.3.9.8.6 Project Security

The Project Security paragraphs for Option D are the same as for Option A, above.

3.3.9.8.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option D are the same as for Option A, above.

3.3.9.8.8 Cost Estimate

The Cost Estimate paragraphs for Option D are the same as for Option A, above.

3.3.9.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option D are the same as for Option A, above.

3.3.9.10 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.9-1 and 3.3.9-2, below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.9-1.
Jackson Co Gulf Park Estates Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$149,200,000
Option B – Elevation 30 ft NAVD88	$220,600,000
Option C – Elevation 20 ft NAVD88	$158,900,000
Option D – Elevation 30 ft NAVD88	$208,700,000

Table 3.3.9-2.
Jackson Co Gulf Park Estates Ring Levee O & M Cost Summary

Option	O&M Cost
Option A – Elevation 20 ft NAVD88	$1,499,000
Option B – Elevation 30 ft NAVD88	$2,404,000
Option C – Elevation 20 ft NAVD88	$1,295,000
Option D – Elevation 30 ft NAVD88	$1,906,000

3.3.9.11 References

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USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

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3.3.10 Jackson County, Belle Fontaine Ring Levee

3.3.10.1 General

Several high density residential and business areas in Jackson County were identified. They are : Pascagoula/Mosspoint, Gautier, Belle Fontaine, Gulf Park Estates, and Ocean Springs. These are subject to damage from storm surges associated with hurricanes. Earthen ring levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.10.2 Location

The location of the Belle Fontaine ring levee in Jackson County is shown below in Figures 3.3.10-1 and 3.3.10-2. Two alignments are shown on Figure 3.3.10-2. These are evaluated separately.

3.3.10.3 Existing Conditions

The subdivision of Belle Fontaine is located just west of Gautier along the gulf coast on Mississippi Sound. The northeastern part of the subdivision is near elevation 10-14 ft NAVD88 and very flat. Ground elevations over the southwestern part of the area vary between elevation 16-20 ft NAVD88. The 4-ft(blue), 8-ft (dark green), 12-ft(light green), 16-ft(brown), and 20-ft(pink) ground contour lines and levee limits (red) are shown below in Figure 3.3.10-3.

The area is drained by very small natural and some improved channels. These channels drain to the north to Graveline Bayou, and to Mississippi Sound.

Drainage from ordinary rainfall is hindered on occasions when the gulf is high, but impacts from hurricanes are devastating. Damage from Hurricane Katrina in August, 2005 in the Belle Fontaine area are shown below in Figures 3.3.10-4 and Figure 3.3.10-5. Many homes are still un-repaired, pending settlement of insurance claims.

Figure 3.3.10-1. Vicinity Map, Jackson County

Figure 3.3.10-2. Belle Fontaine Ring Levee

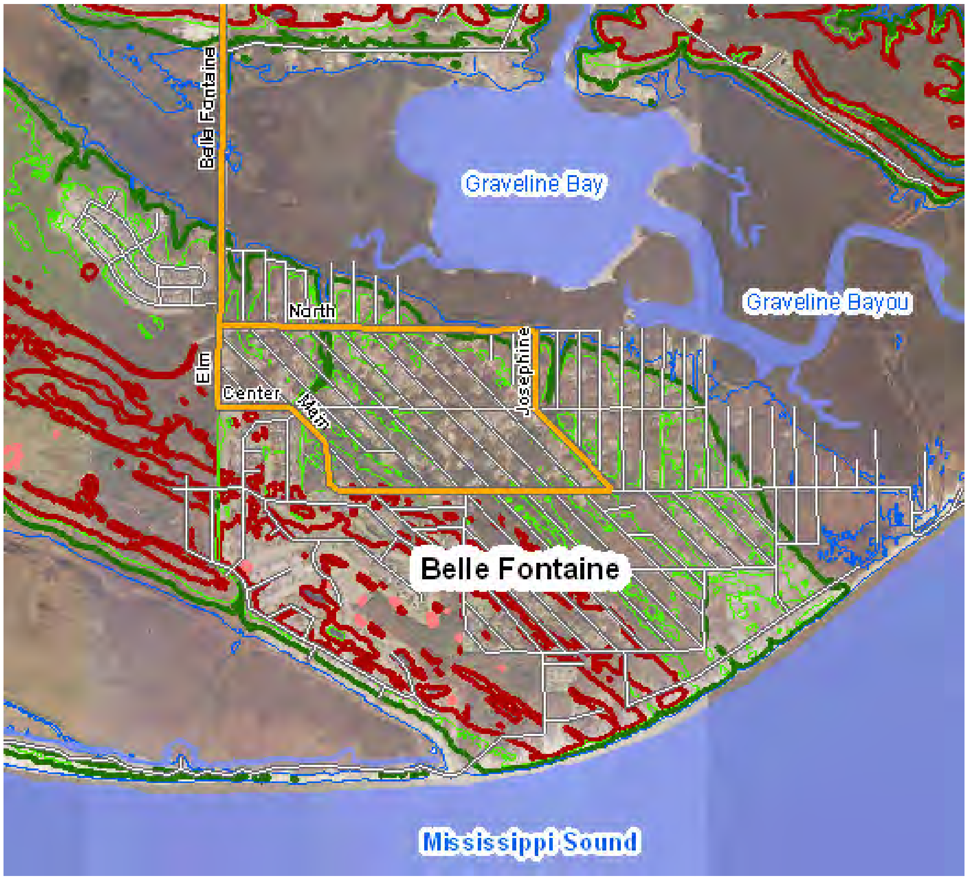

Figure 3.3.10-3. Existing Condition, Belle Fontaine

Source : http://ngs.woc.noaa.gov/storms/katrina/24330547.jpg

Figure 3.3.10-4. Hurricane Katrina Damage in Belle Fontaine

Source: http://ngs.woc.noaa.gov/storms/katrina/24330558.jpg

Figure 3.3.10-5. Hurricane Katrina Damage, Belle Fontaine

3.3.10.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8ft(dark green), 12-ft(light green), 16-ft(brown), and 20-ft(pink) ground contour lines and levee limits are shown below in Figure 3.3.10-6. The data indicates the Katrina high water was as high as 21 ft NAVD88 near the Mississippi Sound, totally inundating the area.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Belle Fontaine at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.10-7.

Existing Condition Stage –Frequency data for Save Point 30, just off the coast of Belle Fontaine, is shown below in Figure 3.3.10-8. The 95% and 5% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

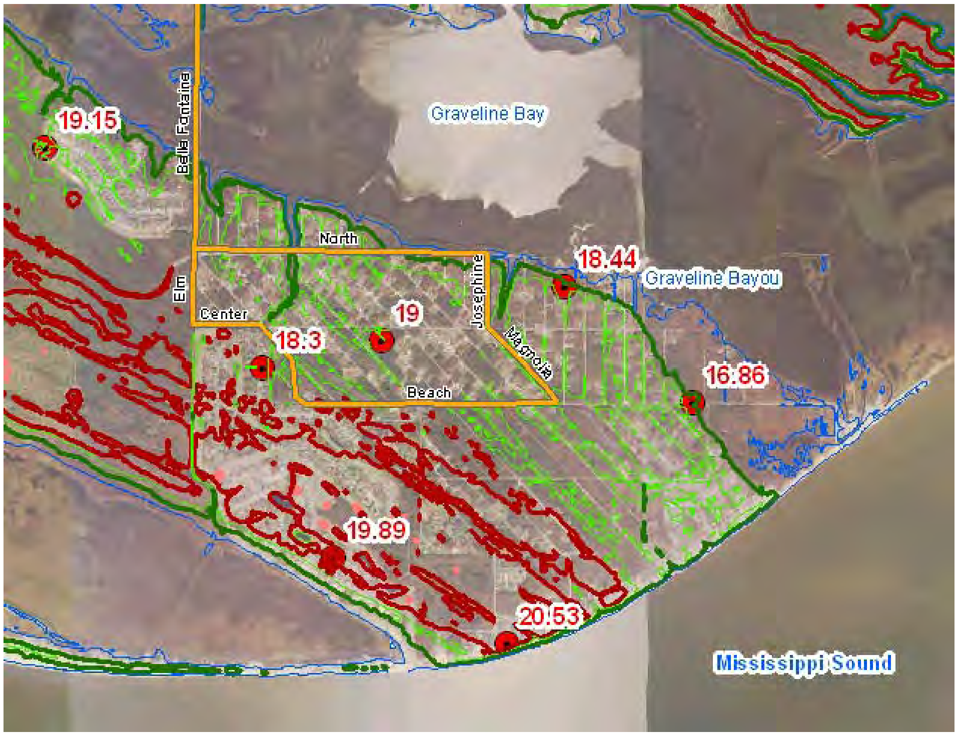

Figure 3.3.10-6. Ground Contours and Katrina High Water Elevations, Belle Fontaine

Figure 3.3.10-7. Hydrodynamic Modeling Save Points near Belle Fontaine

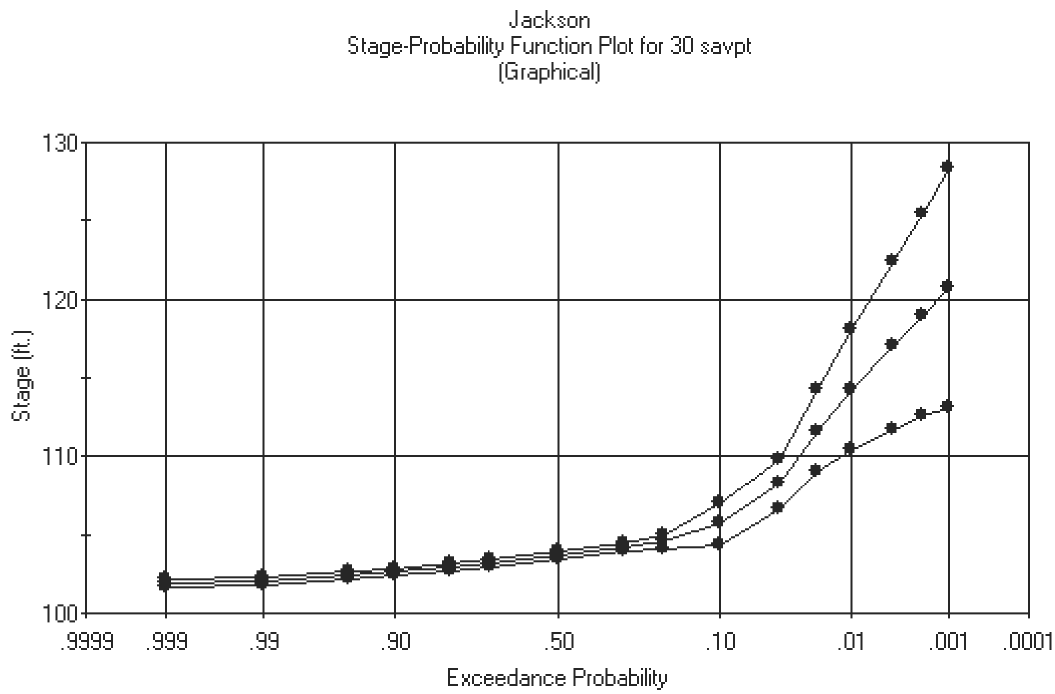

Figure 3.3.10-8. Existing Conditions at Save Point 30, near Belle Fontaine, MS

3.3.10.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 1440 acres around the subdivision of Belle Fontaine as shown on the following Figure 3.3.10-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal.

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in Figure 3.3.10-10.

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown in Figure 3.3.10-11 was caused by approximately 1-2 ft of overtopping crest depth.

Figure 3.3.10-9. Pump/Culvert/Sub-basin Locations

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.10-10. North Sea, Germany, March 1976

Source: ERDC, Steven Hughes

Figure 3.3.10-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.10-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

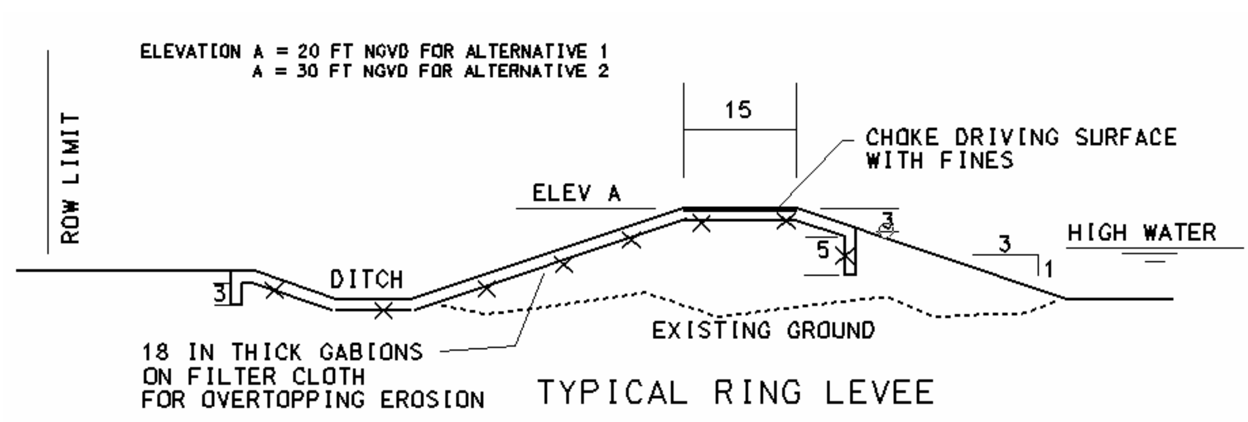

Figure 3.3.10-12. Typical Section at Ring Levee

3.3.10.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.10-13.

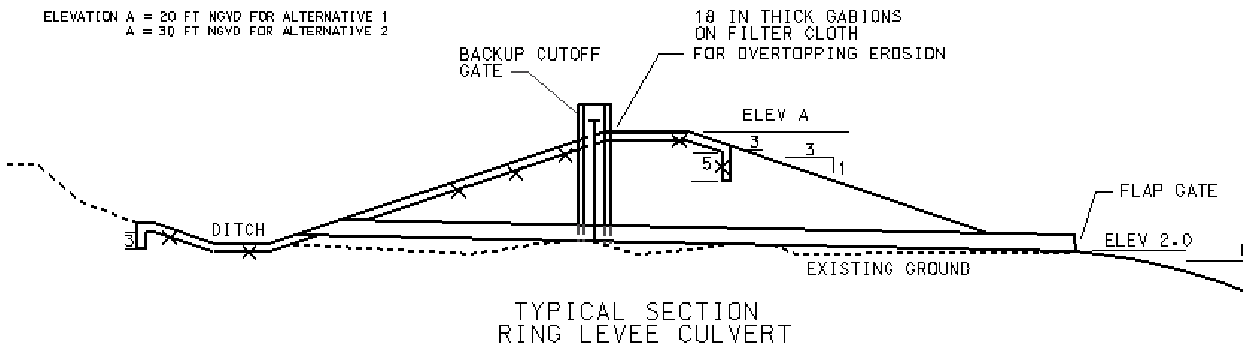

Figure 3.3.10-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types, hydrologic soil groups, and major sub-basins are shown below in Figure 3.3.10-14.

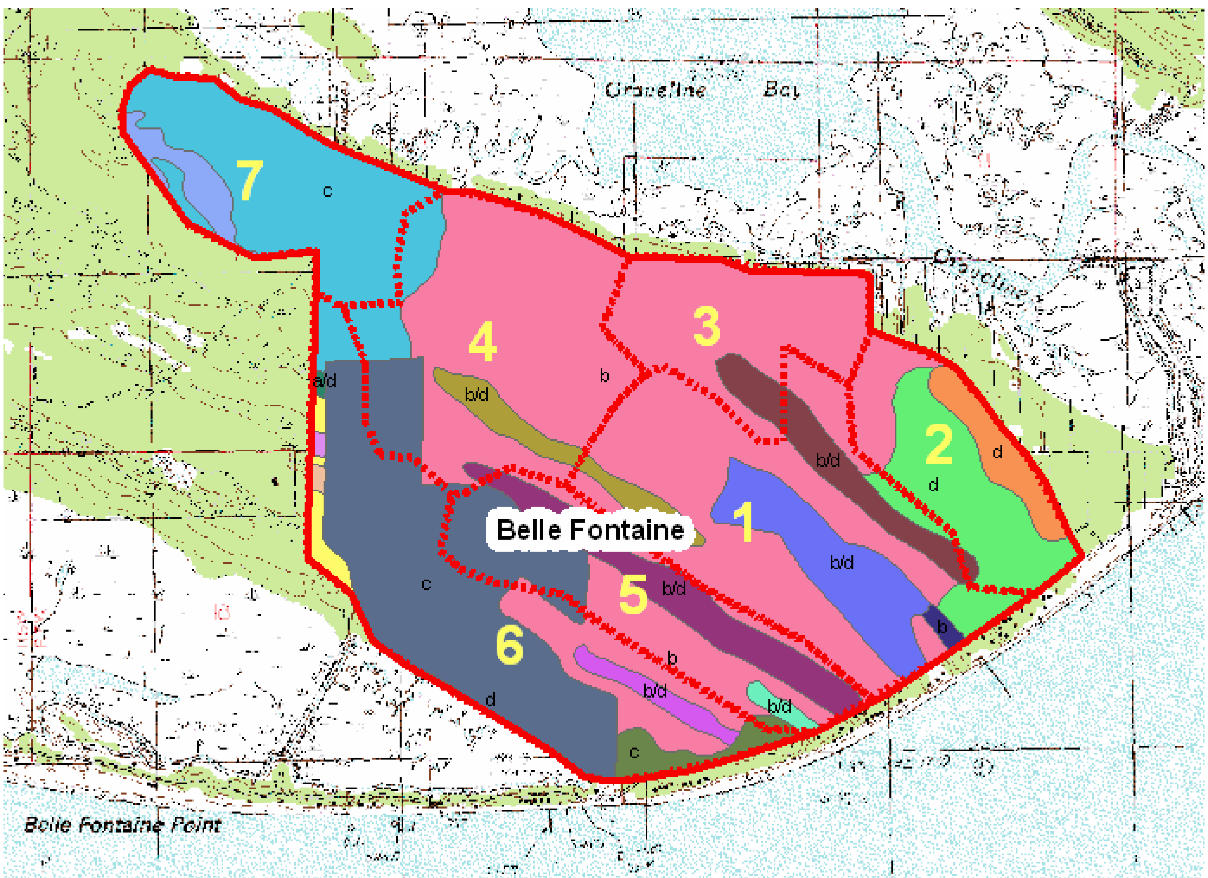

Figure 3.3.10-14. Belle Fontaine Hydrologic Soil Groups

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.10.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.10.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts and pumping facilities. The sites are shown above.

3.3.10.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.10.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.10.5.3.3 Pumping Stations Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer's stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.10.5.3.4 Pumping Stations Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.10.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 7 pumping facilities included in the Bellfontaine Ring Levee system for the elevation 20 protection level varied from approximately 10 to 15 feet and the corresponding flows required varied from 99,191 to 273,787 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 290 horsepower pumps, to one having four 60-inch diameter pumps each running at 560 horsepower.

3.3.10.5.3.6 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.10.5.3.7 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.10.5.3.8 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bellefontaine area to elevation 20, 10 roadway intersections would have to be accommodated. For this study it was estimated that 5 roller gate structures and 5 swing gate structures would be required.

3.3.10.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.10.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.10.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Power plants would require this level of security.

3.3.10.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.10.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.10.9, Cost Summary. Construction costs for the various options are included in Table 3.3.10-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.10-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and

coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.10.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.10.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Belle Fontaine. The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.10.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.10.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.10.6.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

3.3.10.6.3.1 Flow and Pump Sizes

Design hydraulic heads derived for the 7 pumping facilities included in the Bellefontaine Ring Levee system for the elevation 30 protection level varied from approximately 20 to 25 feet and the corresponding flows required varied from 99,191 to 273,787 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 475 horsepower pumps, to one having four 54-inch diameter pumps each running at 775 horsepower.

3.3.10.6.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bellefontaine area to elevation 30, 13 roadway intersections would have to be accommodated. For this study it was estimated that all 13 would require swing gate structures.

3.3.10.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.10.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.10.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.10.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.10.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.10.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.10.7 Option C – Alternate Alignment, Elevation 20 ft NAVD88

This option consists of an earthen levee at elevation 20 ft NAVD88 enclosing an area of 1341 acres around the most populated areas of Belle Fontaine in an alignment slightly different from the alignment for Options A and B. The alignment of the levee is shown in Figure 3.3.10-15 below, which also shows the variation in the drainage sub-basins and the locations of the pumps and culverts.

Figure 3.3.10-15. Alternative Alignment Pump/Culvert/Sub-basin Locations

3.3.10.7.1 Interior Drainage

Interior drainage flows are similar to those computed for Option A, above. However, ditches, culverts and pumps were re-sized by adjusting the previously computed flows by the ratio of the change in areas of the sub-basins to get the revised flows.

3.3.10.7.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option C are the same as for Option A, above.

3.3.10.7.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option C is presented below.

3.3.10.7.3.1 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 7 pumping facilities included in the Bellefontaine Ring Levee system for the elevation 20 protection level varied from approximately 10 to 20 feet and the corresponding flows required varied from 99,453 to 274,644 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 290 horsepower pumps, to one having five 42-inch diameter pumps each running at 475 horsepower.

3.3.10.7.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bellefontaine area to elevation 20, 13 roadway intersections would have to be accommodated. For this study it was estimated that 5 of these would require 10 swing gate structures with the remaining 8 requiring roller gates of varying heights.

3.3.10.7.4 HTRW

The HTRW paragraphs for Option C are the same as for Option A, above.

3.3.10.7.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option C are the same as for Option A, above.

3.3.10.7.6 Project Security

The Project Security paragraphs for Option C are the same as for Option A, above.

3.3.10.7.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option C are the same as for Option A, above.

3.3.10.7.8 Cost Estimate

The Cost Estimate paragraphs for Option C are the same as for Option A, above.

3.3.10.7.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option C are the same as for Option A, above.

3.3.10.8 Option D – Alternate Alignment, Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Belle Fontaine. The alignment of the levee is the same as Option C, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option C is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.10.8.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option C, above, except that the culvert lengths through the levees would be longer.

3.3.10.8.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option D are the same as for Option A, above.

3.3.10.8.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option D is presented below.

3.3.10.8.3.1 Pumping Stations. Flow and Pump Sizes.

Design hydraulic heads derived for the 7 pumping facilities included in the Bellefontaine Ring Levee system for the elevation 30 protection level varied from approximately 20 to 30 feet and the corresponding flows required varied from 99,453 to 274,644 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 475 horsepower pumps, to one having three 60-inch diameter pumps each running at 1150 horsepower.

3.3.10.8.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Bellefontaine area to elevation 30, 11 roadway intersections would have to be accommodated. For this study it was estimated that all 11 would require 22 swing gate structures.

3.3.10.8.4 HTRW

The HTRW paragraphs for Option D are the same as for Option A, above.

3.3.10.8.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option D are the same as for Option A, above.

3.3.10.8.6 Project Security

The Project Security paragraphs for Option D are the same as for Option A, above.

3.3.10.8.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option D are the same as for Option A, above.

3.3.10.8.8 Cost Estimate

The Cost Estimate paragraphs for Option D are the same as for Option A, above.

3.3.10.8.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option D are the same as for Option A, above.

3.3.10.9 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.10-1 and 3.3.10-2, below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.10-1.
Jackson Co Belle Fontaine Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$137,600,000
Option B – Elevation 30 ft NAVD88	$191,900,000
Option C – Elevation 30 ft NAVD88	$103,900,000
Option D – Elevation 30 ft NAVD88	$142,900,000

Table 3.3.10-2.
Jackson Co Belle Fontaine Ring Levee O & M Cost Summary

Option	O&M Cost
Option A – Elevation 20 ft NAVD88	$1,371,000
Option B – Elevation 30 ft NAVD88	$1,939,000
Option C – Elevation 30 ft NAVD88	$989,000
Option D – Elevation 30 ft NAVD88	$1,414,000

3.3.10.10 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

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3.3.11 Jackson County, Gautier Ring Levee

3.3.11.1 General

Several high density residential and business areas in Jackson County were identified. They are: Pascagoula/Mosspoint, Gautier, Belle Fontaine, Gulf Park Estates, and Ocean Springs. These are subject to damage from storm surges associated with hurricanes. Earthen ring levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.11.2 Location

The location of the Gautier ring levee in Jackson County is shown below in Figures 3.3.11-1 and 3.3.11-2.

3.3.11.3 Existing Conditions

Gautier is located on the west side of the Pascagoula River delta at the mouth of the West Pascagoula River at the Mississippi Sound. Ground elevations over most of the residential and business areas vary between elevation 10-20 ft NAVD88. The southern-most part of the area is drained by drowned natural drainage ways. The 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and city limits are shown below in Figure 3.3.11-3.

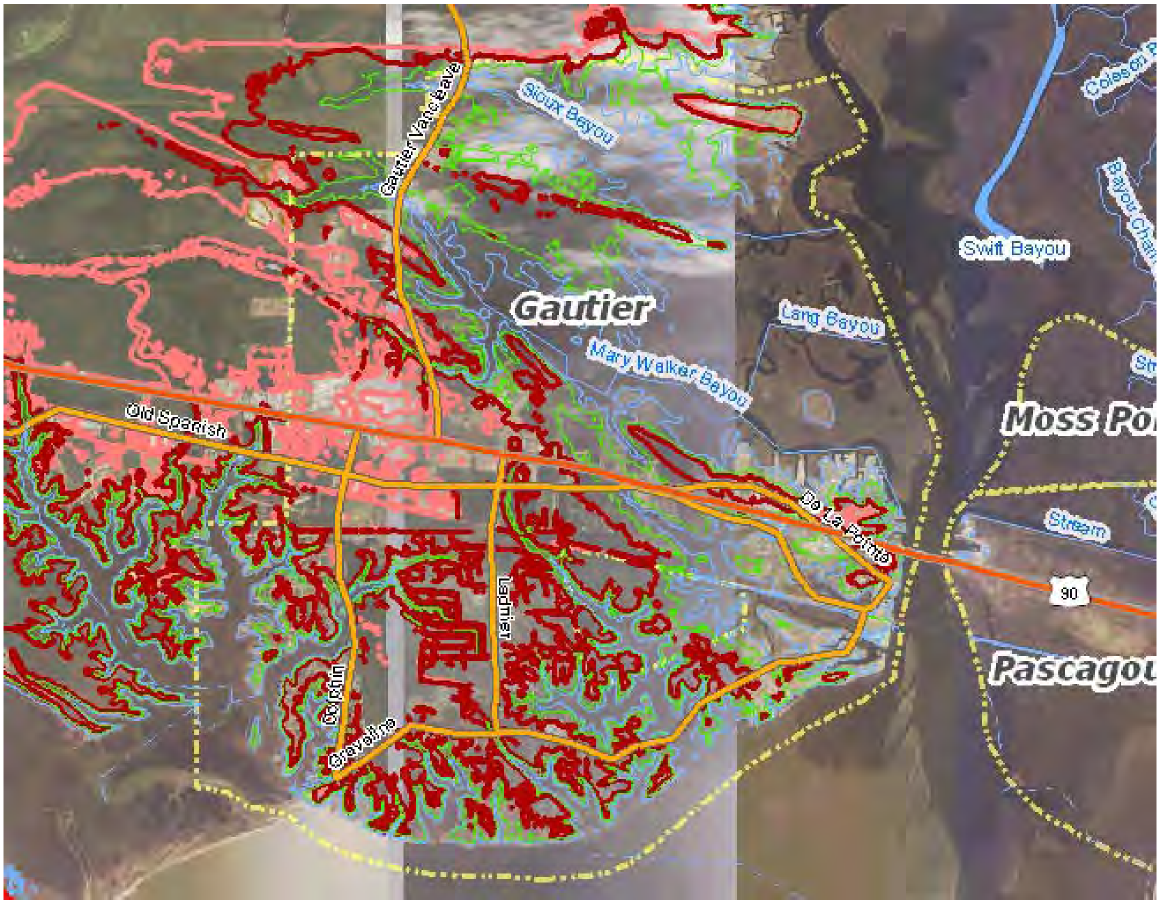

Figure 3.3.11-1 Vicinity Map, Gautier, MS

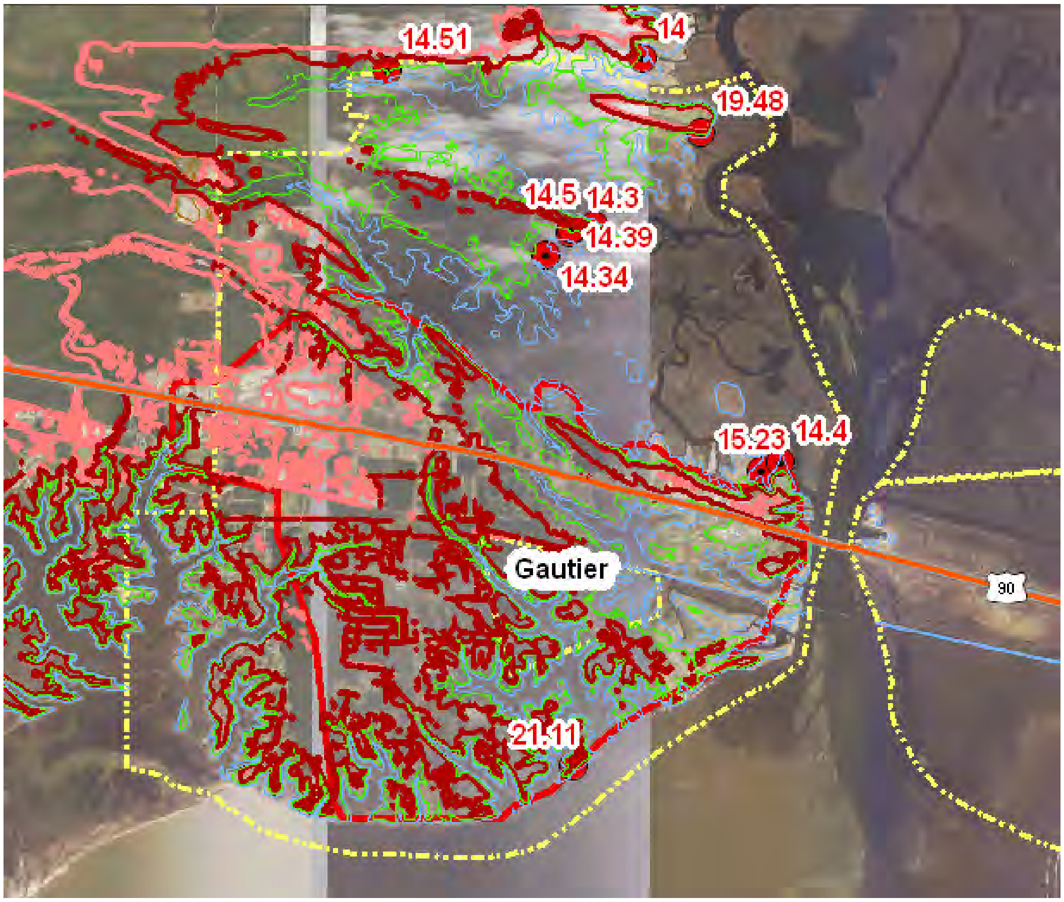

Figure 3.3.11-2 Gautier Ring Levee

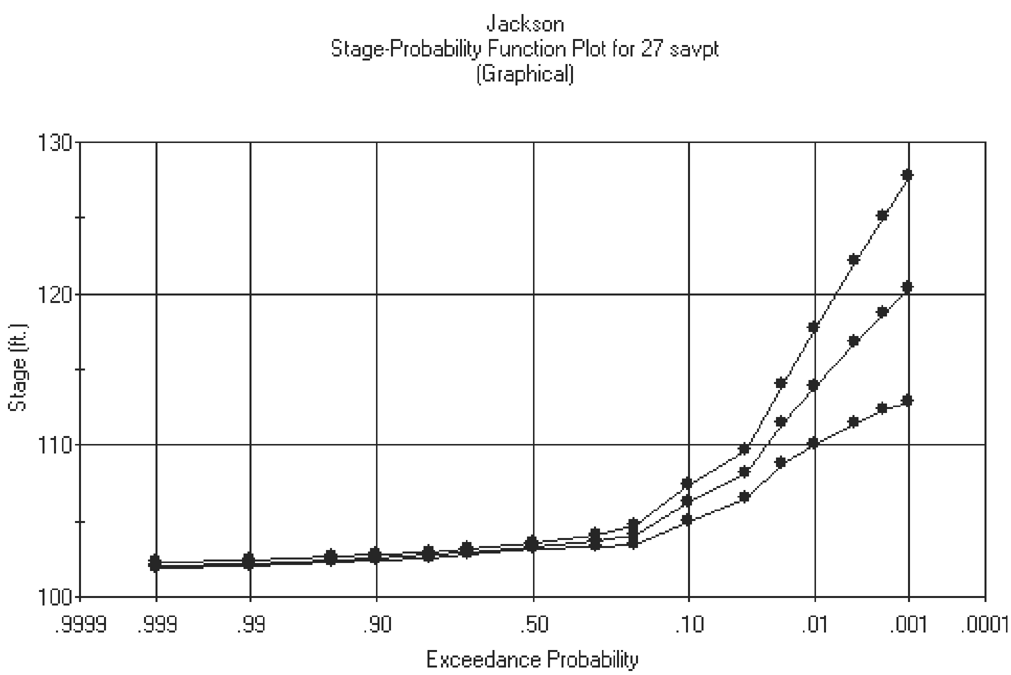

Figure 3.3.11-3. Existing Conditions

Drainage in the southern part of the city is through drowned streams that empty into Mississippi Sound. These are therefore unusually wider at the mouth and have productive environmental and recreational benefits.

Drainage from ordinary rainfall is hindered on occasions when either of the rivers or the gulf is high, but impacts from hurricanes are devastating.

Recent damage from Hurricane Katrina in August, 2005 the Gautier area are shown below in Figure 3.3.11-4 and 3.3.11-5. Many homes are still un-repaired, pending settlement of insurance claims.

Source: http://coastal.er.usgs.gov/hurricanes/katrina/quickphotos/gautier/

Figure 3.3.11-4. Hurricane Katrina Damage in Gautier, MS

Figure 3.3.11-5. Hurricane Katrina Damage in Gautier, MS

3.3.11.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4, of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and city limits are shown below in Figure 3.3.10-6. The data indicates the Katrina high water was as high as 21 ft NAVD88 at the Mississippi Sound and 15 ft NAVD88 north of Hwy 90.

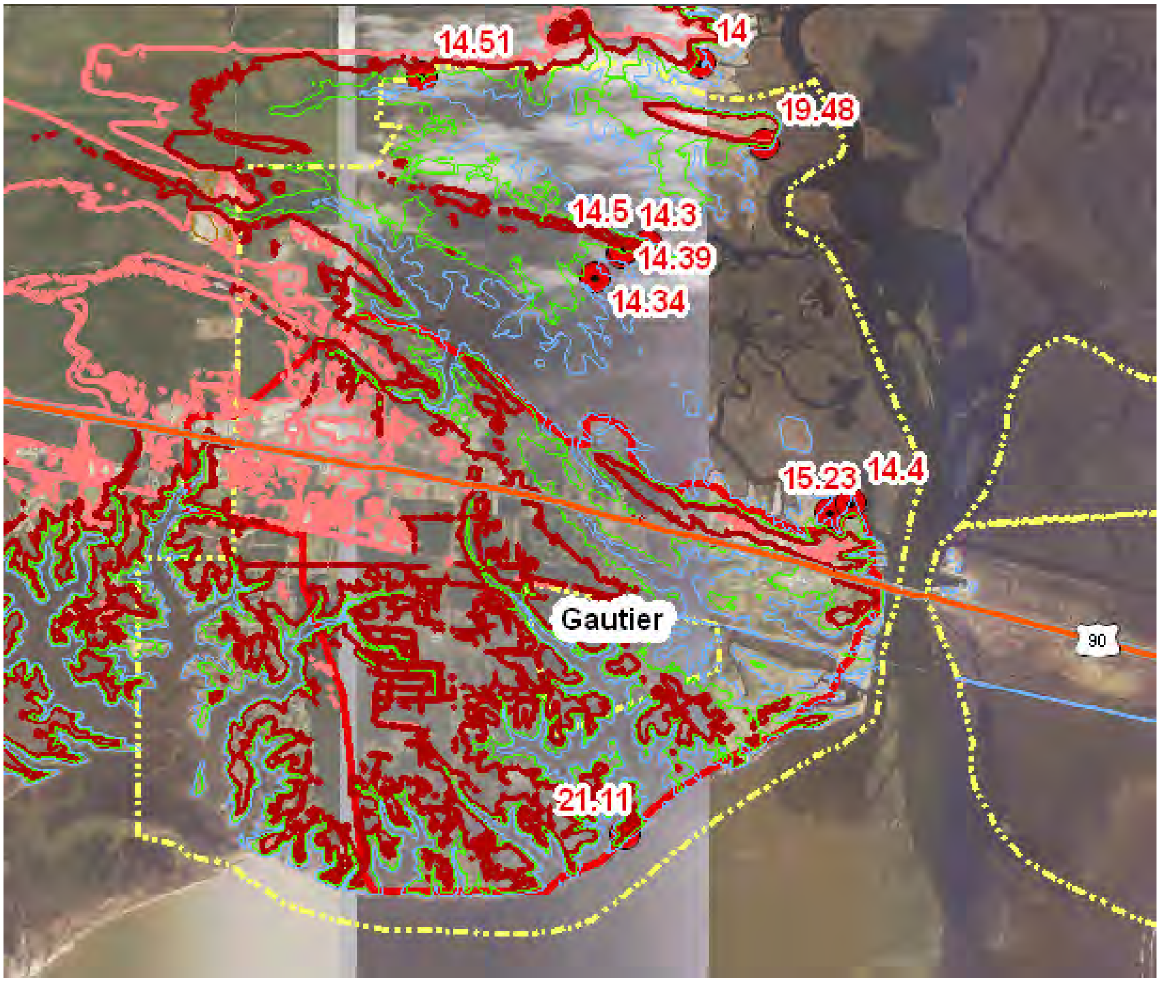

Figure 3.3.11-6. Ground Contours and Katrina High Water Elevations

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix 3.3.11.2. Points near Gautier at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.11-7.

Existing Condition Stage –Frequency data for Save Point 27, just off the coast of Gautier, is shown below in Figure 3.3.11-8. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

Figure 3.3.11-7. Hydrodynamic Modeling Save Points near Gautier

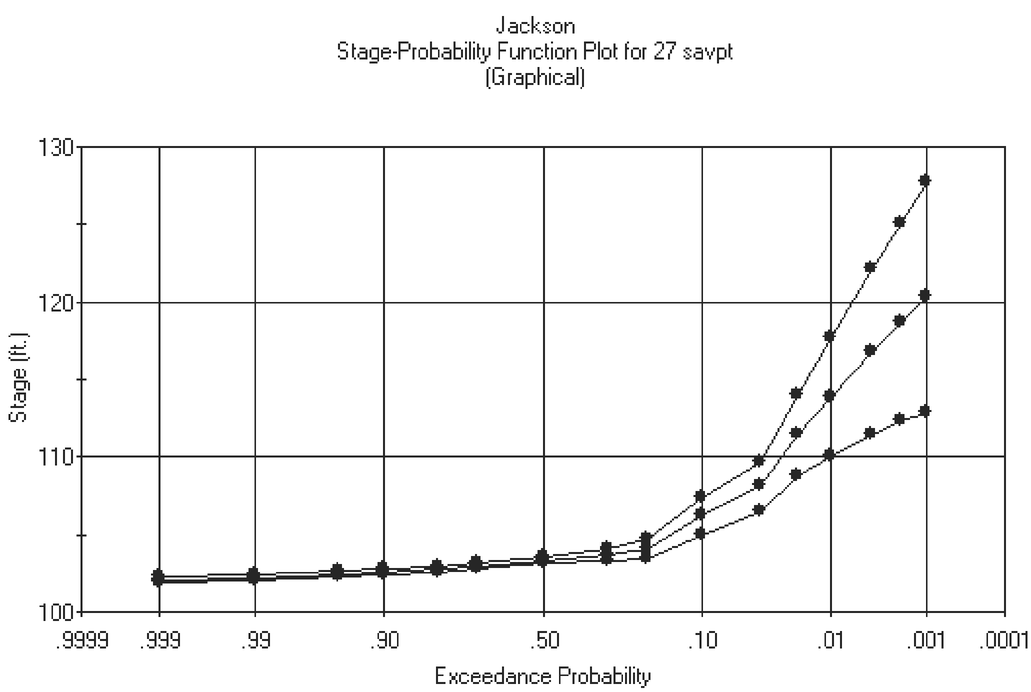

Figure 3.3.11-8. Existing Conditions at Save Point 27, near Gautier, MS

3.3.11.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 4833 acres around the most densely populated areas of Gautier as shown on the following Figure 3.3.11-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal. A small boat access structure is also shown at the mouth of several basins. Rising sector gates will be provided at these sites allowing shallow draft traffic most of the time. The gates will be closed prior to hurricane storm surge. Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown below in Figure 3.3.11-10.

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.11-11 was caused by approximately 1-2 ft of overtopping crest depth.

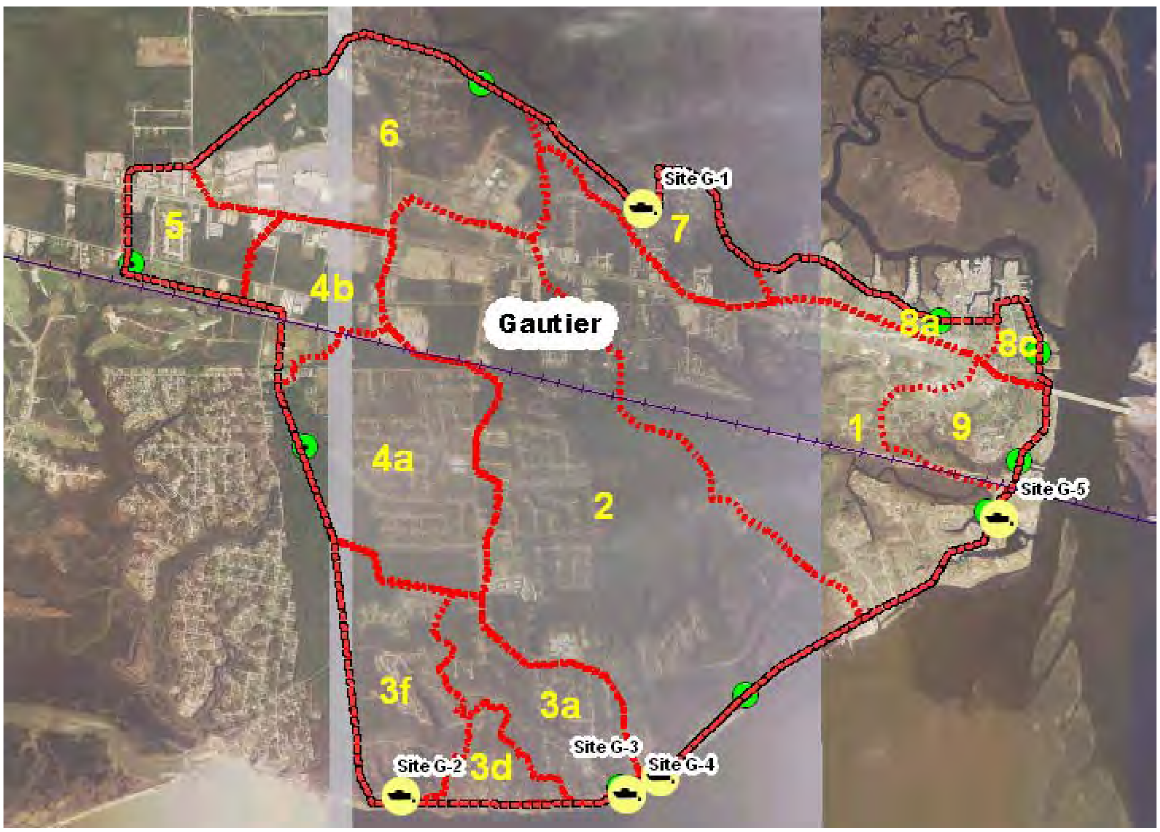

Figure 3.3.11-9. Pump/Culvert/Sub-basin/Boat Access Site Locations

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.11-10. North Sea, Germany, March 1976

Source: ERDC, Steven Hughes

Figure 3.3.11-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.3.11-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

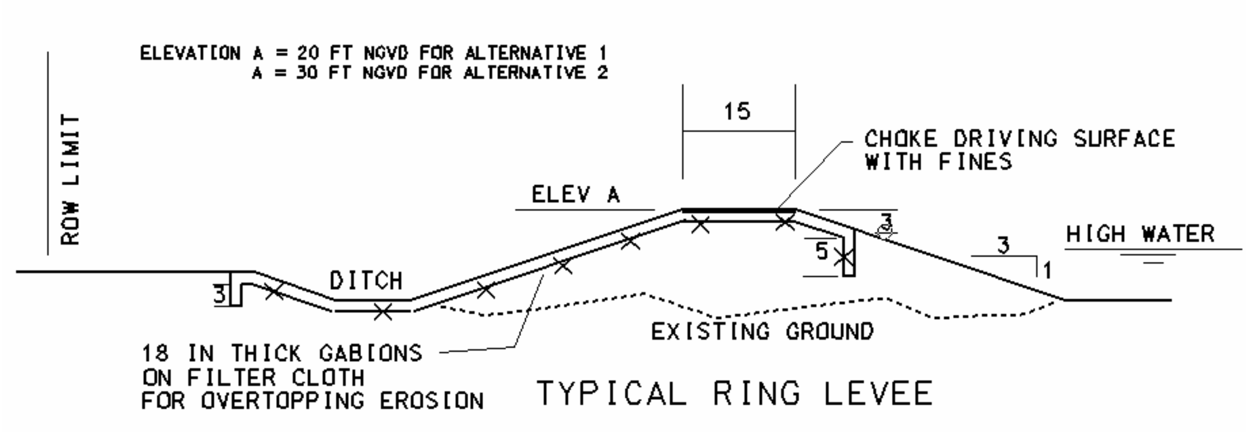

Figure 3.3.11-12. Typical Section at Ring Levee

3.3.11.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above in Figure 3.3.11-9. The culverts would have tidal gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at the upstream end at every culvert in the levee for manual control in the event the tidal gate malfunctions. A typical section is shown is shown below in Figure 3.3.11-13.

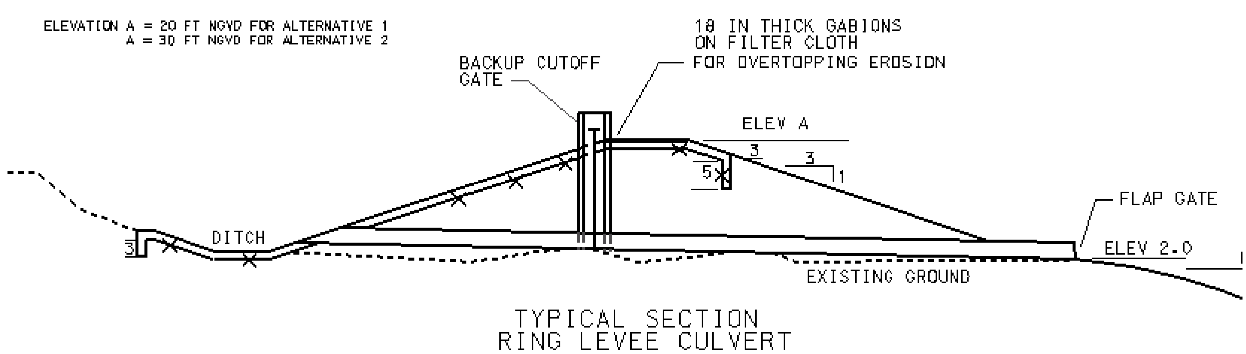

Figure 3.3.11-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown above in Figure 3.3.11-9 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type shown below in Figure 3.3.11-14 and land use to determine a run-off curve number. The variation in soil type, hydrologic soil groups, and sub-basins is shown below in Figure 3.3.11-14.

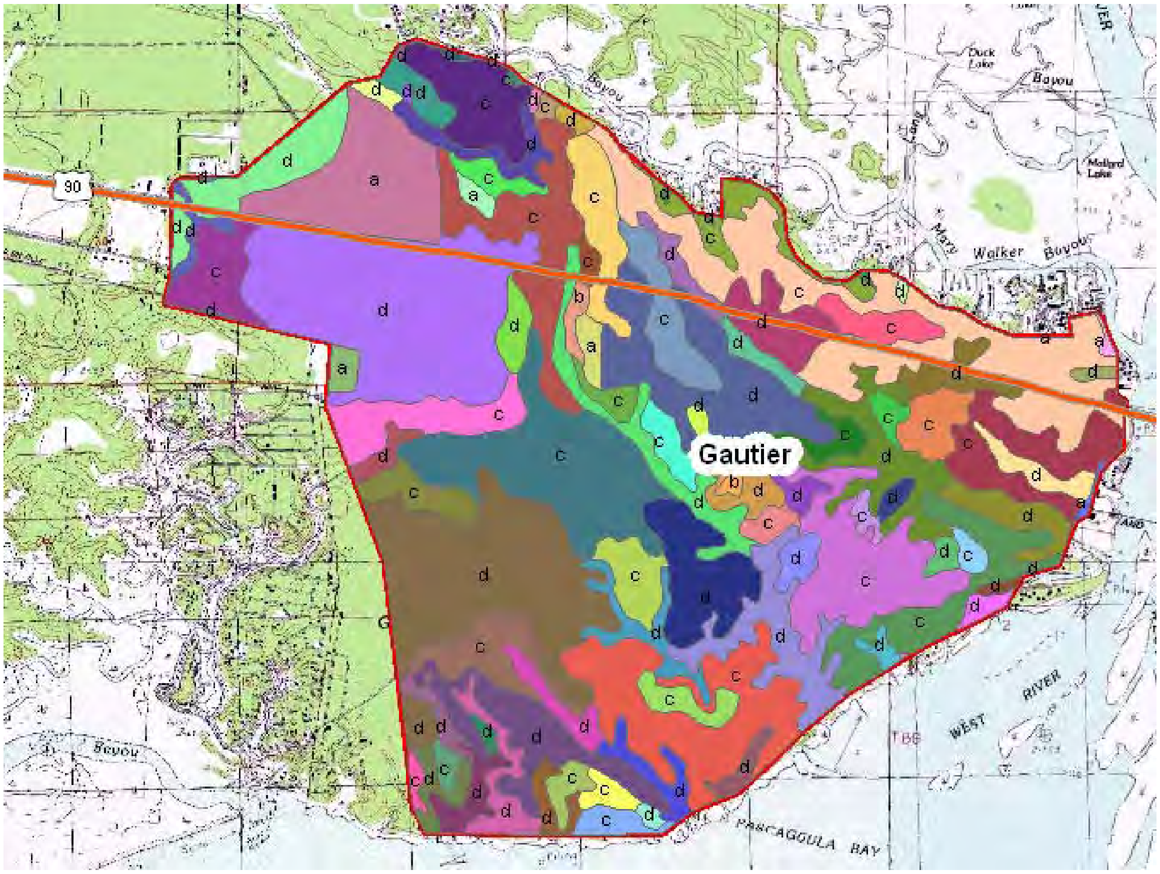

Figure 3.3.11-14. Gautier Hydrologic Soil Groups

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Aerial Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes

(And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.11.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.11.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts, pumping facilities and for boat access sites. The sites are shown above in Figure 3.3.11-9.

3.3.11.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.11.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.11.5.3.3 Pumping Facilities Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.11.5.3.4 Pumping Facilities Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.11.5.3.5 Pumping Stations. Flow and Pump Sizes

Design hydraulic heads derived for the 11 facilities included in the Gautier Ring Levee system for the elevation 20 protection level varied from approximately 15 to 20 feet and the corresponding flows required varied from 65,081 to 558,795 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 300 horsepower pumps, to one having six 60-inch diameter pumps each running at 560 horsepower.

3.3.11.5.3.6 Boat Access Structure

At five sites the Gautier ring levee alignment would cross a moderately sized water course where it is apparent that boats currently traverse the area. (See Figure 3.3.11-9 above). To allow continued free boat access to the areas behind the levee this site was fitted with a scaled down adaptation of the larger rising sector gate structure used for the bay barriers at Biloxi and Bay Saint Louis. This structure would, for the most part, be much smaller and lighter than those used in the bays, however it would be substantial. The operation would be similarly critical in time of storm and they would require the same attention from an Operations and Maintenance standpoint as their larger, heavier counterparts. The structure is shown below in Figure 3.3.11-15 and in Table 3.3.11-1.

Table 3.3.11-1.
Boat Access Structure Dimensional Data by Site

Site Designation	Protection Elevation, ft NAVD88	L1 ft	PW ft	H ft
G-1	20.0	50	18	29.5
	30.0	50	18	42.0
G-2	20.0	70	18	29.5
	30.0	70	18	42.0
G-3	20.0	32	18	29.5
	30.0	32	18	42.0
G-4	20.0	132	18	29.5
	30.0	132	18	42.0
G-5	20.0	104	18	29.5
	30.0	104	18	42.0

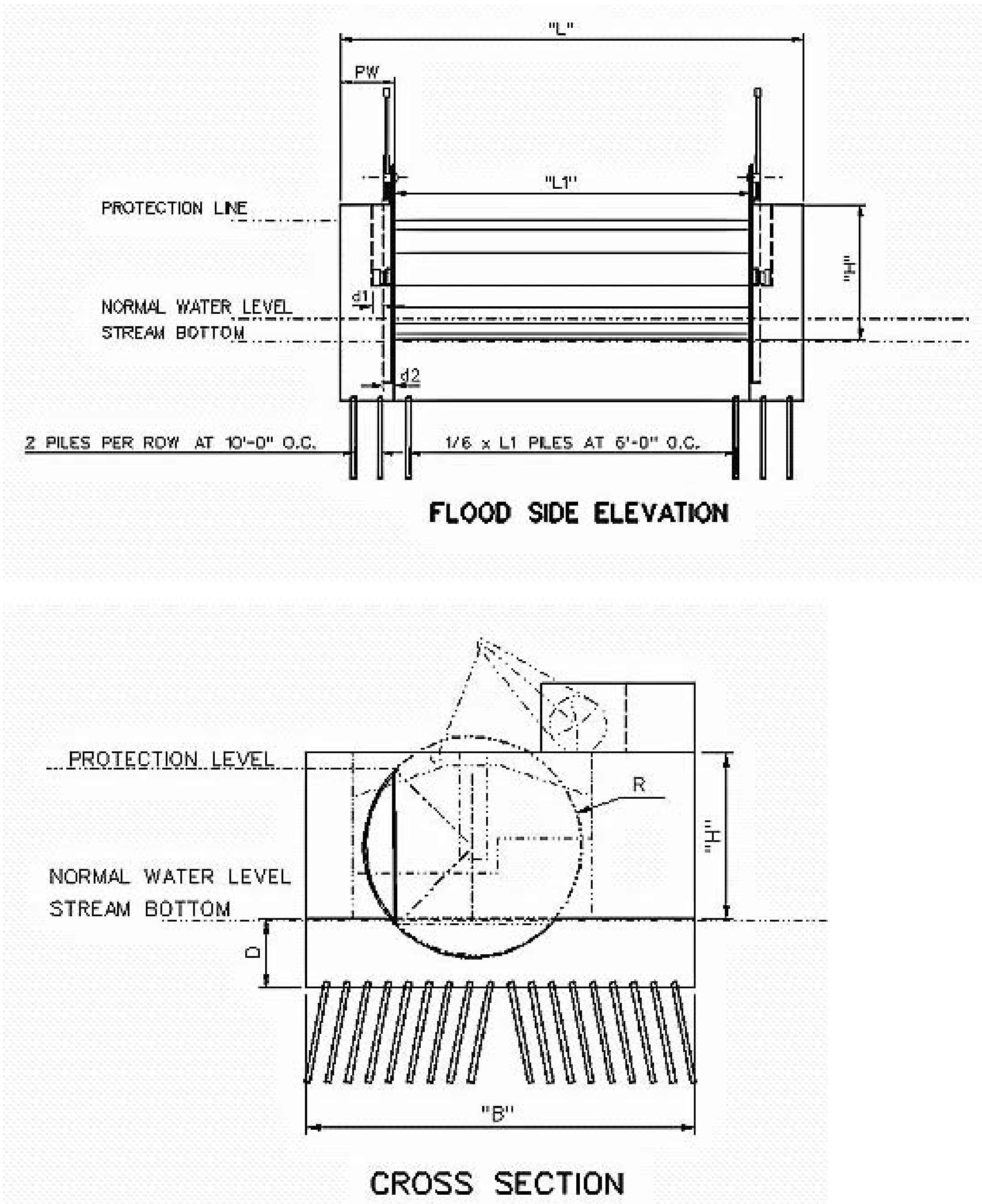

Figure 3.3.11-15. Typical Small Boat Access Structure

3.3.11.5.3.7 Boat Access Structure. Mechanical

The mechanical equipment and operating system for these structures would be similar to those used for the bay barriers, and would include steel gate linkages and hydraulic rams and pivot pins for operation of the gates. Each gate would rotate on large bearings and pivot hubs at the ends of the gate. Various operating hydraulic and lubrication oil systems would also be required. It is estimated that each gate would have a maximum opening/closing time of 15 minutes.

3.3.11.5.3.8 Boat Access Structure. Electrical

Primary electrical power for operating these gates would be provided using dedicated, standard transformers with emergency back-up generators. The electrical load demand at these facilities would be low by comparison to the bay barrier structures. The supplemental generation aspect was considered to be a vital component of the design because of the very high cost of commercial standby power and because commercial electric power would almost certainly be unavailable during and immediately following a storm event.

3.3.11.5.3.9 Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.11.5.3.10 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.11.5.3.11 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Gautier area to elevation 20, 20 roadway intersections would have to be accommodated. For this study it was estimated that 11 roller gate structures and 11 swing gate structures would be required.

3.3.11.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.11.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.11.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Boat access gates and power plants would require this level of security.

3.3.11.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.11.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.11.7, Cost Summary. Construction costs for the various options are included in Table 3.3.11-2 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.11-3. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.11.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.11.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Gautier The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.11.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.11.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.11.6.3 Structural, Mechanical and Electrical

These data are the same as that presented for Option A and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B are presented below.

3.3.11.6.3.1 Pumping Stations Flow and Pump Sizes

Design hydraulic heads derived for the 11 facilities included in the Gautier Ring Levee system for the elevation 20 protection level were steady at approximately 25 feet and the corresponding flows required varied from 65,081 to 558,795 gallons per minute. The plants thus derived varied in size from a plant having two 42-inch diameter, 500 horsepower pumps, to one having six 60-inch diameter pumps each running at 1000 horsepower.

3.3.11.6.3.2 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Gautier area to elevation 30, 23 roadway intersections would have to be accommodated. For this study it was estimated that all 23 would require swing gate structures.

3.3.11.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.11.6.5 Construction and Water Control Plan

The Construction and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.3.11.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.11.6.7 Operation and Maintenance.

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.11.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.11.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.11.7 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are in Tables 3.3.11-2 and 3.3.11-3, shown below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.11-2.
Jackson Co Gautier Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$348,300,000
Option B – Elevation 30 ft NAVD88	$450,100,000

Table 3.3.11-3.
Jackson Co Gautier Ring Levee O & M Cost Summary

Option	Cost for O&M
Option A – Elevation 20 ft NAVD88	$3,744,000
Option B – Elevation 30 ft NAVD88	$4,904,000

3.3.11.8 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

USACE 1993. Hydrologic Frequency Analysis. Engineer Manual EM 1110-2-1415. Department of the Army, US Army Corps of Engineers, Washington, D.C. 5 March 1993.

USACE 1995. Hydrologic Engineering Requirements for Flood Damage Reduction Studies. Engineer Manual EM 1110-2-1419. Department of the Army, US Army Corps of Engineers, Washington, D.C. 31 January 1995.

USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

National Resource Conservation Service (NRCS). 2003. WinTR5-55 User Guide (Draft). Agricultural Research Service. 7 May 2003.

Environmental Science Services Administration. 1968. "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, "Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.3.12 Jackson County, Pascagoula/Moss Point Ring Levee

3.3.12.1 General

Several high density residential and business areas in Jackson County were identified. They are: Pascagoula/Mosspoint, Gautier, Belle Fontaine, Gulf Park Estates, and Ocean Springs. These are subject to damage from storm surges associated with hurricanes. Earthen ring levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center's (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.13 of the Engineering Appendix and in the Economic Appendix.

3.3.12.2 Location

The general location of the Pascagoula/Moss Point ring levee in Jackson County is shown below in Figure 3.3.12-1. Four optional alignments are presented. Each has two levee height options. Each one is presented separately. The optional alignments are shown in Figure 3.3.12-2.

Figure 3.3.12-1. Vicinity Map Pascagoula, MS

The basic alignment is the most extensive and covers the main residential area in Pascagoula and Moss Point.

The Washington Ave. Alternate Alignment is the same as the basic alignment except that the alignment follows Washington Ave. on the southernmost leg of the levee.

The Moss Point Alternate Alignment is the same as the basic alignment except that the alignment follows higher ground on the northernmost part of the levee.

The Combined Washington Ave. and Moss Point Alternate Alignment is the same as the basic alignment except that in includes both the Washington Ave. and the Moss Point modifications on the north and south.

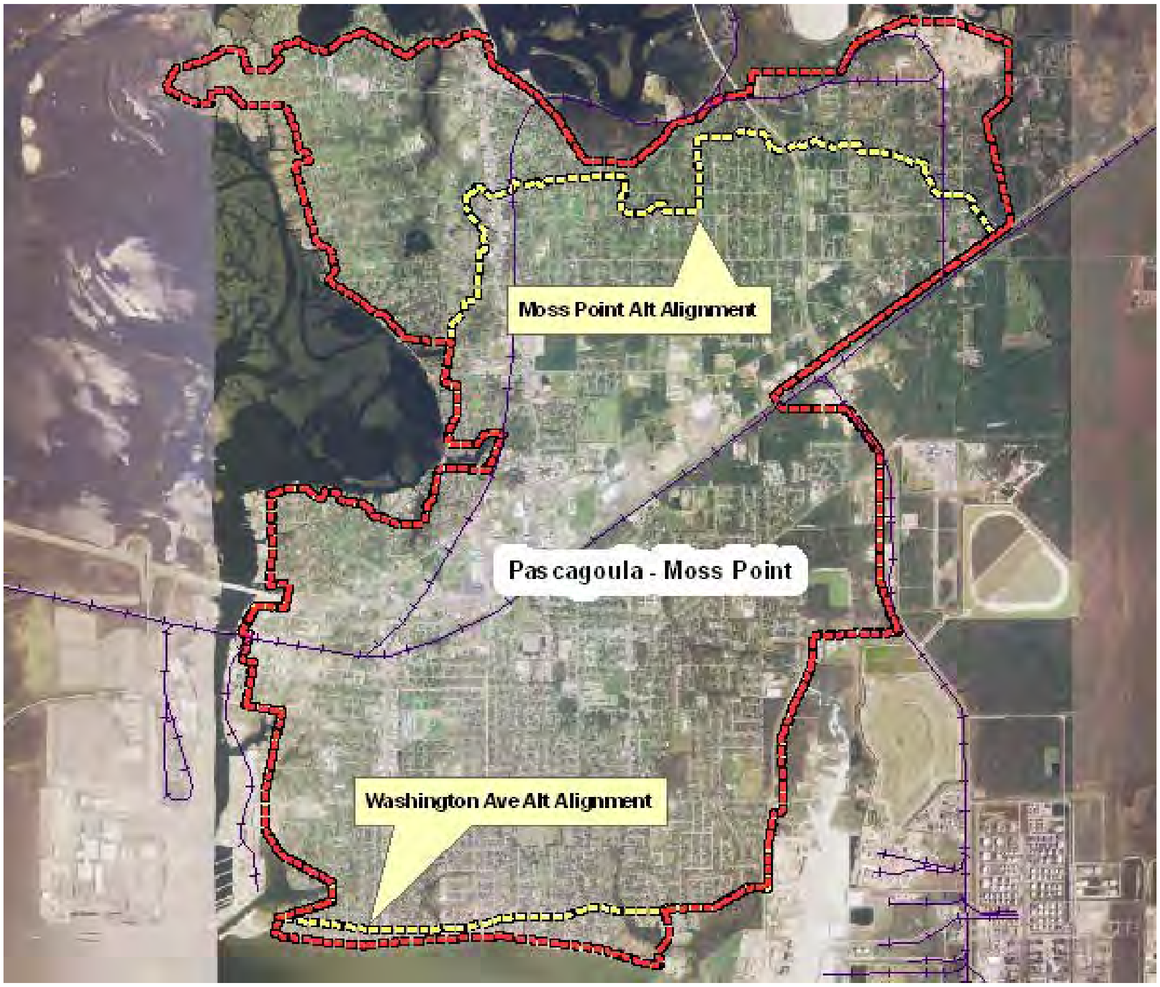

Figure 3.3.12-2. Pascagoula/Moss Point Ring Levees

3.3.12.3 Existing Conditions

The cities of Moss Point and Pascagoula lie at the confluence of the Escatawpa and Pascagoula Rivers along the gulf coast on Mississippi Sound. Both the northern part of Moss Point and the southern part of Pascagoula are very flat. Ground elevations over most of the residential and business areas vary between elevation 10-12 ft NAVD88 in the southern part of the area (Pascagoula) and 14-20 ft NAVD88 in the northern part (Moss Point). The 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and city limits are shown below in Figure 3.3.12-3.

The cities are drained by natural and some improved channels. These channels drain to the north to the Escatawpa River, the west to the Pascagoula River, to the south to the gulf, and to the east to Grand Bay Swamp, thence to the gulf. All are obviously subject to tidal influence.

Drainage from ordinary rainfall is hindered on occasions when either of the rivers or the gulf is high, but impacts from hurricanes are devastating.

Damage from Hurricane Katrina in August, 2005 in the Pascagoula area are shown below in Figures 3.3.12-4 and 3.3.12-5. Many homes are still un-repaired, pending settlement of insurance claims.

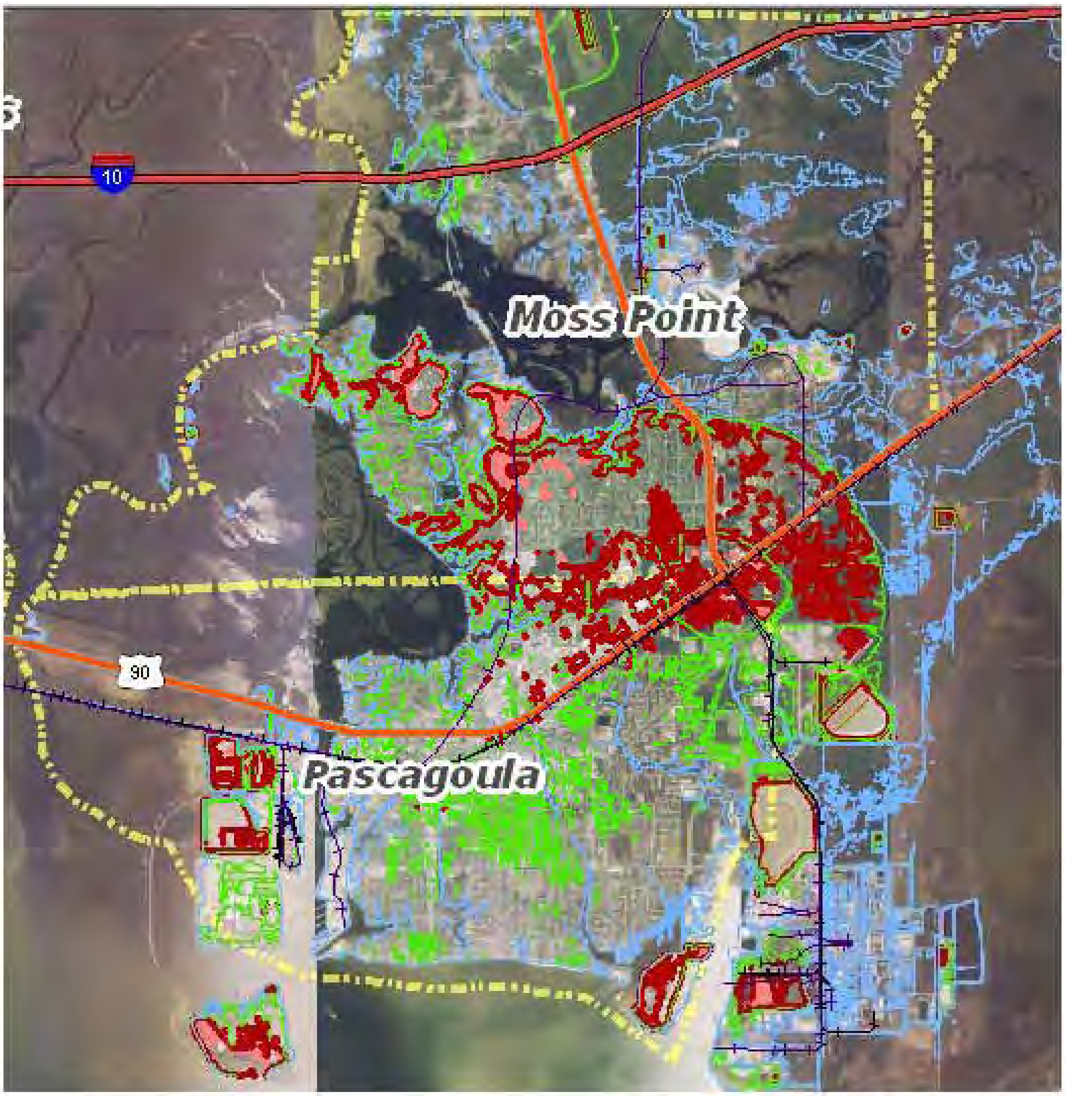

Figure 3.3.12-3. Existing Conditions Pascagoula, MS

Source : http://ngs.woc.noaa.gov/storms/katrina/24330050.jpg

Figure 3.3.12-4. Hurricane Katrina Damage Pascagoula, MS

Source: http://www.wunderground.com/hurricane/Katrinas_surge_part15.asp

Figure 3.3.12-5. Hurricane Katrina Damage Pascagoula, MS

3.3.12.4 Coastal and Hydraulic Data

Typical coastal data are shown in Section 1.4 of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines and city limits are shown below in Figure 3.3.12-6. The data indicates the Katrina high water was as high as 18-20 ft NAVD88 near the Mississippi Sound at Pascagoula and 12-15 ft NAVD88 in Moss Point.

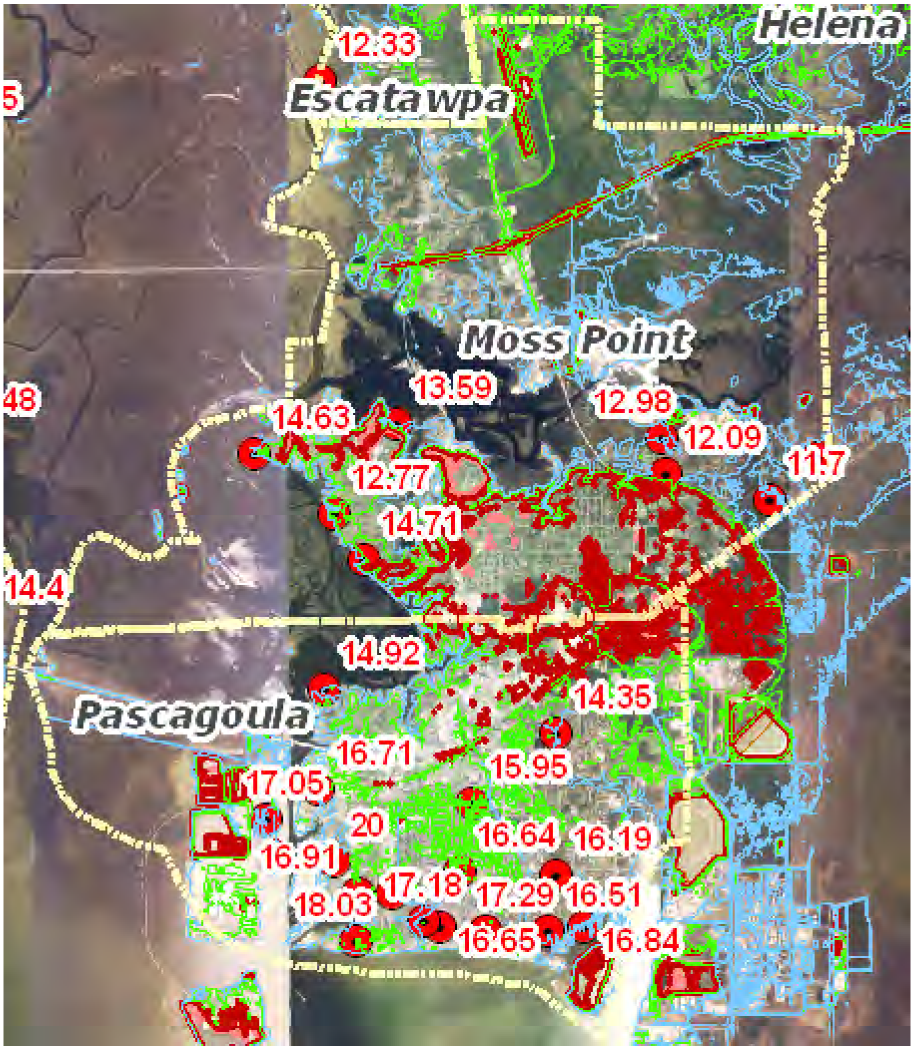

Figure 3.3.12-6. Ground Contours and Katrina High Water Elevations

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.13 of the Engineering Appendix and in the Economic Appendix. Points near Pascagoula at which data from hydrodynamic modeling was saved are shown below in Figure 3.3.12-7.

Figure 3.3.12-7. Hydrodynamic Modeling Save Points near Pascagoula

Existing Condition Stage –Frequency data for Save Point 22, just off the coast of Pascagoula, is shown below in Figure 3.3.12-8. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

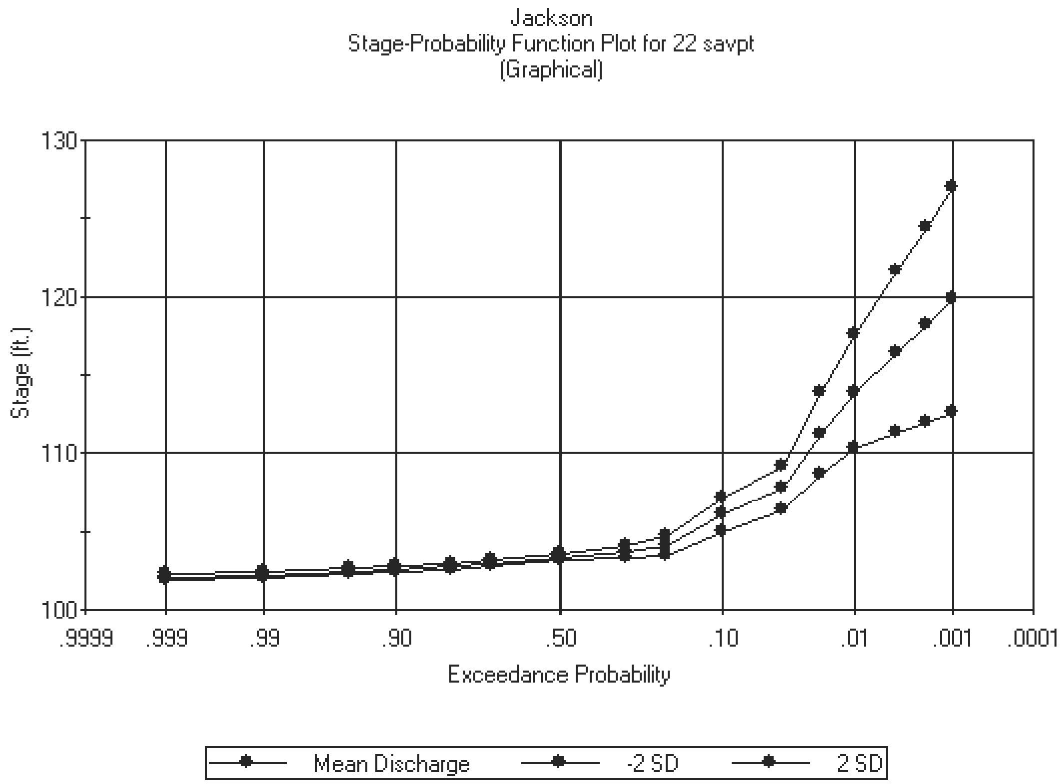

Figure 3.3.12-8. Existing Conditions at Save Point 22, near Pascagoula, MS

3.3.12.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike enclosing an area of 9523 acres around the most densely populated areas of Moss Point and Pascagoula as shown on the following Figure 3.3.12-9, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal. A small boat access structure is also shown at the mouth of Basin 20, PG-1 Site. Rising sector gates will be provided at this site allowing shallow draft traffic most of the time. The gates will be closed prior to hurricane storm surge. A drawing of a typical boat access gate is shown in Figure 3.3.12-15.

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in Figure 3.3.12-10.

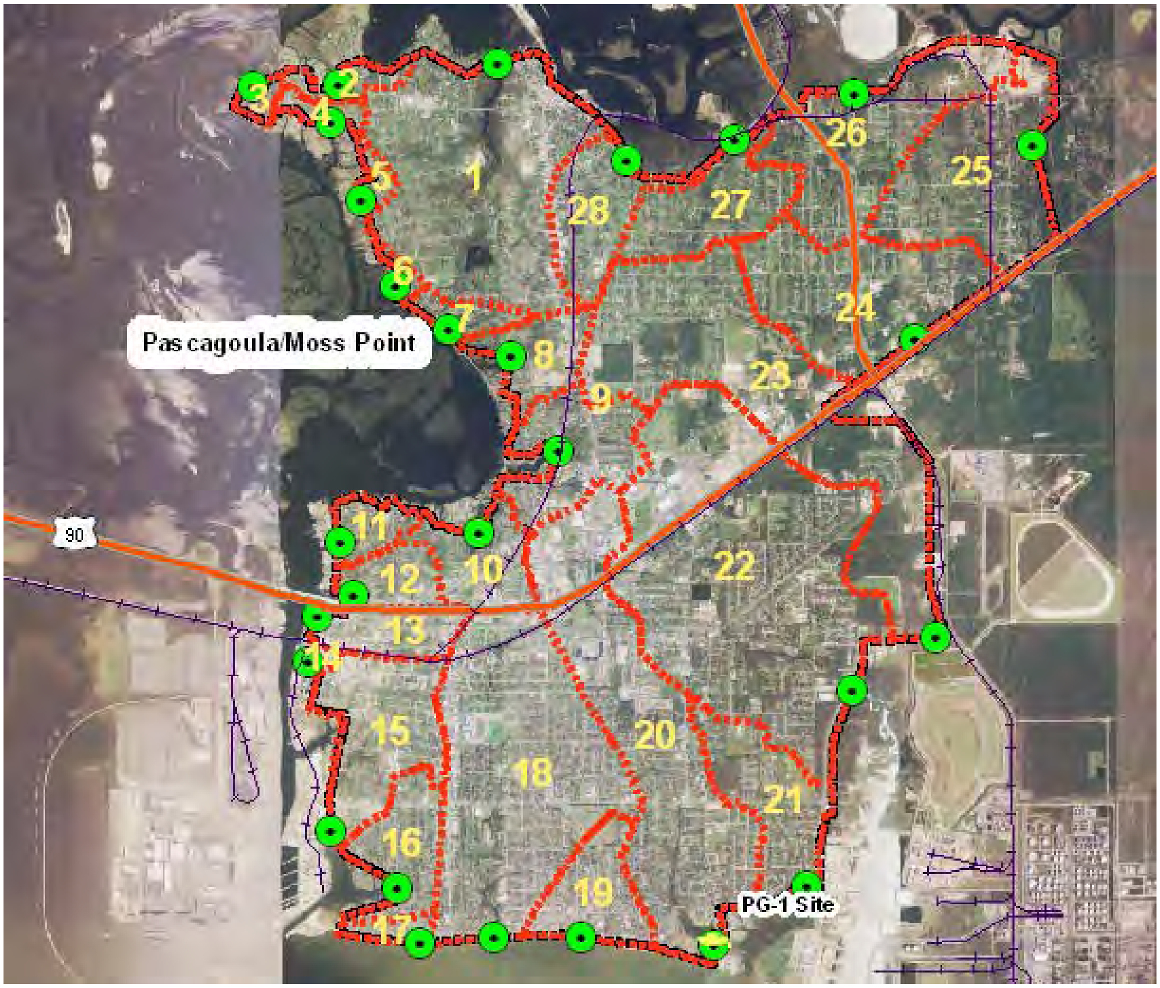

Figure 3.3.12-9. Basic Alignment Pump/Culvert/Sub-basin/Boat Access Site Locations

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo:Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.3.12-10. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown below in Figure 3.3.12-11 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.3.12-11. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment would be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown on Figure 3.3.12-12, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

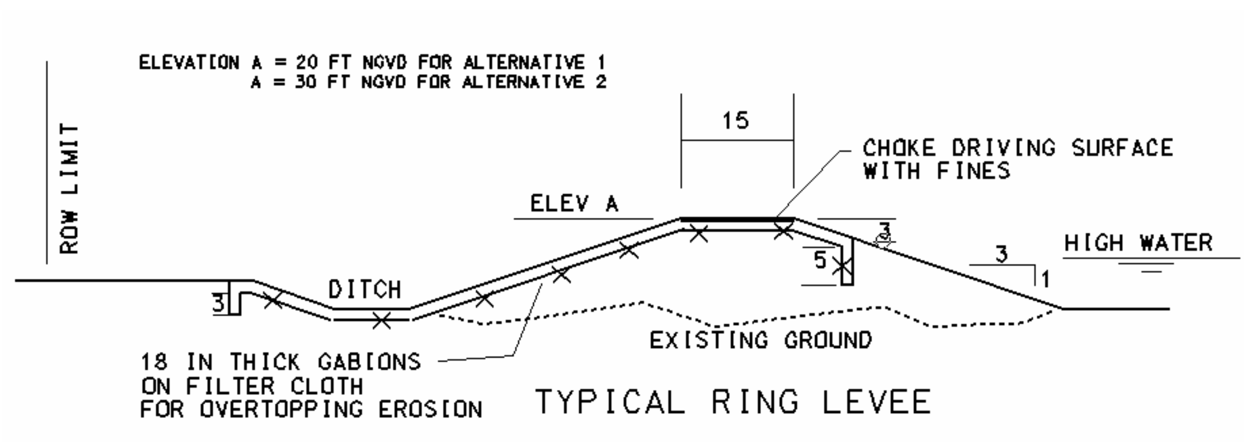

Figure 3.3.12-12. Typical Section at Ring Levee

3.3.12.5.1 Interior Drainage

Drainage on the interior of the ring levee would be collected at the levee and channeled to culverts placed in the levee at the locations shown above in Figure 3.3.12-9. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert in the levee for control in the event the flap gate malfunctions. A typical section is shown below in Figure 3.3.12-13.

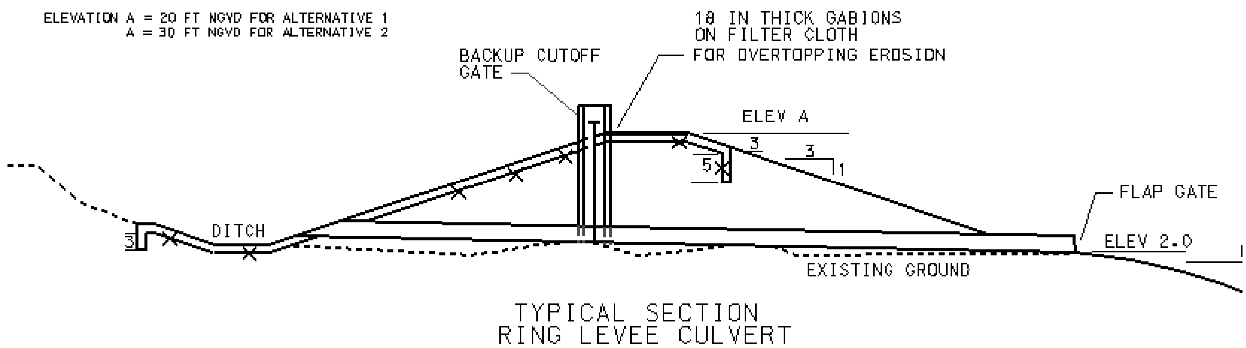

Figure 3.3.12-13. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the ring levee into major sub-basins as shown above in Figure 3.3.12-9 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number. The variation in soil types, hydrologic soil groups, and sub-basins is shown below in Figure 3.3.12-14.

Hydrologic soil group A soils have low runoff potential and high infiltration rates, even with thoroughly wetted and a high rate of water transmission. Hydrologic soil group B soils have moderate infiltration rates when thoroughly wetted and a moderate rate of water transmission. Hydrologic soil group C soils have low infiltration rates when thoroughly wetted and have a low rate of water transmission. Hydrologic soil group C soils have high runoff potential and a very low rate of water transmission.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data are considered beyond the level of detail required for this report.

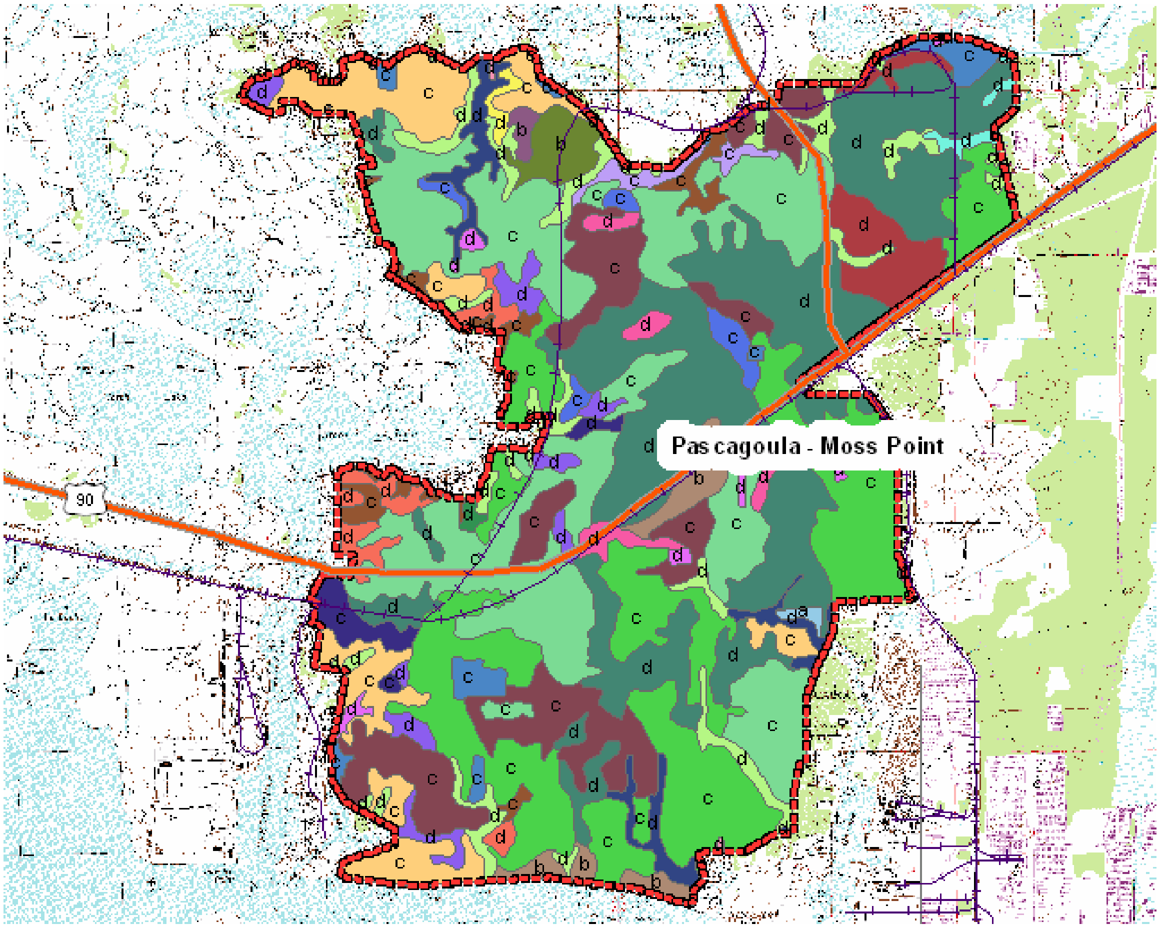

Figure 3.3.12-14. Pascagoula/Moss Point Hydrologic Soil Groups

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report, therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.3.12.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.3.12.5.3 Structural, Mechanical and Electrical

Structural, Mechanical, and Electrical data are presented for culverts, pumping facilities and for boat access sites. The sites are shown above in Figure 3.3.12-9.

3.3.12.5.3.1 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. For this study these were configured as cast-in-place reinforced concrete box structures fitted with flap gates to minimize normal backflows and sluice gates to provide storm closure when needed. The shear number of these structures that would be required throughout the area covered by this study would dictate that an automated system be incorporated whereby the gates could be monitored and operated from some central location within defined districts. Detailed design of these monitoring and operating systems is beyond the scope of this study, however a parametric cost was developed for each site and included in the estimated construction cost for these facilities.

3.3.12.5.3.2 Pumping Facilities Structural

The layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed, along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was added as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet. The lateral extent was estimated at 10 feet per discharge pipe.

3.3.12.5.3.3 Pumping Facilities Mechanical

Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve, and in light of the widespread unavailability of electric power during and immediately after a major storm, it was determined that the pumps should be diesel engine driven.

3.3.12.5.3.4 Pumping Facilities. Electrical

The electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

Because of the number of pumping facilities involved and the need to closely control the pumping operations over a large area, a system of several operation and monitoring stations would be required from which the pumping facilities could be started and their operation monitored during and immediately following a storm event. The detailed design of this monitoring and operation system is beyond the scope of this study, however a parametric estimate of the cost involved in developing and installing such a system was made and included in the estimate of construction costs for these facilities.

3.3.12.5.3.5 Pumping Stations. Flow and Pump Sizes

The design hydraulic heads derived for the 28 facilities included in the Pascagoula-Mosspoint Ring Levee system for the elevation 20 protection level varied from approximately 10 to 20 feet and the corresponding flows required varied from 24,200 to 860,900 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 290 horsepower pump, to one including 10, 54-inch diameter pumps each running at 420 horsepower.

3.3.12.5.3.6 Boat Access Structure

At Site PG-1 the ring levee alignment would cross a moderately sized water course where it is apparent that boats currently traverse the area. (See Figures 3.3.12-9 above and Figure 3.3.12-15 and Table 3.3.12-1, below). To allow continued free boat access to the areas behind the levee this site was fitted with a scaled down adaptation of the larger rising sector gate structure used for the bay barriers at Biloxi and Bay Saint Louis. This structure would, for the most part, be much smaller and lighter than those used in the bays, however it would be substantial. The operation would be similarly critical in time of storm and they would require the same attention from an Operations and Maintenance standpoint as their larger, heavier counterparts.

3.3.12.5.3.7 Boat Access Structure. Mechanical. Option A

The mechanical equipment and operating system for these structures would be similar to those used for the bay barriers, and would include steel gate linkages and hydraulic rams and pivot pins for operation of the gates. Each gate would rotate on large bearings and pivot hubs at the ends of the gate. Various operating hydraulic and lubrication oil systems would also be required. It is estimated that each gate would have a maximum opening/closing time of 15 minutes.

3.3.12.5.3.8 Boat Access Structure. Electrical

Primary electrical power for operating these gates would be provided using dedicated, standard transformers with emergency back-up generators. The electrical load demand at these facilities would be low by comparison to the bay barrier structures. The supplemental generation aspect was considered to be a vital component of the design because of the very high cost of commercial standby power and because commercial electric power would almost certainly be unavailable during and immediately following a storm event.

3.3.12.5.3.9 Mechanical and Electrical. Roadways

At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that all roadways and railways crossing the levee alignment would be retained except where it was very evident that traffic could be combined without undue congestion.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required.

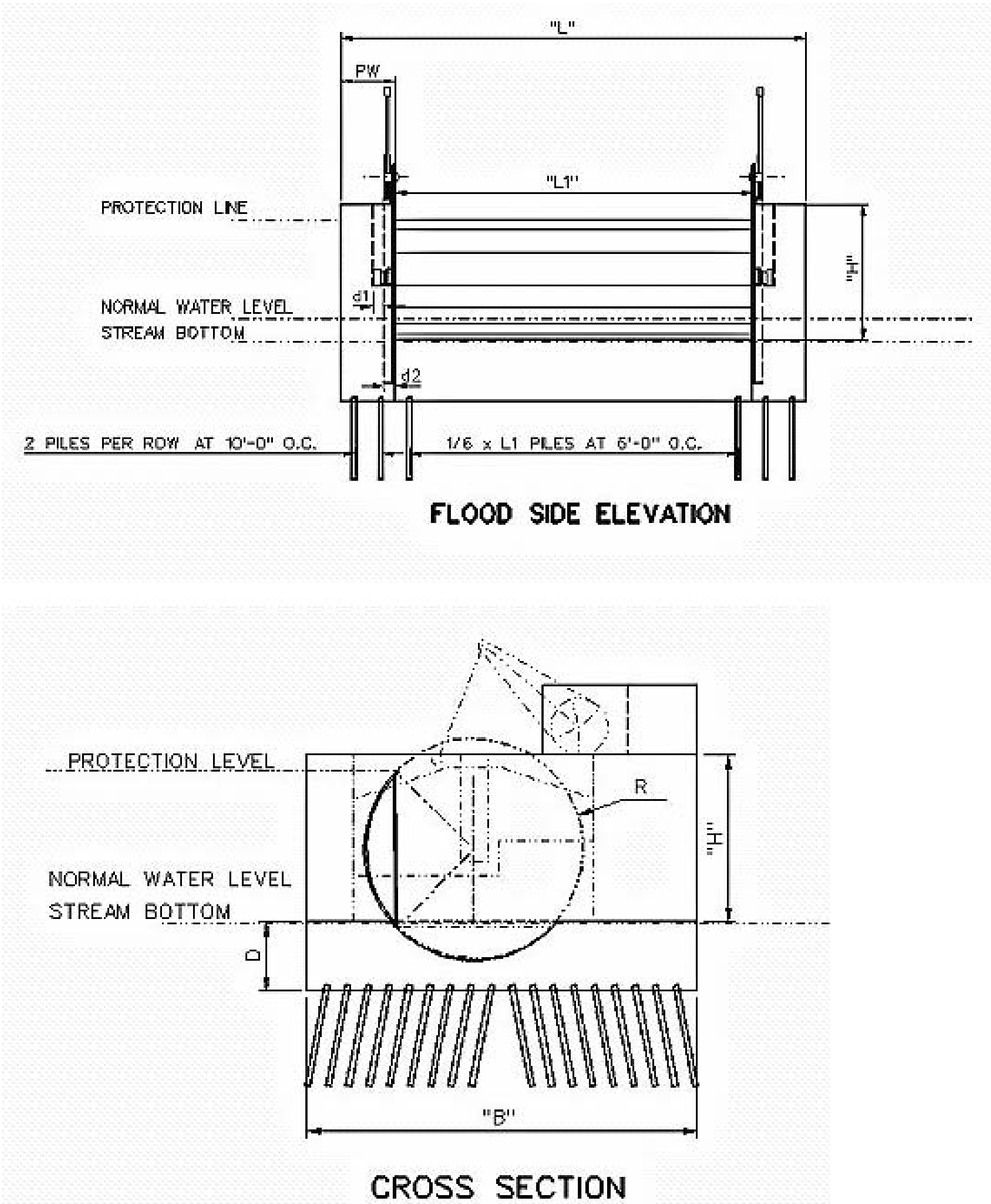

Figure 3.3.12-15. Typical Small Boat Access Structure

Table 3.3.12-1.
Boat Access Structure Dimensional Data by Site

Site Designation	Protection Elevation, ft NAVD88	L1 ft	PW ft	H ft
PG-1	20.0	75	18	29.5

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, in most instances this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

3.3.12.5.3.10 Railways

Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee.

3.3.12.5.3.11 Levee and Roadway/Railway Intersections

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 20, 68 roadway/railway intersections would have to be accommodated. For this study it was estimated that 29 roller gate structures and 35 swing gate structures would be required at the points where roadways would cross the protection line. In addition, 8 railroad gate structures would be required.

3.3.12.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.3.12.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.3.12.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for

Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. Boat access gates and power plants would require this level of security.

3.3.12.5.7 Operation and Maintenance

Operation and maintenance activities for this project will be required on an annual basis. All pumps and gates will be operated to assure proper working order. Debris and shoaled sediment will be removed. Vegetation on the levees will be cut to facilitate inspection and to prevent roots from causing weak levee locations. Rills will be filled and damaged revetment will be repaired. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.3.12.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.3.12.13, Cost Summary. Construction costs for the various options are included in Table 3.3.12-2 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.12-3. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.3.12.5.9 Schedule for Design and Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.3.12.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.12.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.12.6.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.3.12.6.3 Structural, Mechanical and Electrical

These data are the same as that presented for Option A and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The other data for Option B is presented below.

3.3.12.6.3.1 Pumping Stations. Flow and Pump Sizes. Option B

The design hydraulic heads derived for the 28 facilities included in the Pascagoula-Mosspoint Ring Levee system for the elevation 30 protection level varied from approximately 20 to 30 feet and the corresponding flows required varied from 24,200 to 860,900 gallons per minute. The plants thus derived varied in size from a plant having one 42-inch diameter, 475 horsepower pump, to one including 10, 54-inch diameter pumps each running at 775 horsepower.

3.3.12.6.3.2 Levee and Roadway/Railway Intersections. Option B

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 30, 79 roadway intersections would have to be accommodated. For this study it was estimated that 1 roller gate structure and 73 swing gate structures would be required at the points where roadways would cross the protection line. In addition, 5 railroad gate structures would be required.

3.3.12.6.4 HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.3.12.6.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option B are the same as for Option A, above.

3.3.12.6.6 Project Security

The Project Security paragraphs for Option B are the same as for Option A, above.

3.3.12.6.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.3.12.6.8 Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.3.12.6.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.3.12.7 Option C – Washington Ave. Alternate Alignment, Elevation 20 ft NAVD88

This option consists of an earthen levee enclosing an area of 9350 acres around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option A, above, except that is follows Washington Avenue on the south leg of the levee. The alignment is shown below in Figure 3.3.12-16.

Figure 3.3.12-16. Washington Ave Alternate Alignment Pump/Culvert/Sub-basin/Boat Access Site Locations

3.3.12.7.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.3.12.7.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option C are the same as for Option A, above.

3.3.12.7.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the alteration of the levee alignment to roughly follow Washington Avenue. This variance occasioned changes to the pumping requirements and facilities for the sub-basins 16-20 on the south leg of the levee, and alteration of the number of roadway and railroad intersections. This changed data for Option C is presented below.

3.3.12.7.3.1 Pumping Stations. Flow and Pump Sizes. Option C

The design hydraulic heads derived for the facilities included in the Pascagoula-Mosspoint Option C Ring Levee system for the elevation 20 protection level varied from approximately 10 to 20 feet and the corresponding flows required varied from 171,578 to 490,124 gallons per minute. The plants thus derived varied in size from a plant having three 48-inch diameter, 340 horsepower pump, to one including seven 54-inch diameter pumps each running at 290 horsepower.

3.3.12.7.3.2 Levee and Roadway/Railway Intersections. Option C

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 20 and the inclusion of the Washington Avenue alignment, 76 roadway intersections would have to be accommodated. For this study it was estimated that 24 roller gate structures and 108 swing gate structures would be required at the points where roadways would cross the protection line. In addition, 14 railroad gate structures would be required.

3.3.12.7.4 HTRW

The HTRW paragraphs for Option C are the same as for Option A, above.

3.3.12.7.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option C are the same as for Option A, above.

3.3.12.7.6 Project Security

The Project Security paragraphs for Option C are the same as for Option A, above.

3.3.12.7.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option C are the same as for Option A, above.

3.3.12.7.8 Cost Estimate

The Cost Estimate paragraphs for Option C are the same as for Option A, above.

3.3.12.7.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option C are the same as for Option A, above.

3.3.12.8 Option D – Washington. Alternate Alignment, Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option C, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option C is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.12.8.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option C, above, except that the culvert lengths through the levees would be longer.

3.3.12.8.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option D are the same as for Option A, above.

3.3.12.8.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option C is the height of the levee and the resulting differences in the required pumping facilities, number of roadway and railroad intersections, the length of the levee culverts, and the exclusion of the Boat Access Structure. The changed data for Option D is presented below.

3.3.12.8.3.1 Pumping Stations. Flow and Pump Sizes. Option D

The design hydraulic heads derived for the 6 facilities included in the Pascagoula-Mosspoint Ring Levee system for the elevation 30 protection level varied from approximately 20 to 30 feet and the corresponding flows required varied from 171,578 to 490,124 gallons per minute. The plants thus derived varied in size from a plant having three 48-inch diameter, 600 horsepower pumps, to one including five 60-inch diameter pumps each running at 1150 horsepower.

3.3.12.8.3.2 Levee and Roadway/Railway Intersections. Option D

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 30, 87 roadway intersections would have to be accommodated. For this study it was estimated that 1 roller gate structure and 180 swing gate structures would be required at the points where roadways would cross the protection line. In addition, 18 railroad gate structures of varying height would be required.

3.3.12.8.4 HTRW

The HTRW paragraphs for Option D are the same as for Option A, above.

3.3.12.8.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option D are the same as for Option A, above.

3.3.12.8.6 Project Security

The Project Security paragraphs for Option D are the same as for Option A, above.

3.3.12.8.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option D are the same as for Option A, above.

3.3.12.8.8 Cost Estimate

The Cost Estimate paragraphs for Option D are the same as for Option A, above.

3.3.12.8.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option D are the same as for Option A, above.

3.3.12.9 Option E – Moss Point Alternate Alignment, Elevation 20 ft NAVD88

This option consists of an earthen levee enclosing an area of 7535 acres around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option A, above, except that is follows a modified alignment through Moss Point on the north leg of the levee. The alignment is shown below in Figure 3.3.12-17.

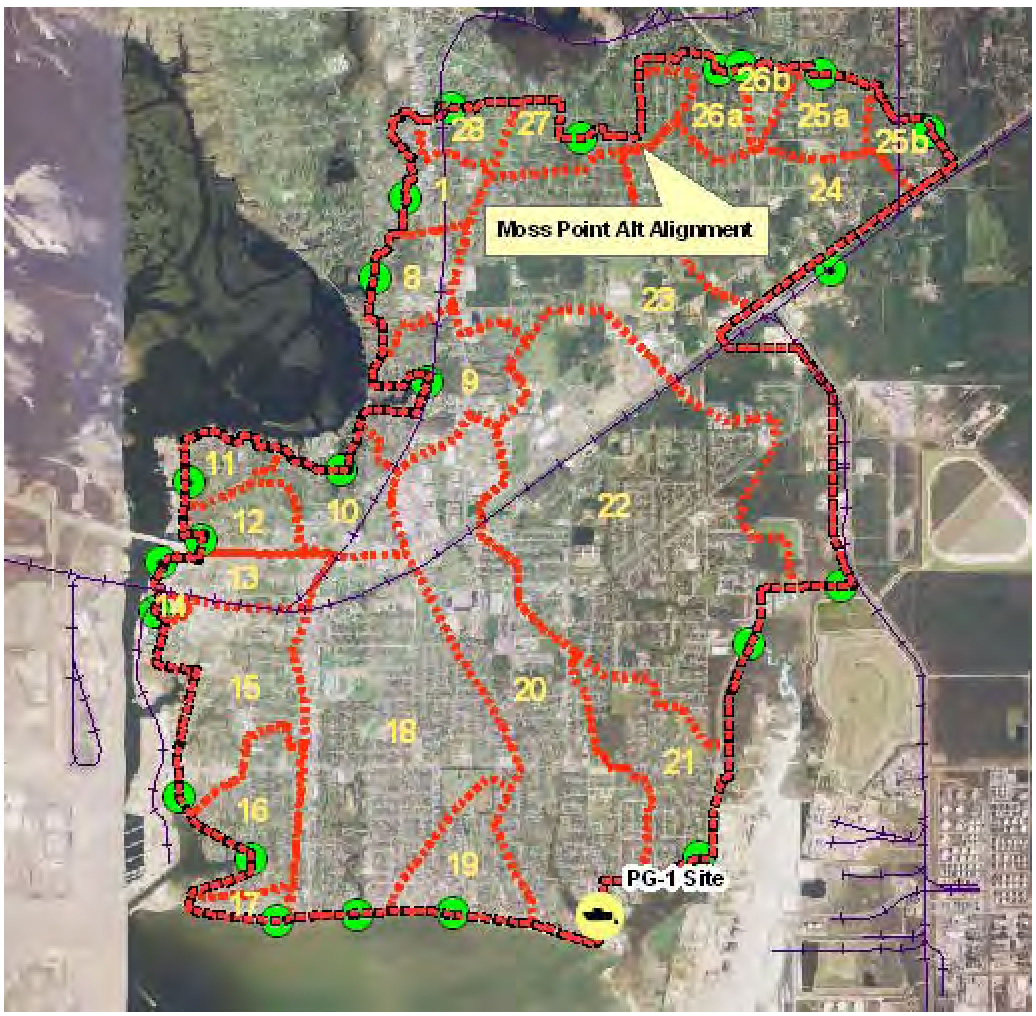

Figure 3.3.12-17. Moss Point Alignment Pump/Culvert/Sub-basin/Boat Access Site Locations

3.3.12.9.1 Interior Drainage

Interior drainage analysis and culvert design methods are the same as those for Option A, above. Culvert/Pump locations are shown in Figure 3.3.12-17, above.

3.3.12.9.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option E are the same as for Option A, above.

3.3.12.9.3 Structural, Mechanical and Electrical

The only difference between the description of this option and preceding description of Option A is the incorporation of the Moss Point Levee with that for Pascagoula and the resulting variance in the pumping requirements and facilities for the sub-basins on the north leg of the levee and the number of roadway and railroad intersections. The changed data for Option E is presented below.

3.3.12.9.3.1 Pumping Stations. Flow and Pump Sizes. Option E

The design hydraulic heads derived for the facilities included in the Pascagoula-Mosspoint Option E Ring Levee system for the elevation 20 protection level varied from approximately 5 to 20 feet and the corresponding flows required varied from 62,549 to 490,083 gallons per minute. The plants thus derived varied in size from a plant having two 36-inch diameter, 125 horsepower pumps, to one including seven 54-inch diameter pumps each running at 290 horsepower.

3.3.12.9.3.2 Levee and Roadway/Railway Intersections. Option E

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 20, 43 roadway intersections would have to be accommodated. For this study it was estimated that 15 roller gate structures and 56 swing gate structures would be required at the points where roadways would cross the protection line. In addition, 10 railroad gate structures would be required.

3.3.12.9.4 HTRW

The HTRW paragraphs for Option E are the same as for Option A, above.

3.3.12.9.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option E are the same as for Option A, above.

3.3.12.9.6 Project Security

The Project Security paragraphs for Option E are the same as for Option A, above.

3.3.12.9.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option E are the same as for Option A, above.

3.3.12.9.8 Cost Estimate

The Cost Estimate paragraphs for Option E are the same as for Option A, above.

3.3.12.9.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option E are the same as for Option A, above.

3.3.12.10 Option F – Moss Point Alternate Alignment, Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option E, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option E is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.12.10.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option E, above, except that the culvert lengths through the levees would be longer.

3.3.12.10.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option F are the same as for Option A, above.

3.3.12.10.3 Structural, Mechanical and Electrical

The primary differences between the description of this option and preceding description of Option A is the incorporation of the Moss Point levee with that for Pascagoula and the increased height of the levee and the resulting changes in the pumping facilities, number of roadway and railroad intersections, and the length of the levee culverts. Culvert length variations are not presented but are incorporated into the cost estimate. The changed data for Option F is presented below.

3.3.12.10.3.1 Pumping Stations. Flow and Pump Sizes. Option F

The design hydraulic heads derived for the 10 facilities included in the Pascagoula-Moss Point Ring Levee system for the elevation 30 protection level varied from approximately 15 to 30 feet and the corresponding flows required varied from 62,549 to 490,083 gallons per minute. The plants thus derived varied in size from a plant having two 36-inch diameter, 250 horsepower pumps, to one including five 60-inch diameter pumps each running at 750 horsepower.

3.3.12.10.3.2 Levee and Roadway/Railway Intersections. Option F

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 30, 75 roadway intersections would have to be accommodated. For this study it was estimated that all of these structures would be swing gates. In addition, seven sites with 14 railroad gate structures would be required.

3.3.12.10.4 HTRW

The HTRW paragraphs for Option F are the same as for Option A, above.

3.3.12.10.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option F are the same as for Option A, above.

3.3.12.10.6 Project Security

The Project Security paragraphs for Option F are the same as for Option A, above.

3.3.12.10.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option F are the same as for Option A, above.

3.3.12.10.8 Cost Estimate

The Cost Estimate paragraphs for Option F are the same as for Option A, above.

3.3.12.10.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option F are the same as for Option A, above.

3.3.12.11 Option G – Combined Washington Ave and Moss Point Alternate Alignments, Elevation 20 ft NAVD88

This option consists of an earthen levee enclosing an area of 7356 acres around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option A, above, except that is follows the same modified alignment along Washington Ave as shown for Options C and D on the south, and the modified alignment in Moss Point as shown for Options E and F along the north leg of the levee. The alignment is shown below in Figure 3.3.12-18.

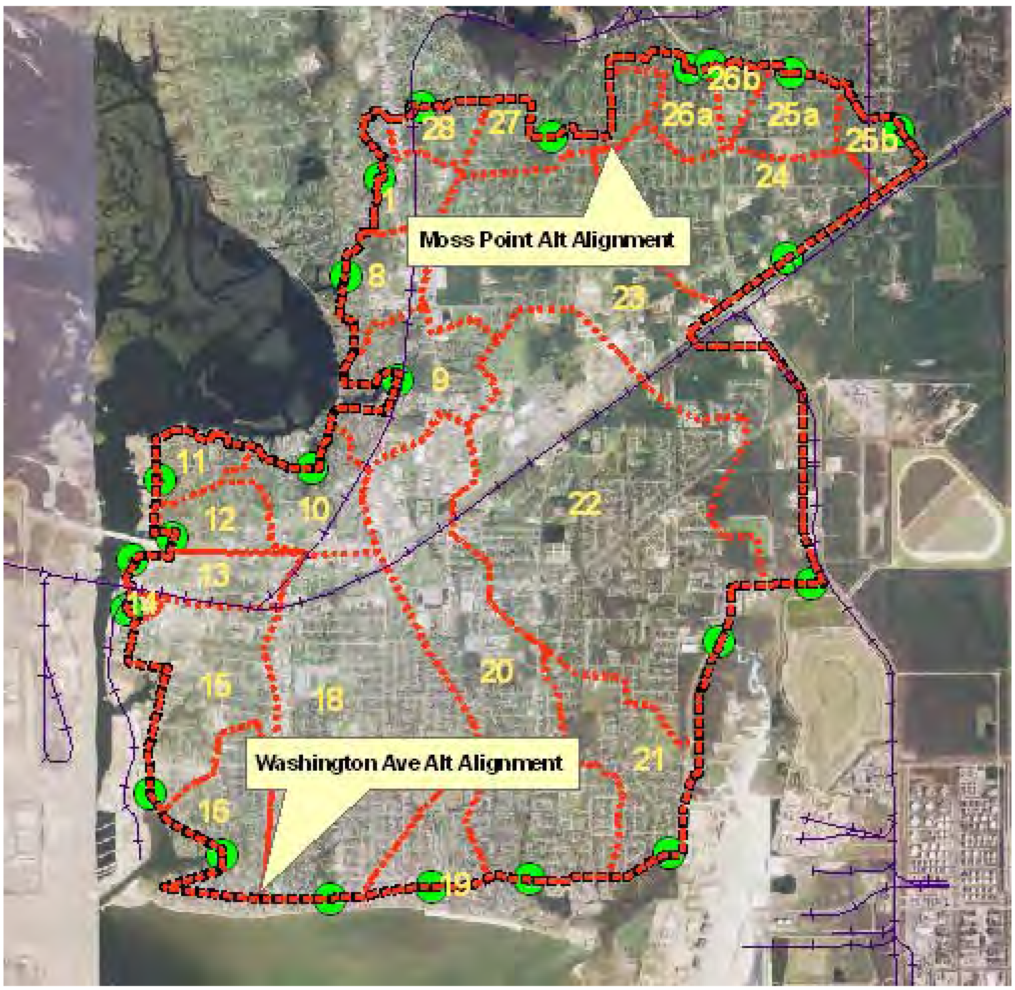

Figure 3.3.12-18. Moss Point Alignment Pump/Culvert/Sub-basin Locations

3.3.12.11.1 Interior Drainage

Interior drainage analysis and culvert design methods are the same as those for Option A, above. Culvert/Pump locations are shown in Figure 3.3.12-18, above.

3.3.12.11.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option G are the same as for Option A, above.

3.3.12.11.3 Structural, Mechanical and Electrical

The primary differences between the description of this option and preceding description of Option A would be the combination of the Pascagoula with Washington Avenue, and the Moss Point alignments resulting in an variation in the pumping requirements and facilities for the sub-basins on the north leg of the levee, number of roadway and railroad intersections, and the omission of the Boat Access Structure south of the revised alignment. The other data for Option G is presented below.

3.3.12.11.3.1 Pumping Stations. Flow and Pump Sizes. Option G

The design hydraulic heads derived for the facilities included in the Pascagoula-Mosspoint Option G Ring Levee system for the elevation 20 protection level varied from approximately 5 to 20 feet and the corresponding flows required varied from 62,388 to 490,083 gallons per minute. The plants thus derived varied in size from a plant having two 36-inch diameter, 125 horsepower pump, to one including seven 54-inch diameter pumps each running at 750 horsepower.

3.3.12.11.3.2 Levee and Roadway/Railway Intersections. Option G

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 20, 48 roadway intersections would have to be accommodated. For this study it was estimated that 15 roller gate structure and 72 swing gate structures would be required at the points where roadways would cross the protection line. In addition, five sites with 10 railroad gate structures would be required.

3.3.12.11.4 HTRW

The HTRW paragraphs for Option G are the same as for Option A, above.

3.3.12.11.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option G are the same as for Option A, above.

3.3.12.11.6 Project Security

The Project Security paragraphs for Option G are the same as for Option A, above.

3.3.12.11.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option G are the same as for Option A, above.

3.3.12.11.8 Cost Estimate

The Cost Estimate paragraphs for Option G are the same as for Option A, above.

3.3.12.11.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option G are the same as for Option A, above.

3.3.12.12 Option H – Combined Washington Ave and Moss Point Alternate Alignment, Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Pascagoula and Moss Point. The alignment of the levee is the same as Option G, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option G is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.3.12.12.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option G, above, except that the culvert lengths through the levees would be longer.

3.3.12.12.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option H are the same as for Option A, above.

3.3.12.12.3 Structural, Mechanical and Electrical

The primary differences between the description of this option and preceding description of Option A are the incorporation of the Pascagoula with Washington Avenue levee, with that for Moss Point, the change in the height of the levee, and the resulting changes in the pumping facilities, number of roadway and railroad intersections, the length of the levee culverts, and the omission of the Boat Access Structure. Culvert length variations are not presented but are incorporated into the cost estimate. The changed data for Option H are presented below.

3.3.12.12.3.1 Pumping Stations. Flow and Pump Sizes. Option H

The design hydraulic heads derived for the 14 facilities included in the Pascagoula-Mosspoint Ring Levee system for the elevation 30 protection level varied from approximately 15 to 30 feet and the corresponding flows required varied from 62,388 to 490,083 gallons per minute. The plants thus derived varied in size from a plant having two 36-inch diameter, 250 horsepower pumps, to one including five 60-inch diameter pumps each running at 1150 horsepower.

3.3.12.12.3.2 Levee and Roadway/Railway Intersections. Option H

With the installation of a ring levee around the Pascagoula-Moss Point areas to elevation 30, 79 roadway intersections would have to be accommodated. For this study it was estimated that all of these would be swing gate structures. Fourteen railroad gate structures would be required.

3.3.12.12.4 HTRW

The HTRW paragraphs for Option H are the same as for Option A, above.

3.3.12.12.5 Construction and Water Control Plan

The Construction and Water Control paragraphs for Option H are the same as for Option A, above.

3.3.12.12.6 Project Security

The Project Security paragraphs for Option H are the same as for Option A, above.

3.3.12.12.7 Operation and Maintenance

The Operation and Maintenance paragraphs for Option H are the same as for Option A, above.

3.3.12.12.8 Cost Estimate

The Cost Estimate paragraphs for Option H are the same as for Option A, above.

3.3.12.12.9 Schedule for Design and Construction

The Schedule for Design and Construction paragraphs for Option H are the same as for Option A, above.

3.3.12.13 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown in Tables 3.3.12-2 and 3.3.12-3, below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.3.12-2.
Jackson Co Pascagoula/Moss Point Ring Levee Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$699,000,000
Option B – Elevation 30 ft NAVD88	$916,000,000
Option C – Elevation 20 ft NAVD88	$671,600,000
Option D – Elevation 30 ft NAVD88	$849,900,000
Option E – Elevation 20 ft NAVD88	$874,400,000
Option F – Elevation 30 ft NAVD88	$1,013,200,000
Option G – Elevation 20 ft NAVD88	$921,400,000
Option H – Elevation 30 ft NAVD88	$1,057,700,000

Table 3.3.12-3.
Jackson Co Pascagoula/Moss Point Ring Levee O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$5,719,000
Option B – Elevation 30 ft NAVD88	$8,309,000
Option C – Elevation 20 ft NAVD88	$4,658,000
Option D – Elevation 30 ft NAVD88	$6,707,000
Option E – Elevation 20 ft NAVD88	$3,761,000
Option F – Elevation 30 ft NAVD88	$5,423,000
Option G – Elevation 20 ft NAVD88	$3,537,000
Option H – Elevation 30 ft NAVD88	$5,197,000

3.3.12.14 References

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3.4 Line of Defense 4 – Inland Barrier and Surge Gates

3.4.1 General

To preserve the shoreline environment as much as possible, a 4th line of defense for very large storms is envisioned that would be inland from the coast. This line of defense would be the highest line and could contain a larger storm surge up to that associated with a "Maximum Possible Intensity" (MPI) hurricane. LOD-4 was be modeled as an infinitely high barrier with the screening storms defining a surge elevation against the barrier. The top elevation could then be defined based on selected protection from a selected screening storm. Storms that will be modeled against this line will vary from a Camille type storm up to the MPI. This alignment would follow the same path as the railway that crosses the state near the coast but not cross either the Pearl River to the west or the Pascagoula River to the east.

In order to protect much of the developed areas around Biloxi and St. Louis Bays, LOD-4 would have to include a structural surge barrier that would also cross the mouth of these bays. These surge barriers, when closed, would prevent storm surge from moving in through the inlets of the bays. The structural barriers across the bays could be similar to designs used in Europe for storm surge protection. While many types of barriers were reviewed, the rising sector design used on the Thames River in London, England was selected. This type of structure would allow the least restriction to natural tidal flow and with gates flush with the natural bottom, provide the least environmental concern.

During initial planning, options were discussed that would provide a LOD-4 line of defense, but not include closing off the bays with surge barriers. Due to the topography and the positions of the bays and river systems, the project team collectively decided that to be effective, LOD-4 had to include a barrier across Biloxi Bay, but that St. Louis Bay could possibly be excluded. The location of Biloxi and Gulfport on a narrow coastal ridge with the Sound to the south and the Back Bay of Biloxi to the north would not allow closure for a levee to higher elevations to the north. This would leave any type of significant defense as a high ring levee or seawall following the shorelines of the sound and the bay, something widely opposed in early public meetings. It would also leave many heavily developed areas around the bay subject to surge from any future storms. Considering this for St. Louis Bay provides a different option. There could be an optional alignment that would extend northward from Long beach to a selected higher elevation. This northern extension would follow the general alignment of Menge Avenue north of the railroad. There are two major drainages that would cross along this alternate path that carry floodwaters from rainfall away from the town of Long Beach. These drainages are canals that extend from the headwaters of Turkey Branch southwestward to drain into St. Louis Bay. These drainages may require large pumping stations to prevent the canals from flooding upstream if they were closed off during hurricanes. This revision of LOD-4 would leave most of the area westward from Long Beach without any type of defense from storm surge including the cities of Pass Christian and Bay St. Louis. The inclusion of a ring levee around Bay St. Louis could be added should planners choose to not close off the bay.

The general alignment of line 4 is envisioned along the path of a railway that crosses the coast of Mississippi. In Harrison County, this pathway is through heavily populated and commercial zones. To the east in Jackson County, a decision was made not to cross the Pascagoula River and associated marshes. To do so would have both technical and environmental concerns. Crossing this major river system would create environmental problems as well as interior flooding. Constructing barriers or levees across the marshes will change the surface water flow, restrict tidal exchange and could alter existing salinity conditions leading to major ecosystem changes. Blocking the rivers with surge gates, even for short periods could cause extensive flooding due to water backing up behind the gates during storms as rain falls inland. This could cause more flooding than the storm surge. The Pascagoula River system is also habitat to the endangered Gulf Sturgeon and any approved construction or modifications in the river would be unlikely.

For these reasons, the first major watershed divide west of the Pascagoula River was selected to turn the barrier north and extend it to a location beyond the extent of the storm surge associated with a MPI event. Similarly to the west in Hancock County, LOD-4 follows the railway to a watershed divide that is located east of the Pearl River where it follows the divide north to the MPI line. Both of these northward extensions will cross the path of Interstate 10 and may dictate some modifications to the highway depending on the selected top elevation of the line.

LOD-4 could also be designed to have roadways, even major highways on top if desired. This line would be the highest defense, but would not protect structures seaward from the larger storms that might overtop Line 3. All facilities seaward of Line 4 would be prone to flooding in a large storm, so flood-proofing would be necessary in this zone. As described prior, this barrier would extend from high ground east of the Pearl River to high ground west of the Pascagoula River for a distance of approximately 57 miles. It would not cross either of these river systems.

Like Line 3, interior drainage behind this barrier must also be considered. The watersheds may be large and large rainfall events would require substantial structures designed to allow the water to drain or be pumped over the structure in a storm.

3.4.1.1 Surge Gates

3.4.1.1.1 Literature Research

As the requirements of the MsCIP project studies were developed it became apparent early on that several massive gate structures would be required to protect the large inlets from tidal surges during larger storm events. Initially it was thought that some adaptation of our customary tainter or vertical lift gate assemblies might serve this purpose, but as the water levels to be resisted and the required length of the structures were developed it became apparent that much more massive construction than we had heretofore experienced would be required. This was further complicated by the need to minimize the visual impact, obstruction to vessel traffic, and normal tidal flow.

Our search for a method of construction that would be efficient and effective while optimizing freedom of tide flow and minimizing visual and physical obstruction under normal conditions, led us to the Netherlands, Italy, Russia, and the River Thames in the United Kingdom, where several very massive and large scale projects of this type have been constructed or are presently in the planning stages.

<u>Oosterscheldt Barrier, Netherlands</u>

The Dutch have fought these coastal flooding battles for centuries and, since the major floods suffered in the middle of the 20th century, have made a concerted effort to protect their land and people from the sea's ravages. As a result of these efforts several large and innovative structures have been constructed by the Dutch, using very specialized construction techniques and involving use of conventional construction materials on a massive scale.

The Eastern Scheldt Barrier (the Oosterscheldt Barrier) completed in 1986/87 effectively enclosed the southwest coastline of Holland and Zeeland protecting some 100,000 people from flooding up to the 1:4000 year storm event. The gate structure is three kilometers long, was constructed in three segments, and consists of 65 reinforced concrete pillars ranging from 30.25 to 38.75 meters high, and weighing approximately 18,000 tons each. The gaps between the piers were filled with massive stones precisely placed to form the lower portion of the cutoff. The cutoff was completed by insertion of massive reinforced concrete upper and lower beams and moveable steel gates. The 62 massive steel gates, each 42 meters wide, are of the vertical lift type, operated by vertical overhead mounted hydraulic rams. In the open (raised) position they are suspended between the piers over the North Sea, and in the closed position they bridge vertically between the upper and lower concrete sill beams. The gates vary in height from 6 to 12 meters. The largest weighs approximately 480 tons and takes 82 minutes to close. These gates were designed for a maximum design head differential of 5 meters. The entire barrier, including the levee and gated portions, was constructed at a total cost to the Dutch Government of approximately $8.7 billion (2005 U.S. price level). The annual cost of operation is approximately 13 million dollars. See Figure 3.4.1.1-1 for a picture of these gates and their intended operation.

(Oostescheldekering, Wikipedia, Internet Encyclopedia; The Delta Project, Ministry of Transport, Public Works, and water Management, The Netherlands)

Figure 3.4.1.1-1. Oosterscheldt Barrier, Netherlands

This type design offers several advantages. Under normal conditions the gates are high and dry leaving the structure exposed for ready access for maintenance. The construction method used included prefabricated pier sections constructed in the dry in a series of below sea level construction yards which were eventually flooded allowing the pier sections to be moved into place using specially made ships, then sunk onto previously prepared stone mattress foundations. No foundation pilings were required. The gates can be completely closed/opened in one hour. In considering application of this design for the MsCIP several disadvantages were identified. The gate and pier structures are always in view, extending above the water's surface, an undesirable feature in the locations under consideration. The design head was relatively low when compared to that which might be encountered along the Mississippi Gulf Coast. Because of the water depth at the site the gate sills were constructed to a point well above the sea floor. This has caused definite and identifiable environmental changes in the lakes and inlets inland of the barrier, one of which is marked reduction in salinity resulting in changes in the species of flora and fauna growing in these areas. These changes have adversely affected the commercial fishing industries and other commercial interests.

Maeslant Barrier, Rotterdam, Netherlands

The latest barrier constructed by the Dutch in their long fight against the North Sea tides was the Maeslant Barrier completed in 1997, near the mouth of the Nieuwe Waterweg, the main access to Rotterdam Harbor. This sea port is the second largest in the world, is surrounded by one of the largest industrial areas in Europe, and is home to approximately 1,000,000 people.

This structural marvel consists of two opposing radial sector gates. Each gate is a watertight steel chamber 22 meters high and 210 meters long mounted on two 237-meter long tubular steel space frame radial arms. These arms extend from the protected side of each gate to massive steel ball joints which are embedded in similarly massive concrete foundations in the banks of the Waterweg. These gates move radially, floating from their moorings in concrete lined pocket channels within the opposite banks of the navigation channel to their "closed" position near the canter of the channel. When the gates are within approximately 1.5 meters of each other they are flooded and sink to rest on a concrete sill in the channel bottom. The entire gate operation is controlled by computers and is linked to a highly sophisticated weather monitoring system. The gate closure operation is automatically triggered when the storm surge of 3 meters above normal sea level is predicted for Rotterdam. The entire closing operation, including ship warning and stopping of navigation traffic, takes approximately 5 hours.

The design criteria for this facility dictated that it provide maximum protection against flooding, maintain optimal channel width and depth for navigation, that its operation require a minimum of interruption to navigation traffic, and that it have no overhead obstructions. This structure and its related protection works were designed to protect against the 1:10,000 year flood event. The total construction cost of the barrier was 450-million Euros (about 500 million dollars) and it took approximately six years to build. See Figure 3.4.1.1-2 for picture of these gates in operation.

(Maeslantkering, Wikipedia, Internet Encyclopedia)

Figure 3.4.1.1-2. Maeslant Barrier, Rotterdam, Netherlands

In assessing the adaptation of this design for the MsCIP flood barrier sites the ease and simplicity of operation were noted as plusses. Also the linkage between the barrier operation and the weather monitoring system would be of great value in our area of concern. However, it was also noted that the width of the water opening at both Biloxi and Saint Louis Bays is substantially greater than that required for the Maeslant site. Also, the hydraulic head for which the structure was designed is significantly less than that which would be experienced along the Mississippi Gulf Coast. While the opening width could possibly be restricted using finger dikes and pass-through culverts to maintain the natural ebb and flow of tide water, this would drastically change the appearance of the bay inlets and might greatly restrict the seaward view from the land side. These factors coupled with the requirement that the barrier be designed to withstand considerably greater hydraulic loading than is seen at the Maeslant site, were viewed as great disadvantages to the use of this type of barrier for the MsCIP sites.

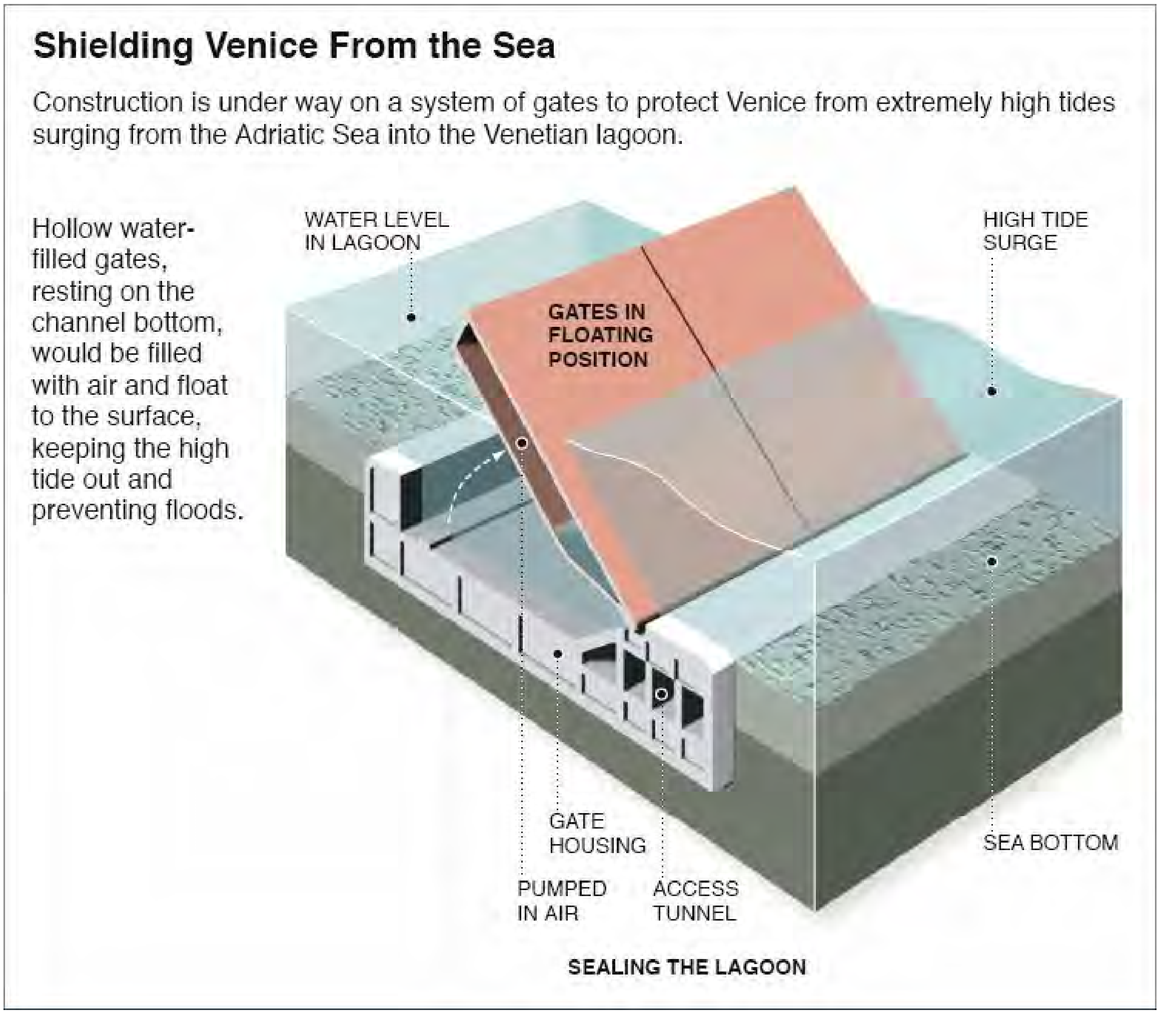
<u>Venice Lagoons</u>

The work done pursuant to addressing flooding problems in Venice, Italy was also cursorily investigated for possible application to the Mississippi Gulf Coast study. This work is still in the investigation and design stages, thus no actual construction details were available. This work would involve the use of "tilt-up" tide gates which would be placed across the lagoon inlets in a string as defense against higher than usual tides. As envisioned for the Venice application these gates would consist of closed hollow chamber gates attached to foundation structures along their seaward edge with hinges. These gates would normally rest filled with water in structure recesses in the sea bottom., They would be raised only when the higher tides are forecast, by injection of air into the hollow gate chambers thus causing the gates to float and hinge upward into their closed position.

The gates designed for the Venice application consisted of 79 separate gate leaves each 20 meters wide providing a total protected length of 1,580 meters configured in three separate groupings. The gates are approximately 30 meters high and were made to retain tides of up to 1.1 meters (approximately 3.6 feet) higher than normal. Further investigation revealed that, as designed, the hinge attachment is the only point of attachment of the gate leaves to the foundation or other structure. The cost estimated for the "Mobile Gates" for the Venice Lagoons in 2004 was approximately $2.7 billion. See Figure 3.4.1.1-3 for graphic depiction of these gates and their intended operation.

(NOVA, <u>Sinking City of Venice</u>, PBS, Internet Transcript; <u>Venice could provide gateway to 21st century flood control method</u>, Denise Brehm, Massachusetts Institute of Technology, 2002, Internet Article)

In order to be functional in a high head situation with gates wide enough to fulfill other project objectives, the gates and structure would have to be designed to resist high lateral loads. The gates would likely need to bear on the structure at the ends of each gate, and the foundations would have to be designed to resist the very large hydraulic loads anticipated. While this design method could possibly be developed to fulfill the needs of our structures, this would have taken considerable work and computation to ascertain the required structure configuration and requirements, much more than the scope of this effort would afford.



Source: Venice turns to the Futureto Rescue its Past, Elisabeth Rosenthal, WITS ARENA

Figure 3.4.1.1-3. Venice Lagoons Flood Barrier, Venice Italy

<u>Thames River Barrier</u>

The Thames River Barrier, Figure 3.4.1.1-4 and 3.4.1.1-5, was constructed during the 1980’s to protect portions of historic London and the surrounding area from tidal flooding. At this site there is a naturally wide variation in the “spring tides” resulting in frequent very high tides, the maximum observed to date being +3.2 meters (i.e. 3.2 meters above the normal tide influenced water level). Also at this site storm surges of as much as +3.66 meters have been experienced. In the event that a storm surge equivalent to the maximum experienced to date and a very high spring tide were to occur at the same time, the water level could conceivably reach as much as +6.86 meters at this site. Based on this possibility, the top of the gates at the Thames River barrier was set at +6.9 meters. This elevation is sufficient to fully contain the 100-year flood event which would yield a water elevation of approximately +5.5 meters. The design flood event was estimated as being the 2000-year flood.

Figure 3.4.1.1-4. Thames River Barrier, Sea Side View

Figure 3.4.1.1-5. Thames River Barrier, Aerial Right Bank View

The Barrier constructed includes a series of reinforced concrete piers and sills, supporting massive steel gates. Each main pier is 11 meters wide and extends to a point slightly above the top of the gates, with the operating machinery and machinery housings mounted atop each pier. Protective and decorative machinery housings were constructed consisting of large curved coverings made of wood and clad with stainless steel. The lowest pier foundations were sunk some 17 meters into the chalk beneath the river bottom.

The barrier includes four main navigation openings measuring 61 meters (approximately 200 feet) in width and two 31.5 meter (approximately 103-foot) openings for passage of smaller vessels. Each of these openings is fitted with a rising sector gate. To allow for free water flow for practically the full width of the river, four more 31.5 meter openings were included each having a falling radial gate, similar to the tainter type gates common to our inland waterway control structures.

The rising sector gates are hollow stainless steel structures with the downriver side curved. Each gate is mounted at either end to large steel disks giving the entire gate structure the appearance of a cut-away cylinder. The gates are supported on trunnion shafts which rotate in bearings mounted in the piers. They are operated by means of reversible hydraulic rams and operating arms mounted on the top of the piers. Under normal conditions the gates lie flat in curved concrete sill recesses in the river bed. Each can be operated upward and stopped at four positions, partially closed (1/8 turn of the disk upward), fully closed (1/4 turn of the disk upward), underspill position (3/8 turn of the disk upward), and maintenance position (1/2 turn of the disk upward). To facilitate operation of the gate the interior of each gate chamber is evacuated of water resulting in a partially buoyant structure.

The sills were set at elevation -9.25 meters and the top of the gates in the fully closed position is +6.9 meters (mean sea level), for an overall protection height of 16.15 meters (approximately 53 feet). The design head for these structures was 6.9 meters (approximately 22.6 feet).

The facilities are operated from a Control Tower located on one bank of the river with a backup control room on the opposite bank. Two service tunnels pass through the foundation of the barrier beneath the river to connect between the two control rooms and to provide power and other utility service access to each pier. In case of extreme emergency each gate can be operated from the individual pier engine rooms. Operating power is provided by three 1.5 MW on-site power generating units, with backup connection to the local electrical grid.

Since its commissioning the Thames River Barrier has been operated 4 to 5 times per year, for a total of 276 times as of 29 April 2002. Each closing cycle takes approximately 15 minutes, though the operation time is greatly extended because of the coordination required with operation of the port facilities.

The Thames River Barrier was constructed between 1972 and 1982 and was formally opened in 1984. The total project construction cost was approximately $760 million. The annual operating and maintenance cost for the Barrier and appurtenant facilities is approximately $13 million.

(Flood London, Thames Barrier: History, Technical Specifications, Why The Barrier is Too Small, Internet articles; Thames Region – Operating the Barrier, Environmental Agency, 2007, Internet Article)

In considering the rising sector gate design for application to the MsCIP barrier structures several points of advantage were identified. Under normal conditions the gates rest out of view at river bottom level. This is appealing in that it would offer a minimum of obstruction to view, to tidal ebb and flow, and to navigation through the structure. The piers, while substantial, are placed wide enough apart that they should be no more obtrusive than the existing bridge structures. The speed of operation would minimize the time the gates would be required to be in place before and after a storm event, and the fact that the gates can be rotated to a full up position for maintenance completely in the dry without installation of unwatering devices or dismantling of the structure is a great maintenance advantage. The maintenance aspect is further enhanced by the fact that the gate surface material is all stainless steel.

Readily observable disadvantages or questionable considerations include the very high construction cost, the relatively small design head required at the Thames River installation as compared to those for the MsCIP sites, the considerably weaker foundation materials existing at the Mississippi Gulf Coast sites, and the relative lengths of the barrier structures required for the MsCIP project sites compared to the Thames River site.

3.4.1.1.2 Design Rationale

The approach to selection of a structural model upon which to base our general design for the MsCIP surge barrier structures was governed by certain basic assumptions and basic criteria:

- The structure must, as completely as possible, block the water surge resulting from the design storm;
- It must be as unobtrusive to view from the sound side or the bay side as possible;
- It must not appreciably alter the natural ebb and flow of water from the Mississippi Sound into/out of the bay areas to be protected;
- It must not appreciably alter the existing navigation of the affected waters by commercial and pleasure craft.

After studying the facilities described above and assessing the features offered by each design approach, along with the associated advantages and disadvantages, it was decided to use the Thames River Barrier model as the basis for the cursory layout and design required for the surge barriers at Biloxi and Saint Louis Bays. A structure layout was made for each bay crossing based on available sounding and water surface information. Uniformity of structure height, gate bay width, and end treatment were used so that one single design might be adapted to each bay crossing.

Preliminary gate designs were made using © STAAD computer modeling, and applying the water pressure and wave action forces based on the prescribed protection levels. Various gate heights were used for each design, as dictated by the protection level under consideration and the configuration of the bay bottom along the route of the surge barrier.

Trial designs were made based on the maximum prescribed protection level and using a 200-foot wide centerline to centerline of pier gate bay and a pier width of 28 feet. The resulting 172-foot wide gates proved to be much too massive, requiring the use of very large structural shapes, very thick covering plate elements, and very closely spaced stiffening frames within the gate proper. The required gate operating disks would have been similarly massive using the 200-foot bay width. This difference form the Thames River gates was primarily caused by the much greater design head possible at the MsCIP sites, amounting to 40 feet at the MsCIP sites, as opposed to approximately 23 feet for the Thames River Site.

The gates were reconfigured using a 160-foot center to center of pier spacing and retaining the 28-foot pier width, resulting in a gate clear width of 132 feet. These gates appeared to be much more reasonable, the framing members required being in the mid range of structural shapes available. It should be noted that these design computations were made purely to obtain rough materials weights upon which to base construction cost estimates. Deflections were not checked, and member connections were not designed. In the event that these facilities were to be designed for construction much more work would be required to bring these gate structures to final design. However, the gate structures arrived at through this effort should provide a good estimate of the materials that would be required to construct such structures.

Once the gate structures were cursorily designed, concrete pier and sill monoliths were laid out and iterative static stability analyses were made to arrive at a structure that would be stable under the applied loading. The foundation bearing pressures resulting from these analyses were above that deemed acceptable for the materials likely to be encountered at these sites. Therefore, as a final design measure, these monoliths were fitted with an array of foundation piles. These piles were battered to resist both vertical and horizontal loading. The resulting materials data are summarized in the tables below, for the various protection levels and resulting monolith configurations.

3.4.1.2 Culverts

As any flood barrier is constructed the natural groundwater runoff would be inhibited. In order to maintain the natural runoff patterns culverts would be inserted through the protection line at appropriate locations. These features would be equipped with gates to provide for closure during extreme storms. The shear number of these structures required throughout the area covered by this study could dictate that some automated system be incorporated whereby the gates could be operated from a series of central locations. From each control point the culverts could be monitored and the sluice gates operated to close off the culverts. Multiple flood protection districts would be set up all along each protection line, each coordinating its efforts with all others.

3.4.1.3 Pumping Stations

The stoppage of normal runoff during storm events would dictate that some means be included by which to evacuate groundwater from behind the protection line during such events. This would be done using pumping stations located at appropriate points along each protection line as described below.

Mechanical. Vertical shaft pumps were used for all of the pumping facilities. Preliminary mechanical design of the required pumping equipment was made by adaptation of manufacturer’s stock pumping equipment to approximate hydraulic head and flow data developed for each pumping location. This data was coordinated with a pump manufacturer who supplied a cross check of the pump selections and cost data for use in preparation of project construction cost estimates. In consideration of the primary purpose which this equipment would serve and in light of the widespread unavailability of electric power during and immediately after a major storm it was determined that the pumps should be diesel engine driven. Each engine would be battery started through activation of a float switch and the start-up of the engines would be properly timed to accommodate variations in required pumping volume.

Structural layout of each pumping facility was made in conformance with Corp of Engineers Guidance document EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations. The basic plant dimensions for each site were set using approximate dimensions derived based on specific pump data (pump impeller diameter, pump bell bottom clearance, etc.). Each facility was roughly fitted to its site using existing ground elevations taken from available mapping and height of levee data. In every case the top of the pump floor was required to be above the 100 year flood elevation. Nominal sidewall and sump and pump floor thicknesses were assumed along with wall and roof thicknesses for the pump room enclosure. Using these basic dimensions and the preliminary number and size of pumping units determined for each site, the overall plant footprint and elevations were set and quantities of basic construction materials computed. The pumping plants were configured, to the greatest extent possible with the data provided, to provide multiple pumps at each site.

Discharge piping for each plant was estimated using over the levee piping with one pipe per pumping unit. For estimating purposes the piping was sized to match the pump diameter. Each pipe was extended approximately 25 feet beyond the toe of the embankment on the discharge end to allow for energy dissipation features to be incorporated into the pipe discharge.

At the discharge end of the piping a heavy mat of grouted riprap was included as protection for the levee slope and immediately adjacent area. In each case the 4-foot deep stone mat was estimated as extending 30 feet up the levee slope and 50 feet out from the levee toe for a total width of 80 feet.

Electrical design for these facilities would consist primarily of providing station power for the facilities. For each of the sites this would include installation of Power Poles, Cable, Power Pole Terminations, miscellaneous electrical appurtenances, and an Electrical 30 kW Diesel Generator Set for backup power.

3.4.1.4 Levee and Roadway/Railway Intersections

Roadways. At each point where a roadway crosses the protection line the decision must be made whether to maintain this artery and adapt the protection line to accommodate it, or to terminate the artery at the protection line and divert traffic to cross the protection line at another location. For this study it was assumed that the majority of roadways and all railways crossing the levee alignment would be retained.

Once the decision has been made to retain a particular roadway, it must then be determined how best to configure the artery to conduct traffic across the protection line. The simplest means of passing roadway traffic is to ramp the roadway over the protection line. This alternative is not always viable because of severe right-of-way restraints caused by extreme levee height, urban congestion, etc. In such instances other methods can be used including partial ramping in combination with low profile roller gates. In more restricted areas full height gates which would leave the roadway virtually unaltered might be preferable, even though this alternative would usually be more costly than ramping. In some extreme circumstances where high levees are required to pass through very congested areas, installation of tunnels with closure gates may be required. See Figures 3.4.1.4-1 and 3.4.1.4-2 for geometric plan representations of typical types of roadway crossing structures. All gates up to and including 9 feet high would be roller gates. All above 9 feet high would be dual leaf swing gates.

Some economy could probably be achieved in this effort by combining smaller arteries and passing traffic through the protection line in fewer locations. However, this would involve detailed traffic routing studies and designs that are beyond the scope of this effort. These studies would be included in the next phase of the development of these options, should such be warranted.

Railways. Because of the extreme gradient restrictions necessarily placed on railway construction, it is practically never acceptable to elevate a railway up and over a levee. Therefore, the available alternatives would include gated pass through structures or much more expensive tunnel structures. Because of the vertical clearance requirements of railroad traffic all railroad pass through structures for this study were configured having vertical walls on either side of the railway with double swing gates extending to the full height of the levee. See Figure 3.4.1.4-3 for geometric plan representation of railroad crossing structures. All railroad gates were assumed to be dual leaf swing gates extending to the full height of levee.

3.4.1.5 Dedicated Flood Barriers

At certain locations there exist properties of vital government interest, extreme historic value, or vital emergency response value in areas where the city congestion would preclude use of levee structures to protect them. As a matter of prudent design these facilities should be removed from the danger zone to a point behind the protection line and where this is possible, this option was followed. However, there are a few instances where removal to a protected area is not desired or expedient. In these instances other structural protection measures would be used as determined by the height of protection required. Generally this protection has been provided using reinforced concrete Tee Walls with sufficient pass through gates to maintain usefulness of the facilities during normal times.

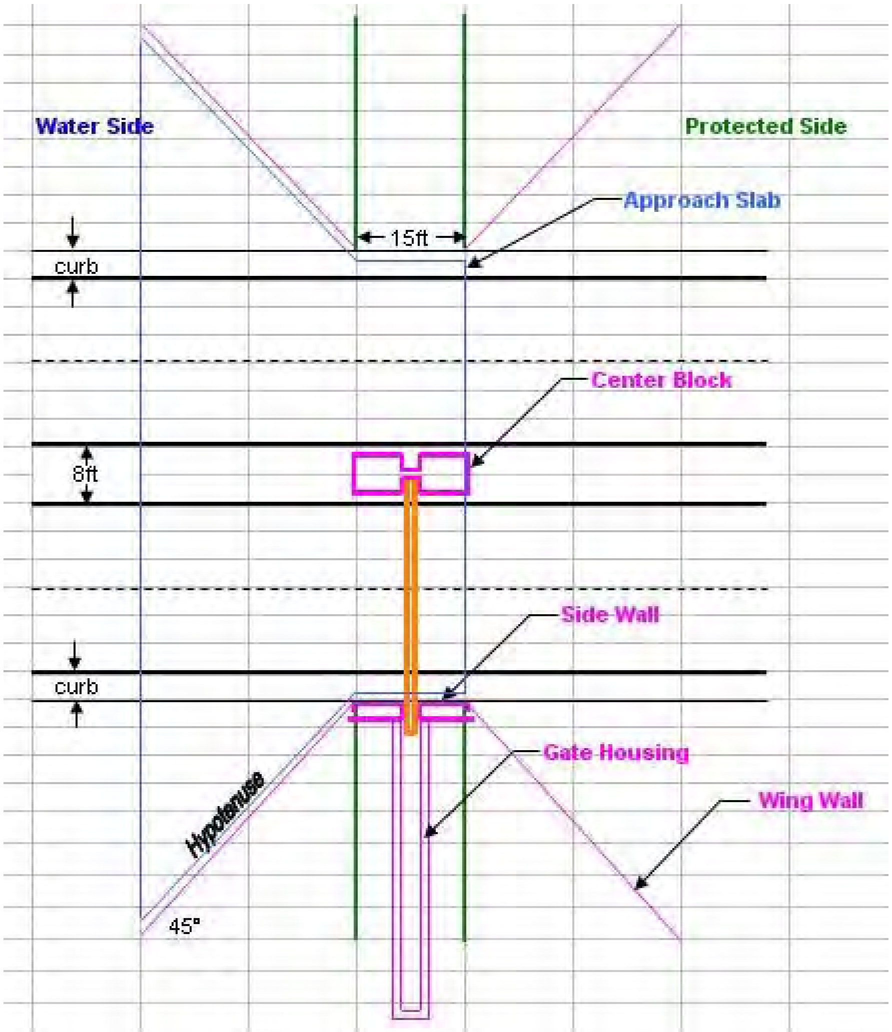


Figure 3.4.1.4-1. Crossings Under 9ft (two lane gate shown; gate and structure would be mirrored to provide for four-lane highway)

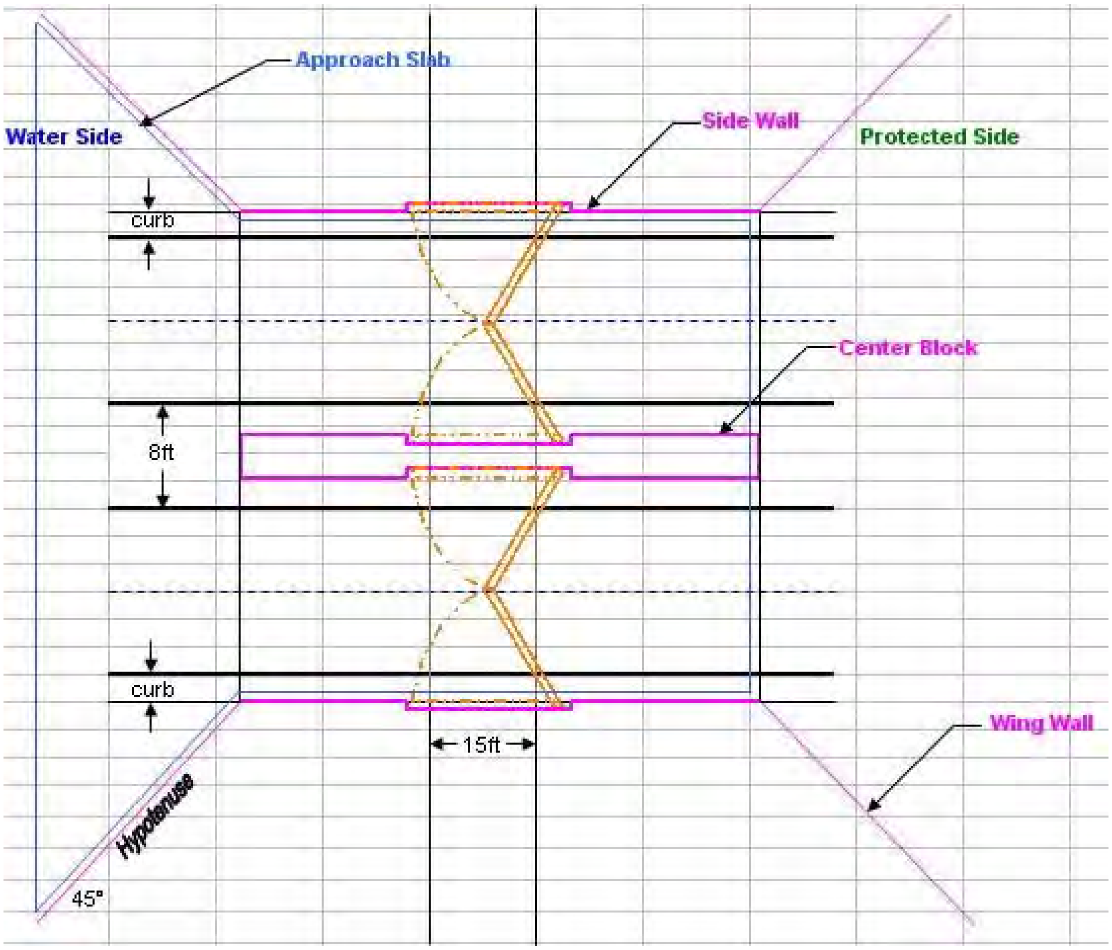


Figure 3.4.1.4-2. Crossing Over 9ft

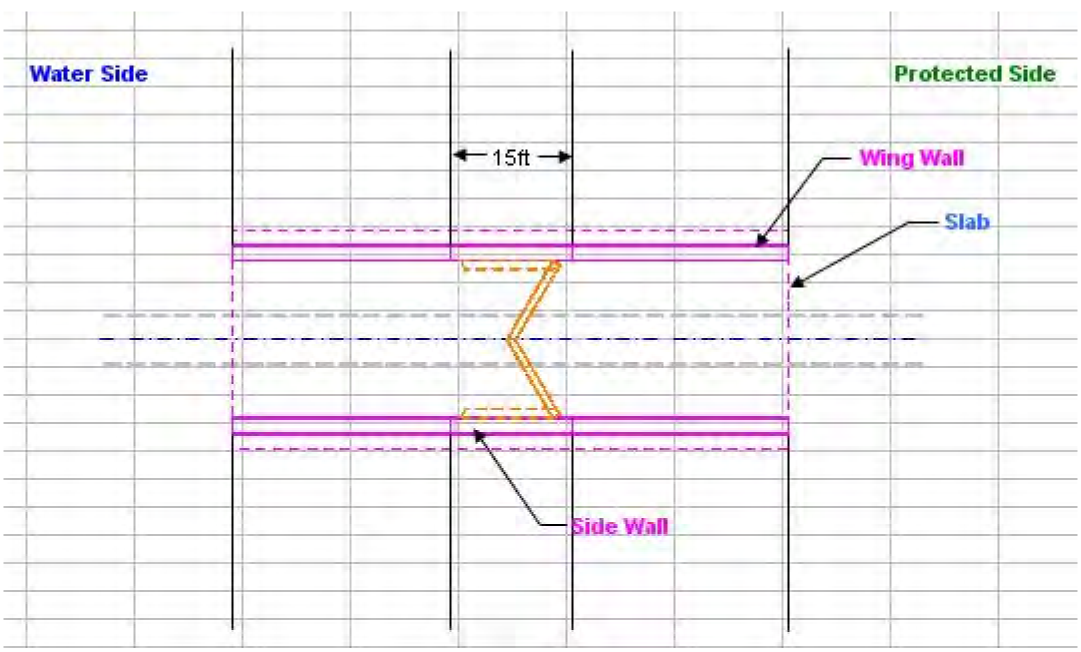


Figure 3.4.1.4-3. Railroad Crossings

3.4.1.6 Operation and Maintenance

3.4.1.6.1 Levee

All levees will require periodic maintenance efforts to include mowing of surface grasses, monitoring of any surface erosion and filling of any resulting cavities. The levees will be periodically monitored for any evidence of subsidence, slope instabilities or seepage.

3.4.1.6.2 Culverts

All culverts penetrating the levee system would have to be periodically and regularly inspected for damage, overgrowth, and sedimentation. The culvert intake and outfall areas would require periodic clearing of vegetation and debris and the surrounding levee slopes and overbanks would have to be kept free of erosion.

The gates and operating mechanisms at each culvert would also require periodic inspection and operation to assure their operability. As planned for this study, all of the culvert gates would be remotely operated. Therefore the periodic maintenance would also cover checks and fine tuning of the remote monitoring and control system. These facilities would require a staff of mechanics and technicians capable of maintaining all mechanical and electrical components in proper working order.

3.4.1.6.3 Pumping Stations

Maintenance of the pumping facilities would require all of the normal civil maintenance activities including clearing of impoundment and outfall areas and general housekeeping activities designed to maintain a workable plant. In addition, the pumps themselves would require periodic inspection and maintenance in keeping with the pump and pump driver manufacturers' warranty requirements. Such requirements would also dictate that each pumping unit be exercised for a minimum duration and a certain number of times during each year. This may pose some degree of difficulty for some of the plants since some were designed almost totally to respond to flood situations. During normal times there may be insufficient inflow to support operation of the pumps, even with the adjacent culverts closed and the normal runoffs collected at the pumps. This difficulty would be addressed in detail in any future study and design work that might be undertaken to refine this system.

3.4.1.6.4 Levee and Roadway/Railway Intersections

These features would require all of the civil maintenance required at the other structures but would in general be more accessible being located along traveled ways. A possible exception to this would be the railway crossings, however these are relatively few and would likely be maintained by railroad personnel. At each of these sites the gates would require lubrication and operation and the gate seals would require periodic inspection and renewal. The gates would be manually operated and would require close coordination with local traffic authorities when any gate movement might be planned.

3.4.1.7 Physical Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study.

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied building and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.1.8 References

(Oostescheldekering, Wikipedia, Internet Encyclopedia; The Delta Project, Ministry of Transport, Public Works, and water Management, The Netherlands)

(Maeslantkering, Wikipedia, Internet Encyclopedia)

(NOVA, Sinking City of Venice, PBS, Internet Transcript; Venice could provide gateway to 21st century flood control method, Denise Brehm, Massachusetts Institute of Technology, 2002, Internet Article)

(Flood London, Thames Barrier: History, Technical Specifications, Why The Barrier is Too Small, Internet articles; Thames Region – Operating the Barrier, Environmental Agency, 2007, Internet Article)

EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations

3.4.2 Hancock County Inland Barrier

3.4.2.1 General

Several high density residential and business areas are located in Hancock County. These are subject to damage from storm surges associated with hurricanes. Earthen levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88 and 40 ft NAVD88. The top width was assumed 15 ft with side-slopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Storm surge gates across St Louis are also included to prevent flooding from hurricanes. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center's (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented elsewhere in this report.

3.4.2.2 Location

The location of the levee in Hancock County is shown in Figures 3.4.2-1 through 3.4.2-4 parallel to the CSX Railroad and the coast and turning northward across I-10 to tie into the corresponding elevation.

3.4.2.3 Existing Conditions

Hancock County is located on the west side of the Mississippi coast of Mississippi Sound. The main residential and business area is at Bay St Louis and Waveland. Ground elevations over the areas behind the levee vary between elevations 10-20 ft NAVD88 at low areas to as low as 5 ft NAVD88 in the Shoreline Park area. The drains to the south along the coast to Mississippi Sound, to the north and east to St Louis Bay, and on the far west to Pearl River. The 4-ft(blue), 8-ft(Dark green), 12-ft(light green), 16-ft(brown), 20-ft(pink), and 24-ft(purple) ground contour lines are shown in Figure 3.4.2-5.

Figure 3.4.2-1. Vicinity Map Hancock County, MS

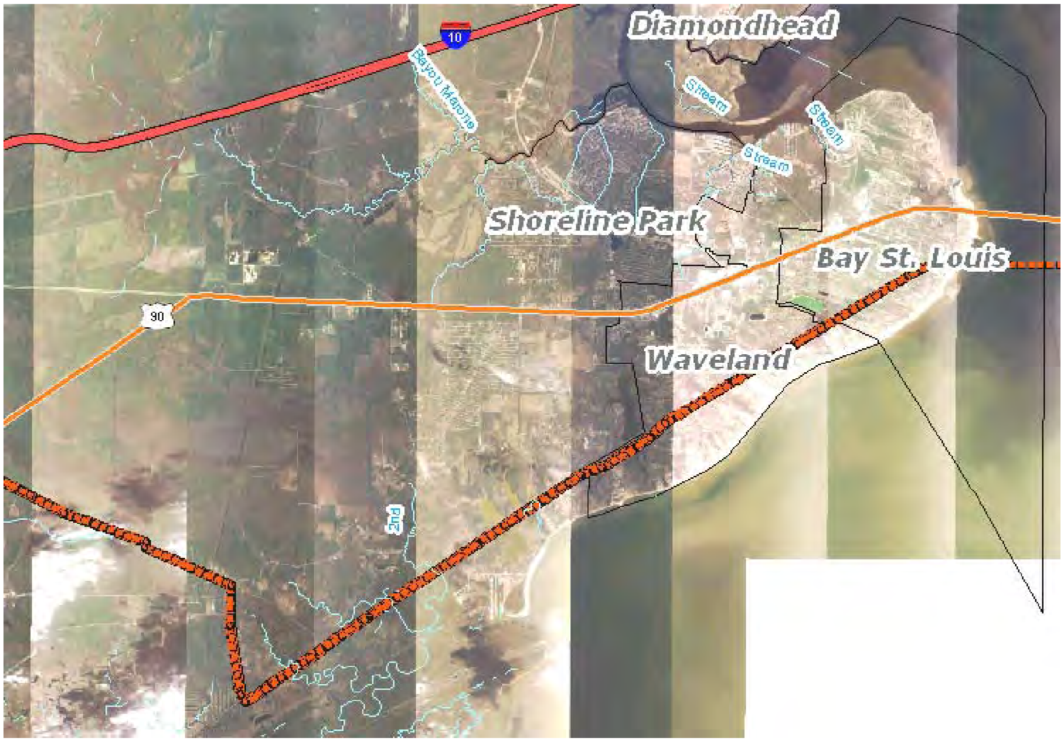


Figure 3.4.2-2. Hancock County Inland Barrier

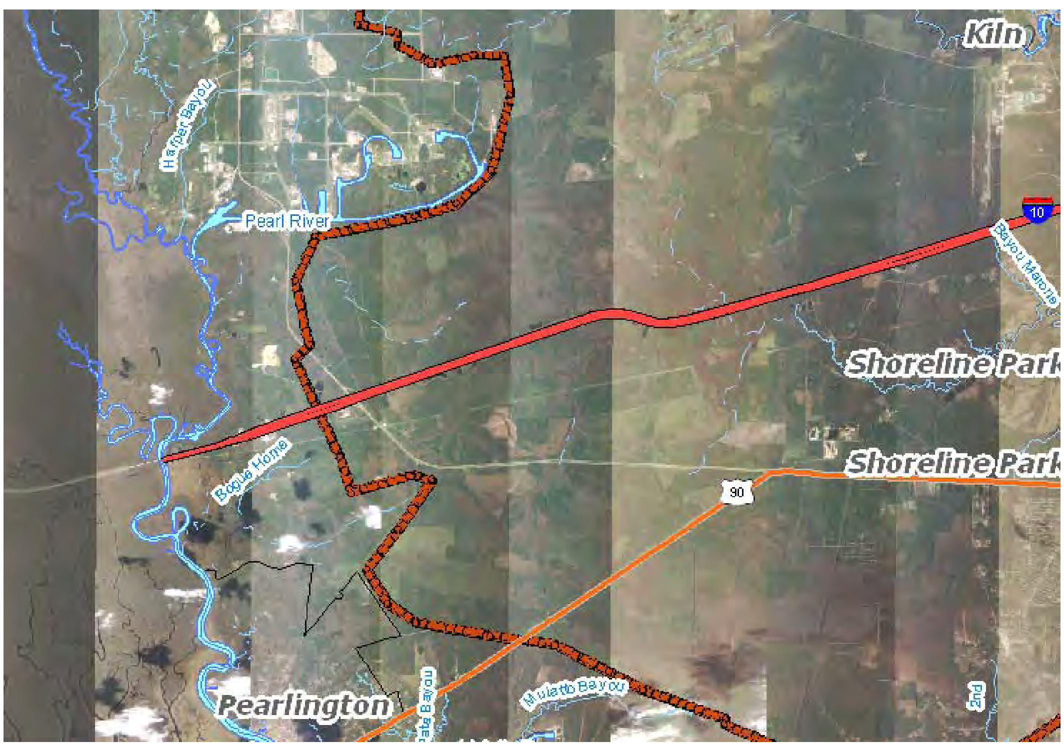


Figure 3.4.2-3. Hancock County Inland Barrier

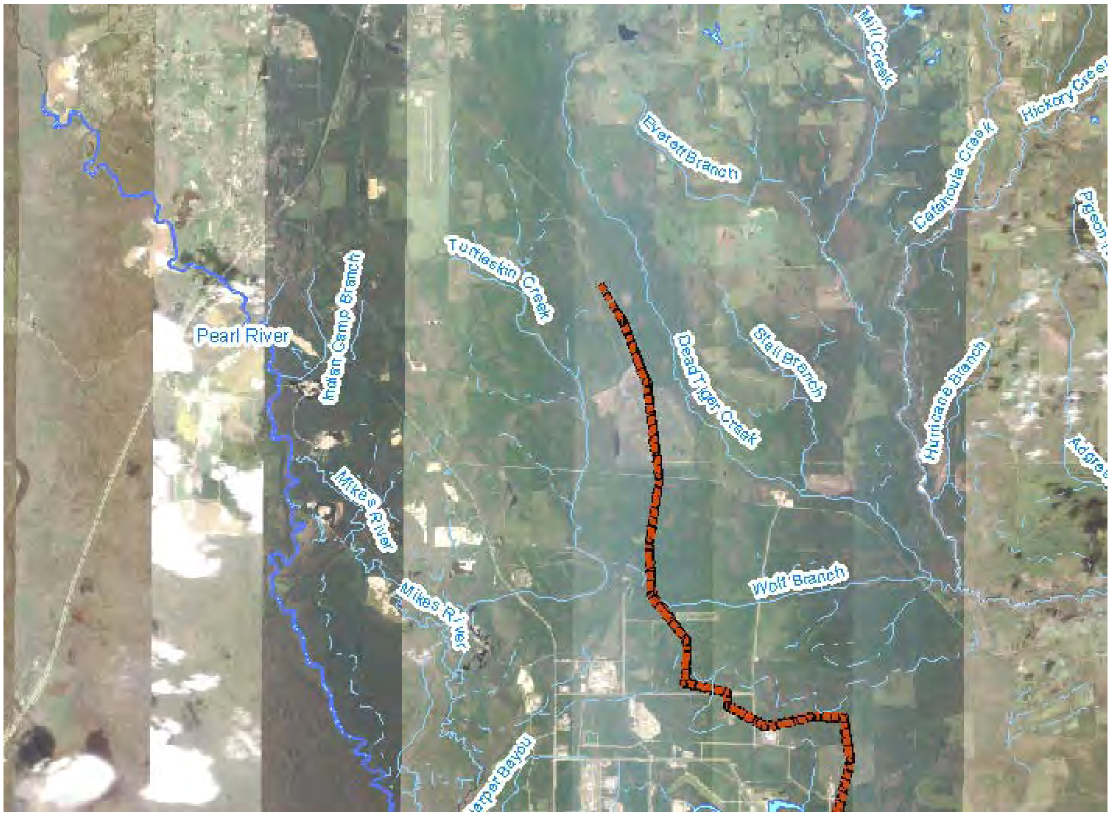


Figure 3.4.2-4. Hancock County Inland Barrier

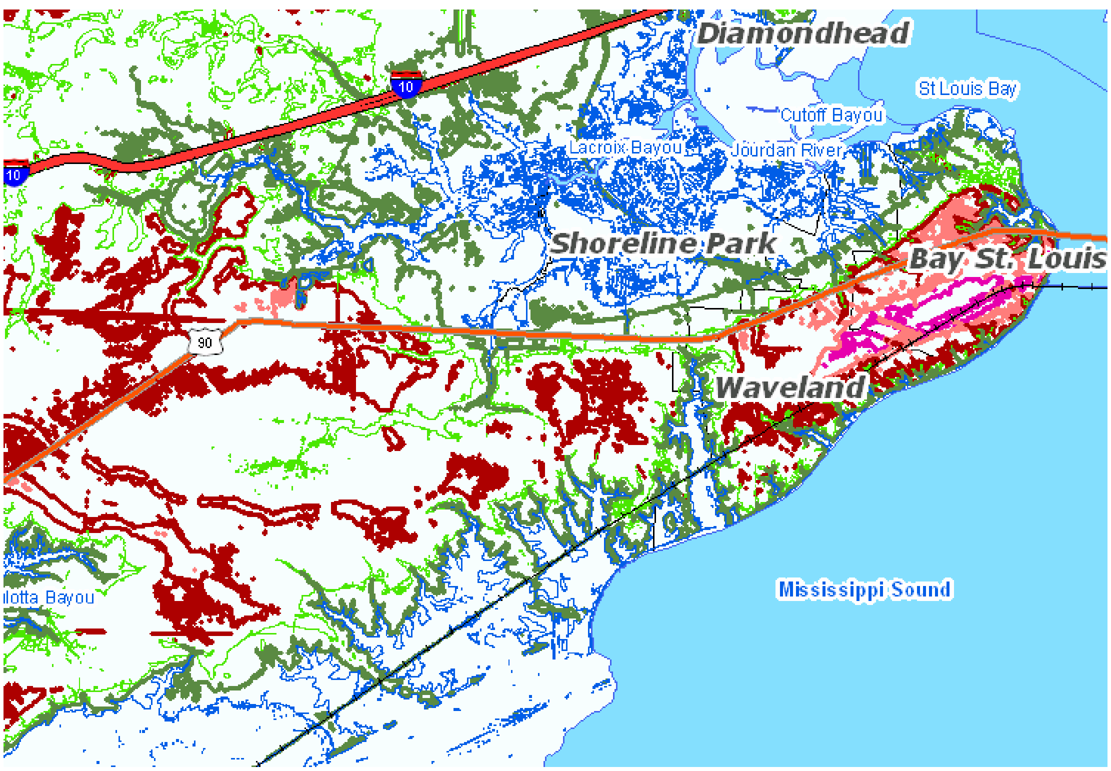


Figure 3.4.2-5. Existing Conditions Hancock County, MS

Drainage from ordinary rainfall is hindered on occasions when either of the rivers or the gulf is high, but impacts from hurricanes are devastating. Damage from Hurricane Katrina in August, 2005 in the Waveland area are shown in Figure 3.4.2-6 and 3.4.2-7.

Source: http://ngs.woc.noaa.gov/storms/katrina/24334552.jpg

Figure 3.4.2-6. Hurricane Katrina Damage Hancock Co, MS

Source: G.J. Charlet III, http://www.flickr.com/photo_zoom.gne?id=46937047&size=m

Figure 3.4.2-7. Hurricane Katrina Damage Hancock Co, MS

3.4.2.4 Coastal and Hydraulic Data

Historic coastal data is shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 4-ft(blue), 8-ft(Dark green), 12-ft(light green), 16-ft(brown), 20-ft(pink), and 24-ft(purple) ground contour lines are shown below in Figures 3.4.2-8 and 3.4.2-9. The data indicates the Katrina high water was as high as 26 ft NAVD88 in the Waveland/Bay St Louis area.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown elsewhere in this report. Points near Waveland/Bay St Louis at which data from hydrodynamic modeling was saved are shown in Figures 3.4.2-10 and 3.4.2-11.

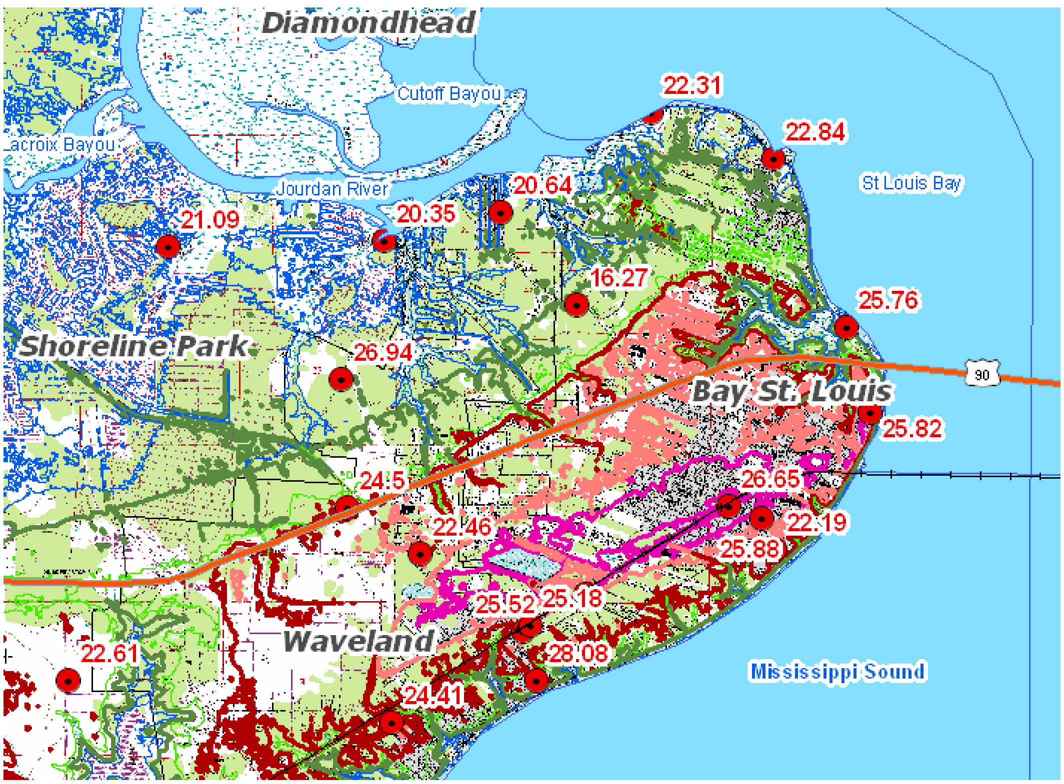


Figure 3.4.2-8. Ground Contours and Katrina High Water Elevations

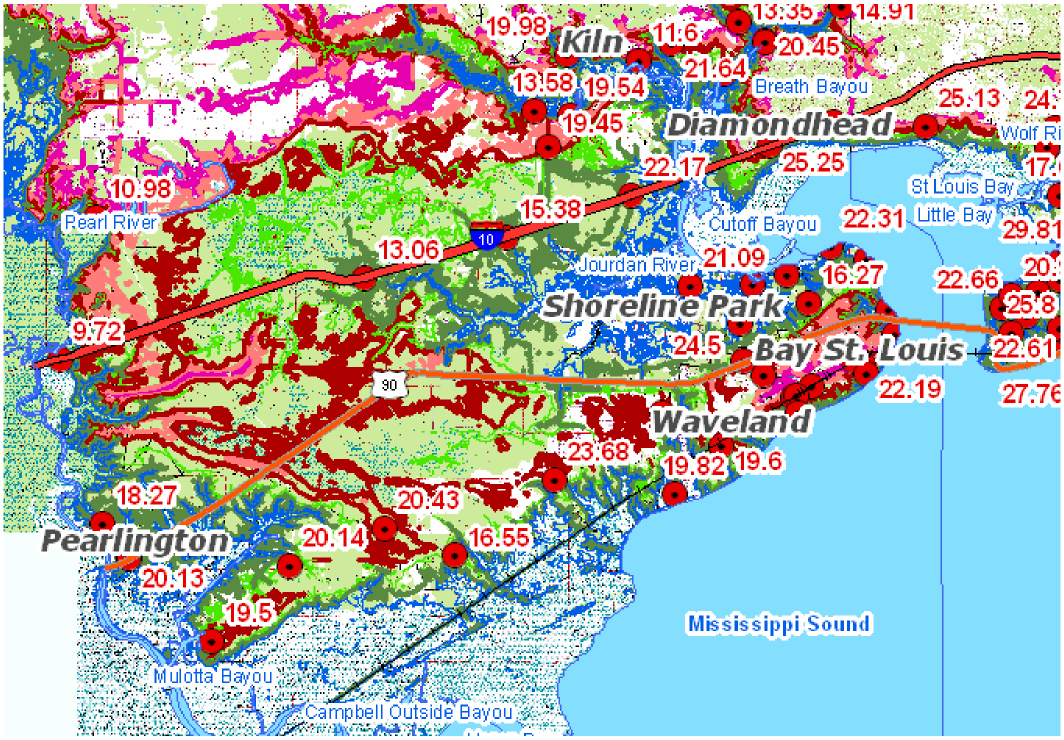


Figure 3.4.2-9. Ground Contours and Katrina High Water Elevations

Figure 3.4.2-10. Hydrodynamic Modeling Save Points near Waveland/Bay St Louis

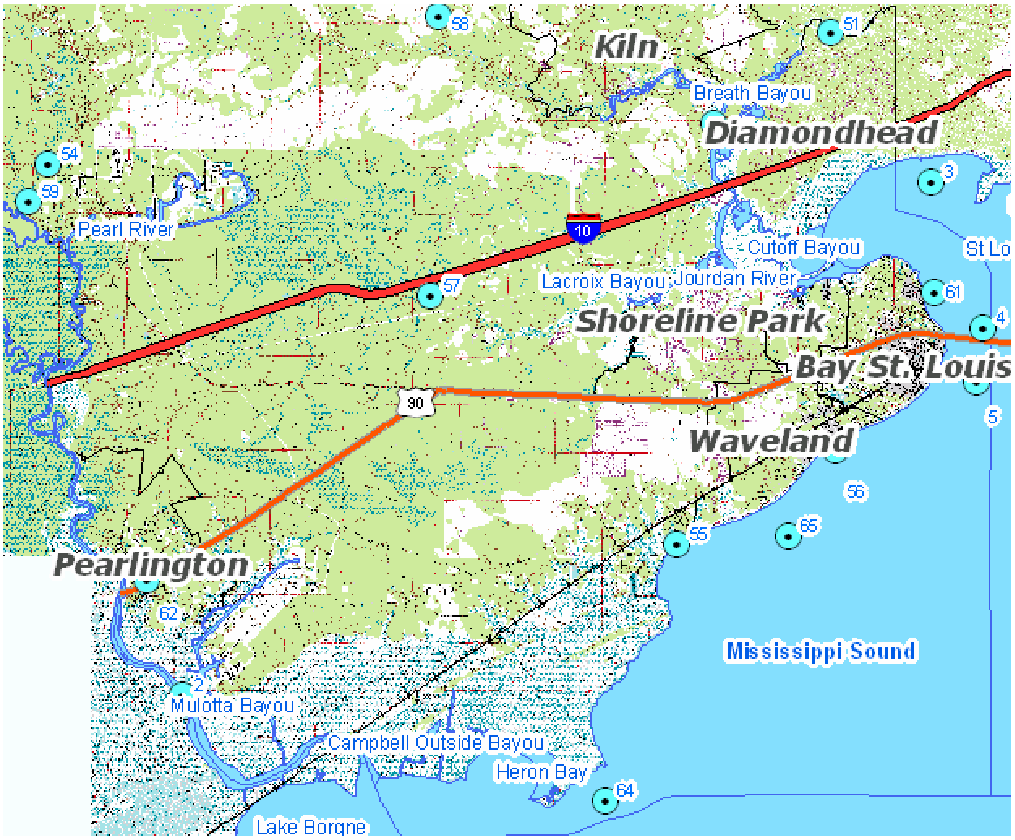


Figure 3.4.2-11. Hydrodynamic Modeling Save Points near Waveland/Bay St Louis

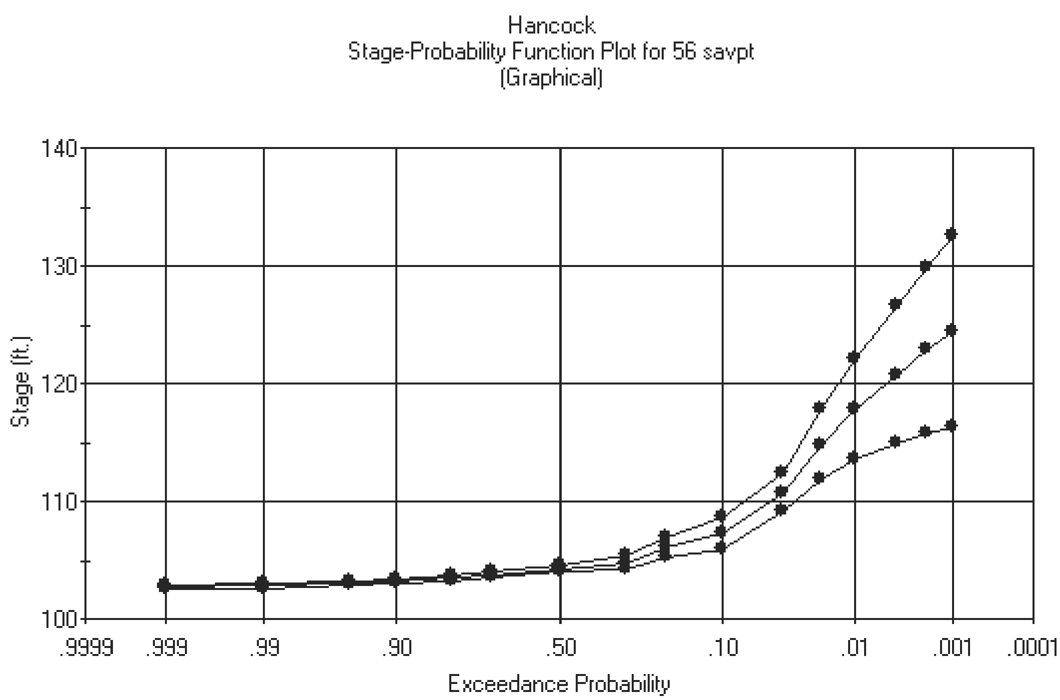


Figure 3.4.2-12. Existing Conditions at Save Point 56, near Waveland, MS

3.4.2.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike across the high ground of the county as shown on Figures 3.4.2-13 through 3.4.2-15, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal. The levee is located mostly along high ground so ponding at the levee would be minimal. Some ditching would be required on the outside of the levee which is shown in dark blue below. Small boat access structures are also shown at the basin 2. Rising sector gates will be provided at these sites allowing shallow draft traffic most of the time. The gates will be closed prior to hurricane storm surge. A drawing of a typical boat access gate is shown in Figure 3.3.12-5.

Figure 3.4.2-13. Pump/Culvert/Sub-basins/Boat Access Site Locations

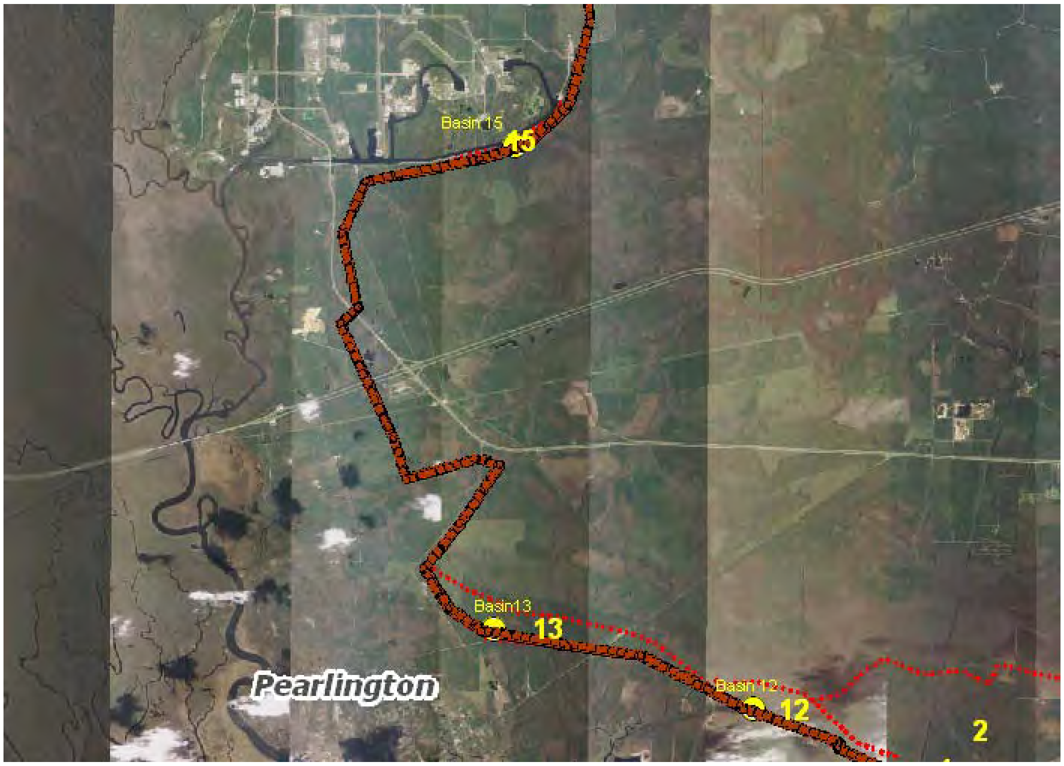


Figure 3.4.2-14. Pump/Culvert/Sub-basins/Boat Access Site Locations

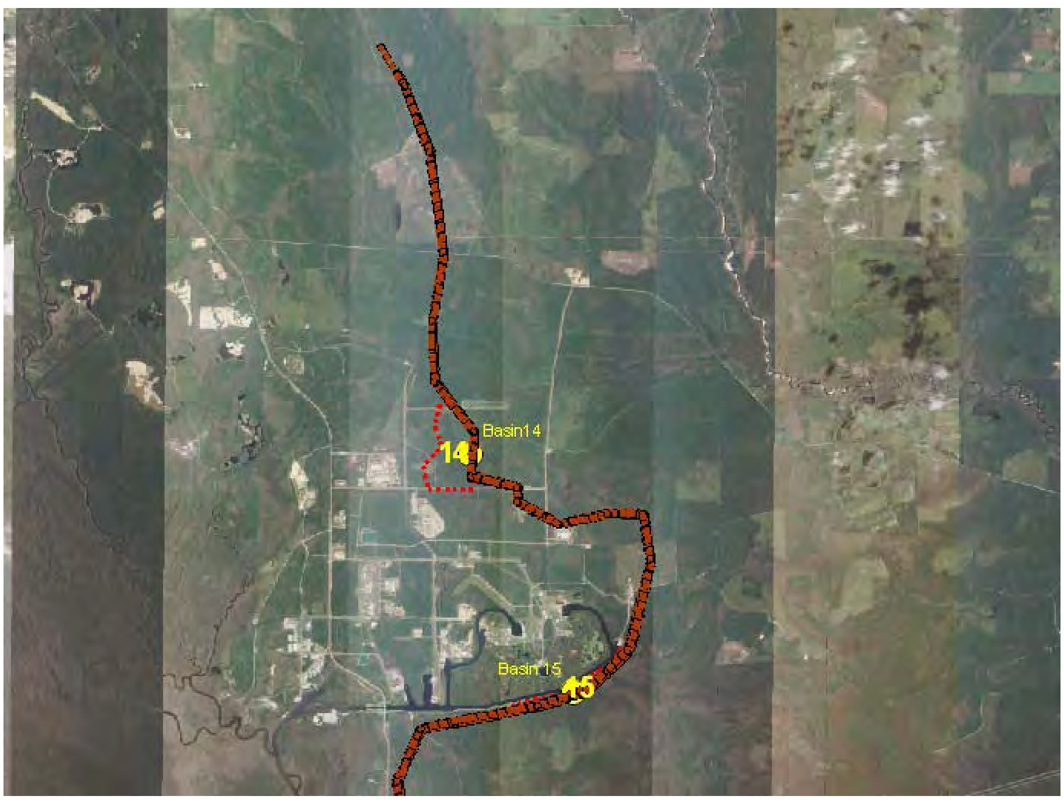


Figure 3.4.2-15. Pump/Culvert/Sub-basins/Boat Access Site Locations

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as depicted in Figure 3.4.2-16.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo: Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.4.2-16. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown in Figure 3.4.2-17 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.4.2-17. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment will be included in the levee design to prevent overtopping failure.

The levee would be protected by gabions on filter cloth as shown in Figure 3.4.2-18, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

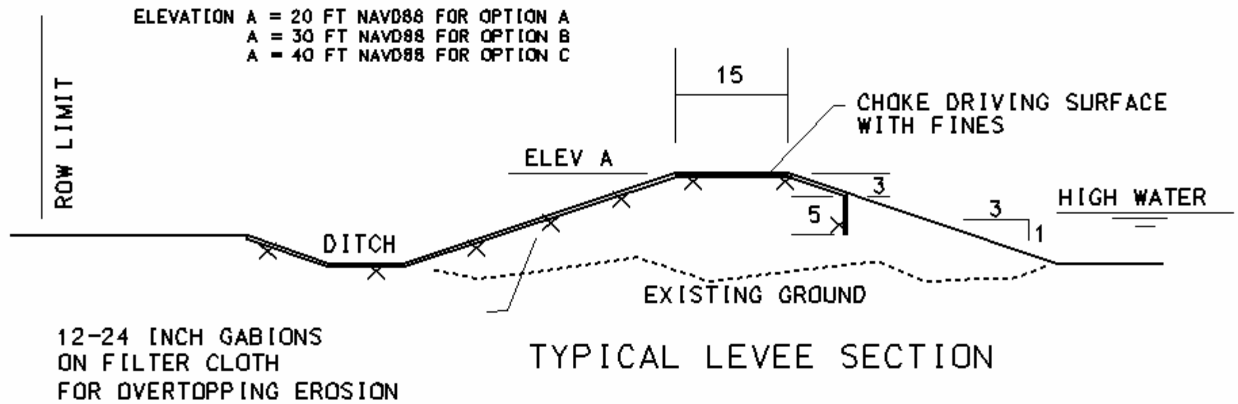


Figure 3.4.2-18. Typical Section at Inland barrier

3.4.2.5.1 Interior Drainage

For smaller drainage areas, drainage on the interior of the inland barrier would be collected at the levee and channeled to culverts placed in the levee at the locations shown in Figures 3.4.2-13 through 3.4.2-15. The culverts would have tidal gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at the upstream end at every culvert in the levee for manual control in the event the tidal gate malfunctions. A typical section is shown is shown in Figure 3.4.2-19.

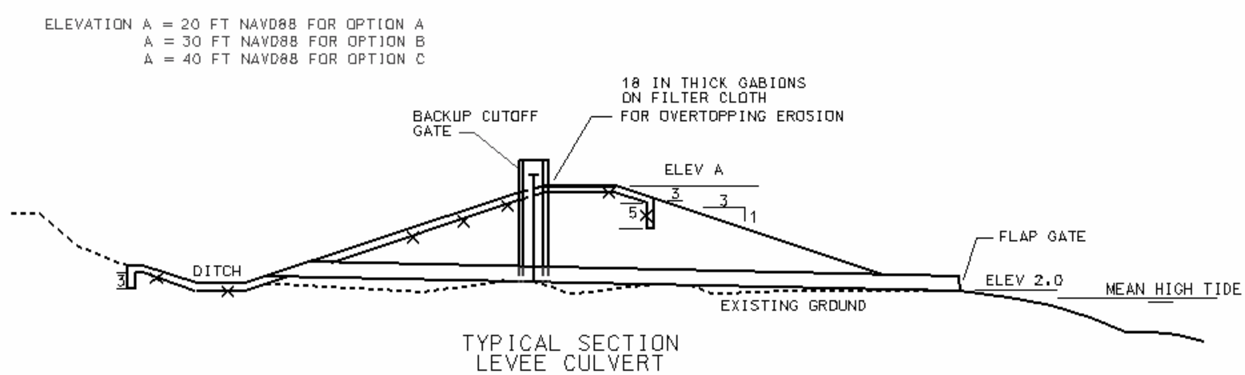


Figure 3.4.2-19. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the inland barrier into major sub-basins as shown in Figure 3.4.2-13 through 3.4.2-15 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Aerial Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report; therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

In addition to the local drainage outlets at the levee described above, in the event of an imminent hurricane, barrier gates across the St Louis Bay would be closed, and flow from the Jourdan and Wolf Rivers, as well as local runoff would pond behind the gates. The location of the barrier is shown in Figure 3.4.2-20.

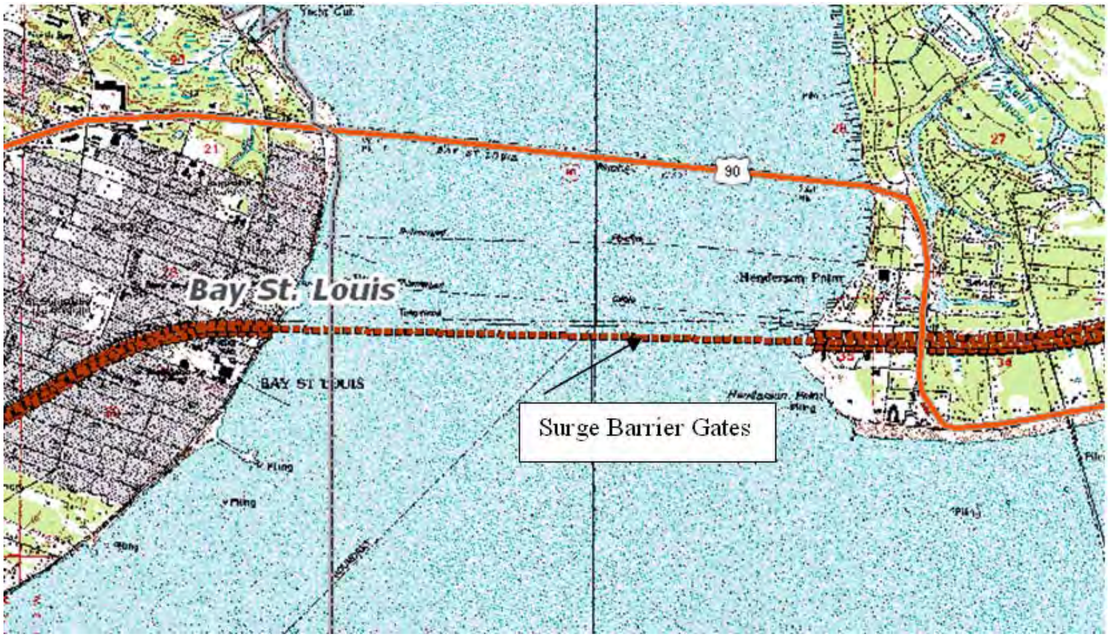


Figure 3.4.2-20. St Louis Bay Surge Barrier Location

The gates would be similar to the gates across the Thames River in London, England, shown in Figure 3.4.2-21.

Figure 3.4.2-21. Thames River Barrier Gates

The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used to model the St Louis Bay watershed in order to predict the maximum water elevation behind the gates in the bay under several different scenarios.

The St. Louis Bay watershed covers approximately 654 square miles and is comprised of six sub-basins that stretch across the Mississippi counties of Harrison, Hancock, Stone, and Pearl River. There is one United States Geological Survey (USGS) discharge stream gage (#2481510) located in the watershed along the Wolf River, near Landon, Mississippi. There are three significant National

Oceanic and Atmospheric Administration (NOAA) hourly precipitation gages located nearby to the watershed: #109617 White Sand located to the west, #87720 Purvis 2 N to the north, and #109617, 87720, and 107840 Saucier Experimental Forest to the east of the basin. Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering St. Louis Bay from the St. Louis Bay watershed for the 2 year, 5 year, 10 year, 25 year, 50 year and 100 year rainfall events. The St. Louis Bay watershed is shown in Figure 3.4.2-22.

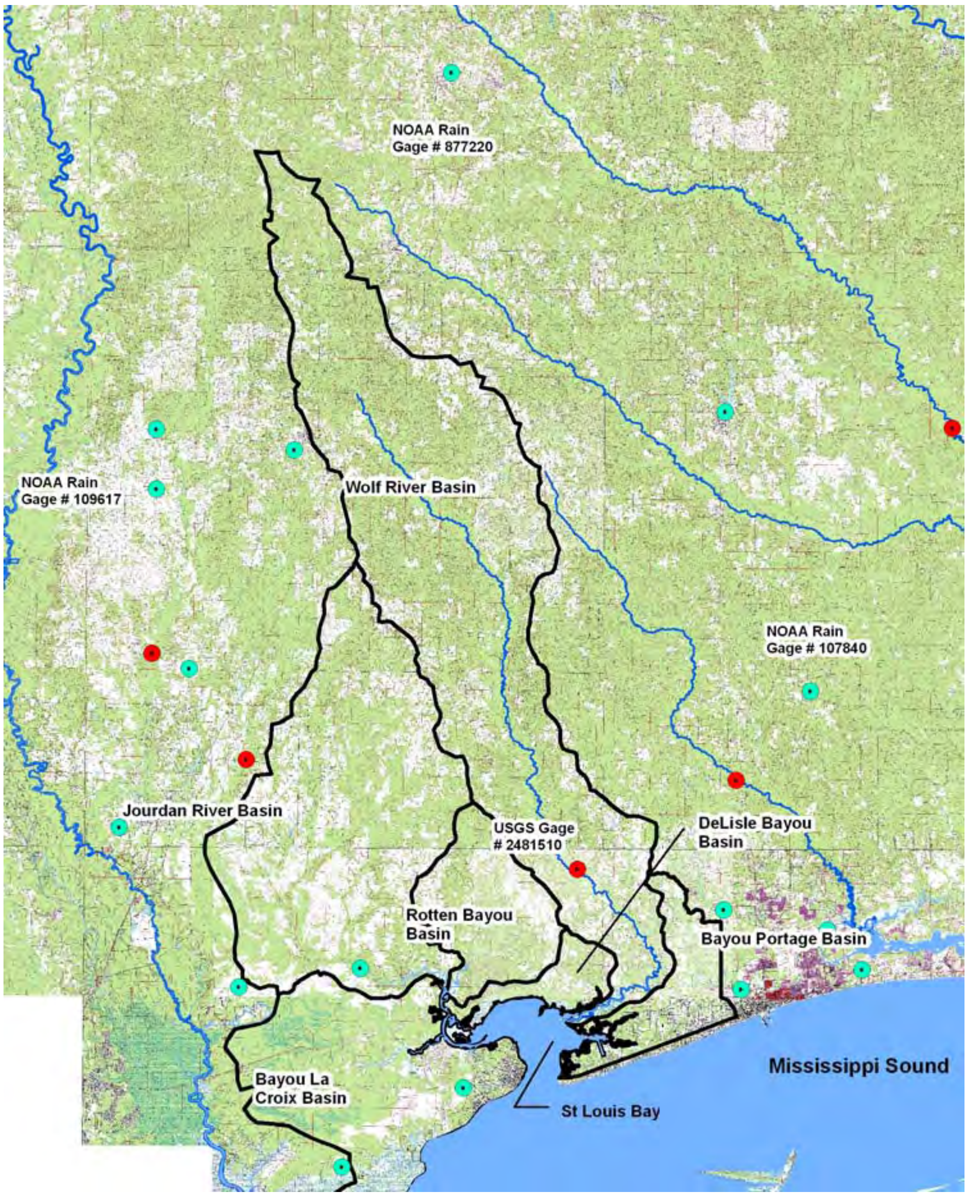


Figure 3.4.2-22. St Louis Bay Watershed

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gages 109617, 87720, and 109617, 87720, and 107840 and the 2-100 year 24-hour TP-40 rainfall events. The initial and constant loss rate method was used for the loss model while the Snyder's unit hydrograph (UH) method was used for the direct runoff model. The model was calibrated to observed hourly discharge data for one event at USGS gage 2481510. Several other events were analyzed but not used because the observed hourly precipitation for those events did not match the TP-40 rainfall.

The HEC-HMS St. Louis Bay watershed model was calibrated to the September 24-30, 2002 storm events. The model was calibrated at the Upper Wolf River sub-basin using observed precipitation data from NOAA gages 109617, 87720, and 107840 and observed discharge data from USGS gage 2481510. This event had a total rainfall of 13.75 inches and peak discharge of 17,854 cfs. This event was chosen due to the availability of both the hourly precipitation and discharge data. The observed and computed hydrographs are shown in Figure 3.4.2-23.

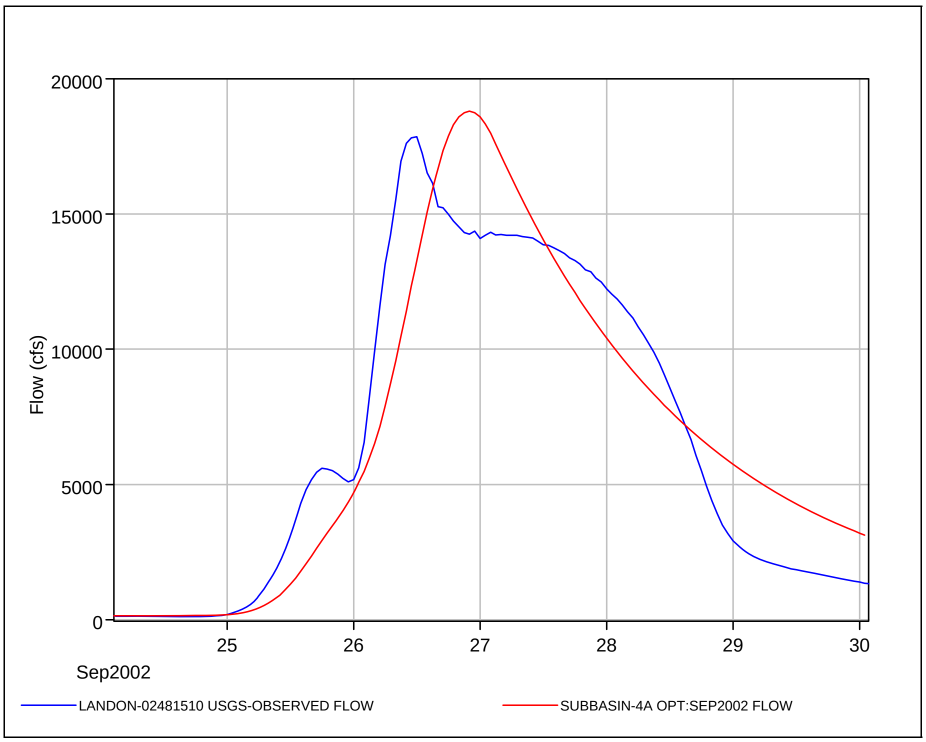


Figure 3.4.2-23. St. Louis Bay Watershed Calibration

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition, in accordance with studies cited above. The highest inflow period of the inflow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

Based on this method of analysis, the resulting elevations for the various storms are shown in Table 3.4.2-1, with the 10-yr elevation of 6.8 ft NAVD88 the design condition.

This ponded water area in Hancock County above the surge barrier gates is approximated by the 8-ft ground contour line shown in Figure 3.4.2-24.

Table 3.4.2-1.
St. Louis Bay Ponding

St. Louis Bay 4 ft. Base Elevations	
Strom Event	**Bay Elevation (ft NAVD88)**
2-year	5.5
5-year	6.3
10-year	6.8
25-year	7.5
50-year	7.9
100-year	8.4

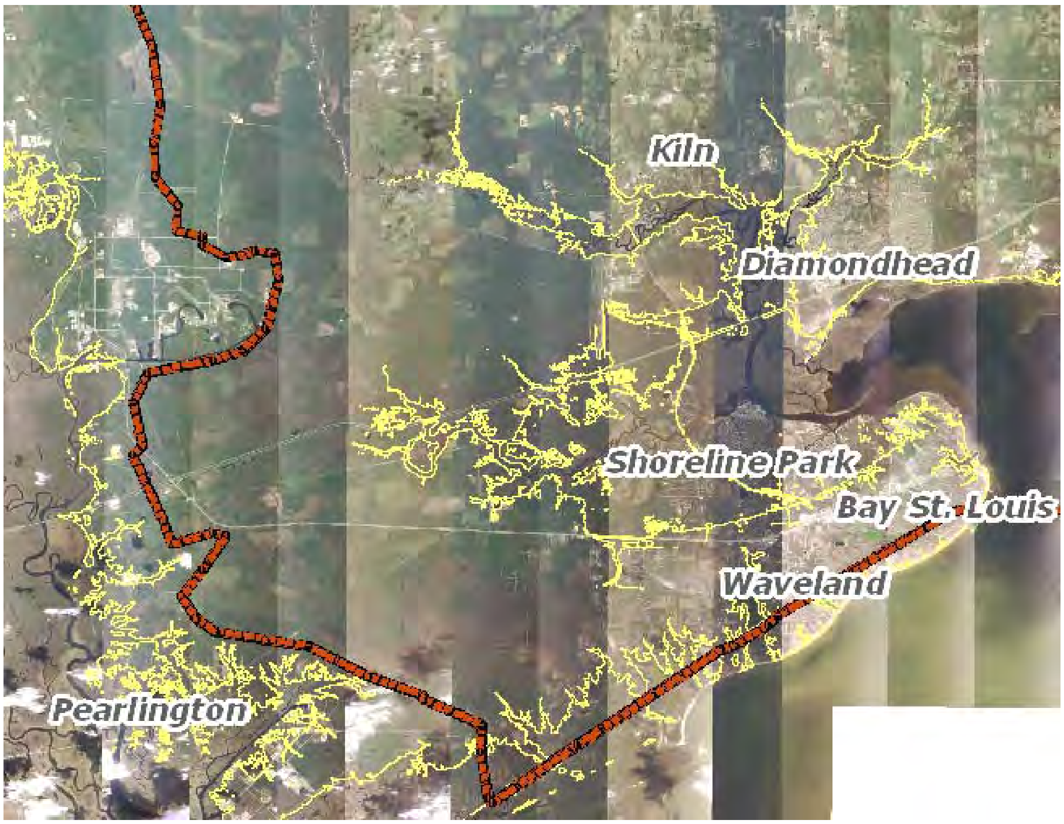


Figure 3.4.2-24. St Louis Bay 10-yr Ponding to Elev. 6.8 ft NAVD88

3.4.2.5.2 Geotechnical Data

Geology: The Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.2.5.3 Structural, Mechanical and Electrical

See sections 3.4.2.5.3.1 through 3.4.2.5.3.3.

3.4.2.5.3.1 Culverts

Drainage features would be required at 16 locations ranging from 20-inch diameter reinforced concrete pipe to reinforced concrete box culverts having 11 water passages, each measuring 12’ wide by 4’ high. Each of the culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.2.5.3.2 Pumping Stations

The design hydraulic heads derived for the three pumping facilities included in the Hancock County Inland Barrier for the elevation 20 protection level varied from 15 to 20 feet and the corresponding flows required varied from 59,694 to 390,483 gallons per minute. The facilities thus derived would vary from a plant having two, 42-inch diameter, 300 horsepower pumps, to one having four, 60-inch diameter pumps operating at 560 horsepower.

3.4.2.5.3.3 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 20, 14 roadway intersections would have to be accommodated. For this study it was estimated that 4 roller gate structures and 6 swing gate structures would be required. In addition, 4 railway closures would be required.

3.4.2.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.2.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.2.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the <u>Risk Assessment Methodology for Dams (RAM-D</u>) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. This level is the most applicable to this option.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would

possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.2.5.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.2.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.2.8 Cost Summary. Construction costs for the various options are included in Table 3.4.2-2 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.4.2-3 Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.2.5.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.2.6 Option B – Elevation 30.0 NAVD 88

3.4.2.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.4.2.6.2 Geotechnical Data

Geology: Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 12 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.2.6.3 Structural, Mechanical and Electrical

See sections 3.4.2.6.3.1 through 3.4.2.6.3.3.

3.4.2.6.3.1 Culverts

Drainage features would be required at 16 locations ranging from 20-inch diameter reinforced concrete pipe to reinforced concrete box culverts having 11 water passages, each measuring 12’ wide by 4’ high. Each of the culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.2.6.3.2 Pumping Stations

Design hydraulic heads derived for the three pumping facilities included in the Hancock County Inland Barrier for the elevation 30 protection level varied from 25 to 30 feet and the corresponding flows required varied from 59,694 to 390,483 gallons per minute, respectively. The facilities thus derived would consist of one plant having two, 42-inch diameter, 500 horsepower pumps to one having four, 60-inch diameter pumps operating at 1000 horsepower.

3.4.2.6.3.3 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 30, 31 roadway/railway intersections would have to be accommodated. For this study it was estimated that 9 roller gate structures and 18 swing gate structures would be required. In addition, 4 railway closure gates would be required.

3.4.2.6.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.2.6.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.2.6.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. This level of security is the most applicable to this option.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would

possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.2.6.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.2.6.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.2.8 Cost Summary. Construction costs for the various options are included in Table 3.4.2-2 and costs for the annualized Operation and Maintenance of the options are included in Table 3.3.4.2-3. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.2.6.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.2.7 Option C – Elevation 40.0 NAVD 88

3.4.2.7.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.4.2.7.2 Geotechnical Data

Geology: Citronelle formation extends north of Interstate 10 and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into somewhat friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County and western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will not be armored since the elevation will not allow overtopping. All surfaces of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.2.7.3 Structural, Mechanical and Electrical

See sections 3.4.2.7.3.1 through 3.4.2.7.3.3.

3.4.2.7.3.1 Culverts

Drainage features would be required at 16 locations ranging from 20-inch diameter reinforced concrete pipe to reinforced concrete box culverts having 11 water passages, each measuring 12’ wide by 4’ high. Each of the culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.2.7.3.2 Pumping Stations

Design hydraulic heads derived for the three pumping facilities included in the Hancock County Inland Barrier for the elevation 40 protection level varied from 30 to 35 feet and the corresponding flows required varied from 59,694 to 390,483 gallons per minute, respectively. The facilities thus derived would consist of one plant having two, 42-inch diameter, 500 horsepower pumps to one having six, 54-inch diameter pumps operating at 1000 horsepower.

3.4.2.7.3.3 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 40, 40 roadway/railway intersections would have to be accommodated. For this study it was estimated that all 36 of the highway crossings would require swing gates. In addition, 4 railway closure gates would be required.

3.4.2.7.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.2.7.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.2.7.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. This level of security is the most applicable to this option.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.2.7.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.2.7.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.2.8 Cost Summary. Construction costs for the various options are included in Table 3.4.2-2 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.2-3. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and

Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.2.7.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.2.8 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.4.2-2.
Hancock Co Inland Barrier Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$379,400,000
Option B – Elevation 30 ft NAVD88	$852,200,000
Option C – Elevation 40 ft NAVD88	$790,800,000

Table 3.4.2-3.
Hancock Co Inland Barrier O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$3,390,000
Option B – Elevation 30 ft NAVD88	$8,934,000
Option C – Elevation 40 ft NAVD88	$7,562,000

3.4.2.9 References

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USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

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Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

Weather Bureau and USACE. 1956. National Hurricane Research Project Report No. 3, “Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers.

3.4.3 St. Louis Bay Surge Barrier

3.4.3.1 General

In order to protect the properties surrounding Saint Louis Bay and along the lower portions of the various rivers and streams flowing into the bay, a barrier would be required at some point to block storm waters during major storm events. A proposed alignment for the surge barrier is shown in Figure 3.4.3.1-1. As outlined above, a search of other similar facilities constructed world wide revealed that the structure model best satisfying both the engineering and socio-ecological necessities of this site was that used for the Thames River Barrier in London, UK. The structure tentatively investigated for incorporation into this work was thus, patterned after the Thames River Barrier with certain minor modifications to adapt to the site and environment specific conditions enumerated previously.

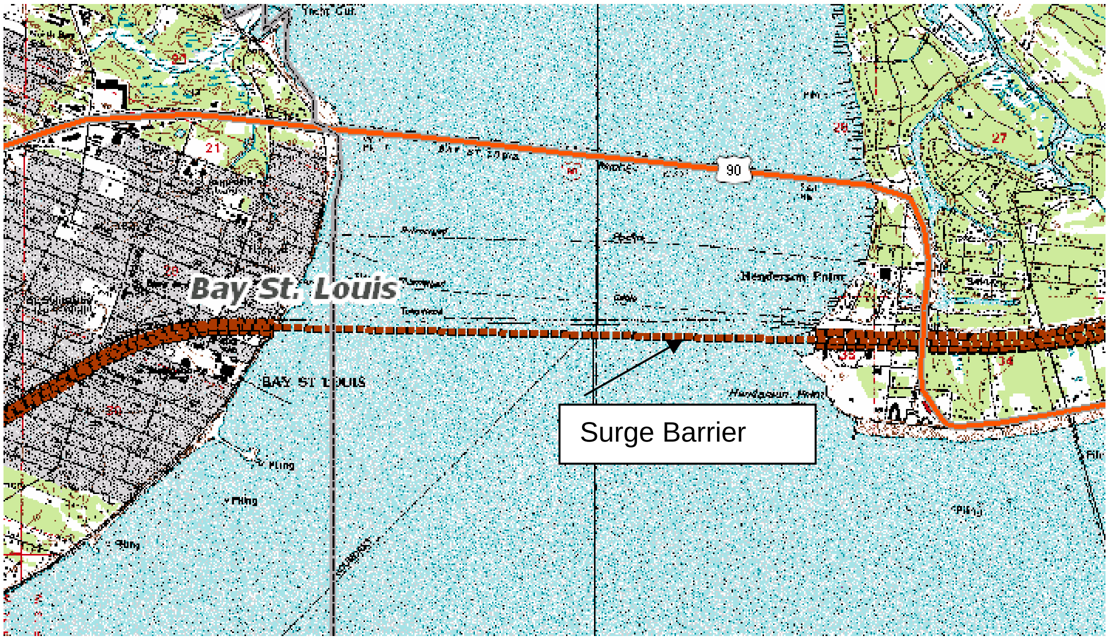


Figure 3.4.3.1-1. St. Louis Bay Surge Barrier Location

3.4.3.1.1 Interior Drainage

In the event of an imminent hurricane, the gates St Louis Bay would be closed, and flow from the rivers feeding these bays, as well as local runoff would pond behind the gates. The tentative location of the barrier chosen for this study is shown below.

The St. Louis Bay watershed, Figure 3.4.3.1-2, covers approximately 654 square miles and is comprised of six sub-basins that stretch across the Mississippi counties of Harrison, Hancock, Stone, and Pearl River. There is one United States Geological Survey (USGS) discharge stream gage (#2481510) located in the watershed along the Wolf River, near Landon, Mississippi. There are three significant National Oceanic and Atmospheric Administration (NOAA) hourly precipitation gages located nearby to the watershed: #109617 White Sand located to the west, #87720 Purvis 2 N to the north, and #109617, 87720, and 107840 Saucier Experimental Forest to the east of the basin. Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering St. Louis Bay from the St. Louis Bay watershed for the 2 year, 5 year, 10 year, 25 year, 50 year and 100 year rainfall events.

The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gages 109617, 87720, and 109617, 87720, and 107840 and the 2-100 year 24-hour TP-40 rainfall events. The initial and constant loss rate method was used for the loss model while the Snyder's unit hydrograph (UH) method was used for the direct runoff model. The model was calibrated to observed hourly discharge data for one event at USGS gage 2481510. Several other events were analyzed but not used because the observed hourly precipitation for those events did not match the TP-40 rainfall.

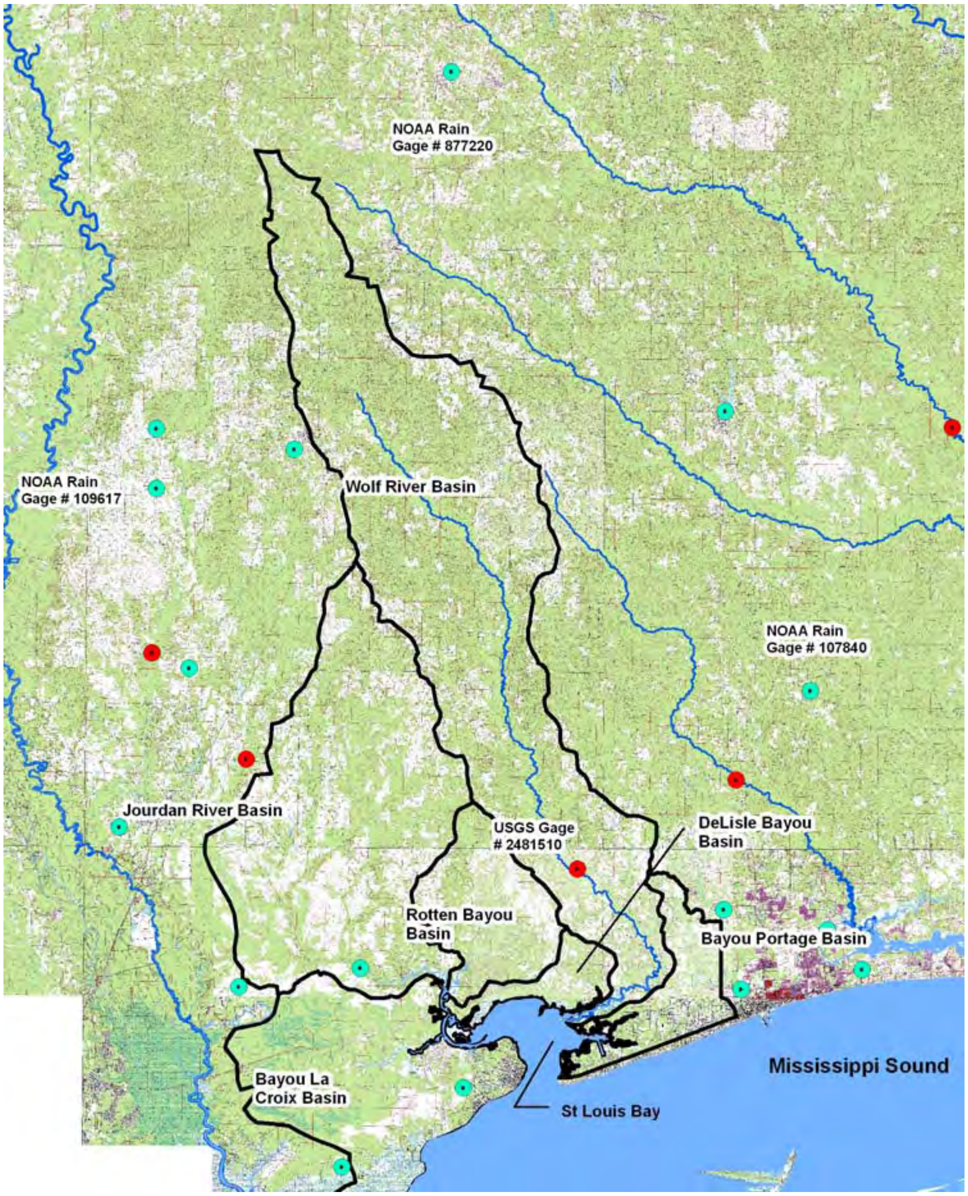


Figure 3.4.3.1-2. St. Louis Bay Watershed

Calibration results agree reasonably well with observed data as shown in Figure 3.4.3.1-3.

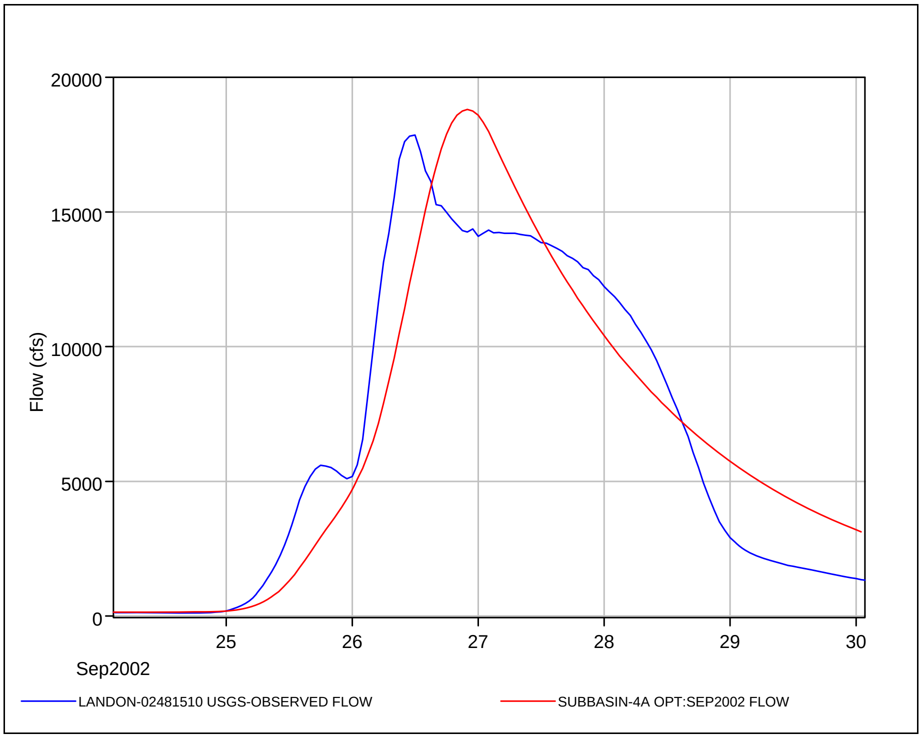


Figure 3.4.3.1-3. St. Louis Bay Watershed Calibration

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V. Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District, U.S. Army Corps of Engineers.

The 24-hour period of highest inflow from the flow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

Based on this method of analysis, the resulting elevations for the various storms are shown in Table 3.4.3.1-1, with the 10-yr elevation of 6.8 ft NAVD88 the design condition.

Table 3.4.3.1-1.
St. Louis Bay Ponding

St. Louis Bay 4 ft. Base Elevations	
Strom Event	**Bay Elevation (ft NAVD88)**
2-year	5.5
5-year	6.3
10-year	6.8
25-year	7.5
50-year	7.9
100-year	8.4

The ponded water area in above the surge barrier gates is approximated by the 8-ft ground contour line shown in Figure 3.4.3.1-4.

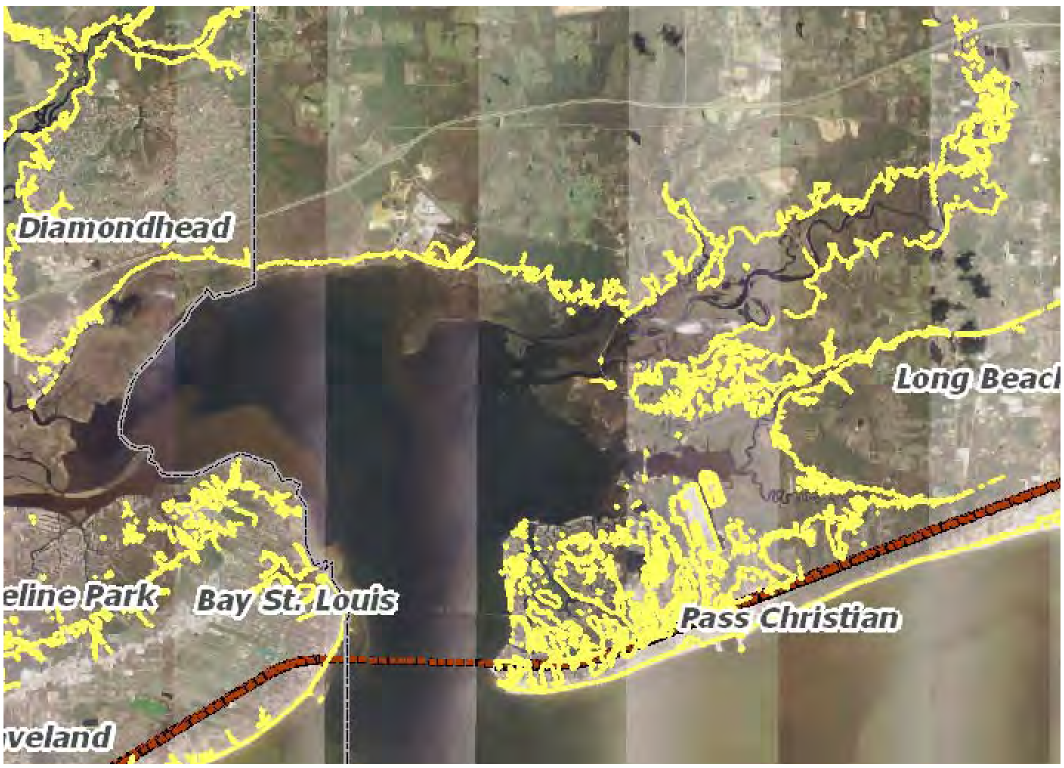


Figure 3.4.3.1-4. St. Louis Bay 10-yr Ponding to Elev 6.8 ft NAVD88

3.4.3.1.2 Geotechnical Data

The available mapping covering the bay bottom is very sketchy consisting mostly of quad maps. This data indicates that the existing bay bottom elevation along the study alignment would be fairly uniform at approximate (-)7 to (-) 8 feet across much of the bay width. The water depth naturally tapers from full depth to the water's edge over some distance out from each bank. Information gathered from the Mississippi Department of Transportation indicates that the bay bottom materials are very loose and unstable to a significant depth below the bay bottom indicating that a significant amount of undercutting would be required for any structure that might be installed, and that structures of the magnitude under consideration would require very deep pile foundations.

3.4.3.1.3 Structural, Mechanical and Electrical

See sections 3.4.3.1.3.1 through 3.4.3.1.3.3.

3.4.3.1.3.1 Structural

Structurally, the Barrier as configured for this study would consist of a series of 38 large stainless steel clad, structural steel framed gates called rising sector gates. Each of these would be supported on reinforced concrete piers resting on large continuous concrete sills with pile foundations. The tentative layout used to estimate the scope of the structure was configured having gates 132 feet long mounted on 28-foot wide piers. The number of gates was determined by the extent of water having depth sufficient to support their operation. To facilitate as nearly as possible the normal ebb and flow of tide waters through the barrier, the concrete connector wall and rock fill portions of the barrier either side of the gated structure would be fitted with a series of closely spaced low level gated culverts. The gate and pier heights were varied to accommodate the "level of protection" under consideration. The three elevations selected for this study were 20, 30, and 40 NAVD88. In each instance the gate heights were set to match the protection level elevations with pier heights set approximately 3 feet higher to provide minor wave clearance for protection of operating equipment. Atop each pier an operating machinery block would be mounted to house the operating equipment. No lateral access over the tops of the piers was envisioned because of the long spans and the desire to keep the vista across the structure as clear as possible. Operating and utility access would be provided through two continuous tunnels passing through the sill section and the rock fill, to operating facilities located on each bank.

3.4.3.1.3.2 Mechanical

The mechanical equipment and appurtenances required for operation of these facilities would include very large steel gate linkages and hydraulic rams and pivot pins for operation of each gate. Each gate would rotate on large bearings and pivot hubs at each end of the gate. Various operating hydraulic and lubrication oil systems would also be required. Each gate would have an opening/closing time of approximately 15 minutes.

3.4.3.1.3.3 Electrical

Primary electrical power for operating the gates would be provided using dedicated, standard transformers with emergency back-up generators. The size of the generators would be greatly reduced by minimizing the wattage output through reduction of the demand on the facility. The demand would be minimized by phasing the operation of the gates to the greatest extent possible. For this study it was determined that this could possibly be done by operating a maximum of eight gates at a time, with the last eight gates being left open until the storm threat was definite and eminent. The operation would require that a maximum of four gates be started at one time, with the remaining four gates sequenced to start 1 minute later. It was determined that this would allow the entire closure and subsequent opening operations to be done over a period of 4 to 6 hours. The supplemental generation aspect was considered to be a vital component of the design because of the very high cost of Commercial standby power and because commercial electric power would almost certainly be unavailable during and immediately following a storm event.

3.4.3.1.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report

therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.3.1.5 Construction Procedures and Water Control Plan

Following is a very tentative description of a sequence of construction by which the barrier structure and embankments might be built. There are admittedly myriad other means by which this could be accomplished as demonstrated by the construction methods used in construction of the Thames River Barrier and various structures in The Netherlands and elsewhere, any one of which might result in more economical and expeditious construction of the barrier. However, at this juncture, in the interest of clarity and brevity, it was considered expedient to describe this work using customary construction techniques common to most of our large civil works projects constructed to date.

3.4.3.1.5.1 Construction Procedure

As configured for this study, the physical construction of the barrier would begin with installation of the first of what would likely be a three stage cellular cofferdam. The arrangement assumed for this study consisted of a series of circular sheet pile cells and connecting arcs measuring approximately 60 feet in diameter and extending 100 feet from the top of the cell to the pile tip elevation. These cells would encompass either the east side or west side transition monoliths and approximately one-third of the gated portion of the structure. It was assumed that for structures designed to provide the highest protection level (Elevation 40 NAVD88) the top of cells could be placed at elevation 35 with reasonable degree of safety. This would provide cell embedment of approximately 30 feet below the lowest structure foundation elevation. This configuration was, naturally, modified to fit the lower levels of protection, but in each case the configuration was made to provide the same relative of protection during construction. With the cofferdam in place the interior would be dewatered using hydraulic pumps, and excavation for the concrete structures would begin. Once the excavation in a given area is brought to the required grade work would continue in this area with the installation of foundation piles. Prior to completion of this phase of the work, installation would begin on the next phase of the cofferdam.

Once the first phase of the concrete structure is completed and the first phase cofferdam removed, installation of the gates and operating machinery would begin. Fabrication of the gates would have been done on land in an outfitting yard and the gates transported by water to the proper installation site. Note that this would likely require dredging of a temporary construction channel parallel to the barrier for a portion of its length.

Construction of the rock fill embankments would require surcharging and pre-consolidation of the bay bottom materials. (See section 3.4.3.1.2 above for discussion of the Geotechnical aspects of this site.)

3.4.3.1.5.2 Water Control Plan

As this work progresses the flow into and out of Saint Louis Bay would be somewhat restricted for practically the entire construction time. This restriction could be minimized by removal of the cofferdams immediately upon completion of the concrete piers to some point above the normal high tide level thus allowing flow over the completed sill sections as construction continues on the piers and as the gates are being installed. It is estimated the maximum flow restriction at any time would be approximately 30% of the inlet width and that this restriction could endure for as much as four to seven years using the methods and approximate sequence of construction indicated above.

3.4.3.1.6 Physical Security

As described in 3.4.1.7, the construction of the project the contractor would be responsible for maintaining security of all his work sites. This would be done in accordance with latest AT/FP guidance for projects of this type and scope in addition to the normal site security requirements.

Upon completion of the project the facilities security responsibilities would pass to the U.S. Army Corps of Engineers and the state, county and municipal law enforcement entities, all of whom would coordinate a program of oversight under which the facilities would be operated and maintained and under which specific security responsibilities would be defined and allocated. These agreements would also be required to meet AT/FP requirements in addition to normal security criteria.

3.4.3.1.7 Operations and Maintenance

In order to assure proper functioning of the facilities once they are placed in service a program of Operations and Maintenance would be developed by the U.S. Army Corps of Engineers, in conjunction and cooperation with the affected state and local entities. This O & M Plan would address specific responsibilities as to daily operation of the facilities, the periodic testing and maintenance of the operating machinery, maintenance of specified stocks of replacement parts, security of the facilities, and maintenance of any buildings and grounds associated with the operation and maintenance of the facilities. As presently envisioned, this O & M responsibility would remain under control of the U.S. Army Corps of Engineers and would be administered under its Operations mission.

3.4.3.1.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.3.8 Cost Summary. Construction costs for the various options are included in Table 3.4.3.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.3.8-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates exclude project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.3.1.9 Schedule and Design for Construction

The scheduling for events following this conceptual study would of necessity include further study to ascertain in greater detail the specific requirements of the project and the most feasible means by which to fulfill these requirements. The Sequence of events would include but not be limited to the following:

a. The alignment and extent of the proposed barrier should be subjected to detailed study to determine the most feasible routing. This study should address, among other factors, the exact location of utilities features crossing the bay inlet, the present and projected future needs of boat traffic passing through the barrier, and how best to minimize the effects that the barrier could have on the existing marine environment.

b. Detailed deep geotechnical investigation should be made to determine as accurately as possible the engineering capabilities of the soils making up the bay bottom along the alignment (or alignments) under consideration.

c. A more thorough and painstaking investigation of various types of gate structures should be undertaken to confirm the choice of the rising sector gate for this application, or to replace this type gate with another perhaps more appropriate to the circumstances.

d. Once exhaustive search and investigations and analyses have been completed a thorough design of the structures to be included in the final facility would be undertaken addressing the full range of hydraulic events that the structure might see, and making certain that all pertinent design considerations are accounted for.

e. A thorough analysis of the power required to operate the gates in a timely manner in time of storm must be made and the very best, most dependable means of providing this power determined.

f. The link between the operation of the gates and the best available storm forecasting system(s) would be designed and its operating features and equipment detailed.

3.4.3.2 Location

The alignment suggested herein for the barrier structure would run parallel with and south of the Railroad Bridge crossing Saint Louis Bay. This would approximate the shortest route across the inlet leading form the Mississippi Sound into the bay. As the layout of the barrier was developed it became apparent that, because of the excavation required, a significant amount of separation would be required between the railroad bridge and the ultimate location of the structures included in the barrier. For this study the centerline of the barrier was positioned approximately 260 feet from the center of the railroad bridge. This was left unaltered for all protection levels. The entire barrier would be approximately 10,320 feet in length from water's edge to water's edge, and would consist of rock fill levees extending from the overland levee at each bank for some distance into the bay and enveloping the mass concrete non-overflow wall sections leading to each end of the gated structure.

3.4.3.3 Existing Conditions

The points at which the barrier would come ashore in Jackson County on the east and Harrison County on the west, are in urban areas with extensive residential and commercial development. Several structures would need to be relocated and it is uncertain the extent to which existing utilities might have to be relocated to clear the way for this facility.

3.4.3.4 Coastal and Hydraulic Data

Historic coastal data is shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 8-ft(dark green), 12-ft(light green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown in Figure 3.4.3.4-1. The data indicates the Katrina high water was as high as 22 ft NAVD88 at the mouth of the bay.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown elsewhere in this report. Points near the mouth of the bay at which data from hydrodynamic modeling was saved are shown in Figure 3.4.3.4-2.

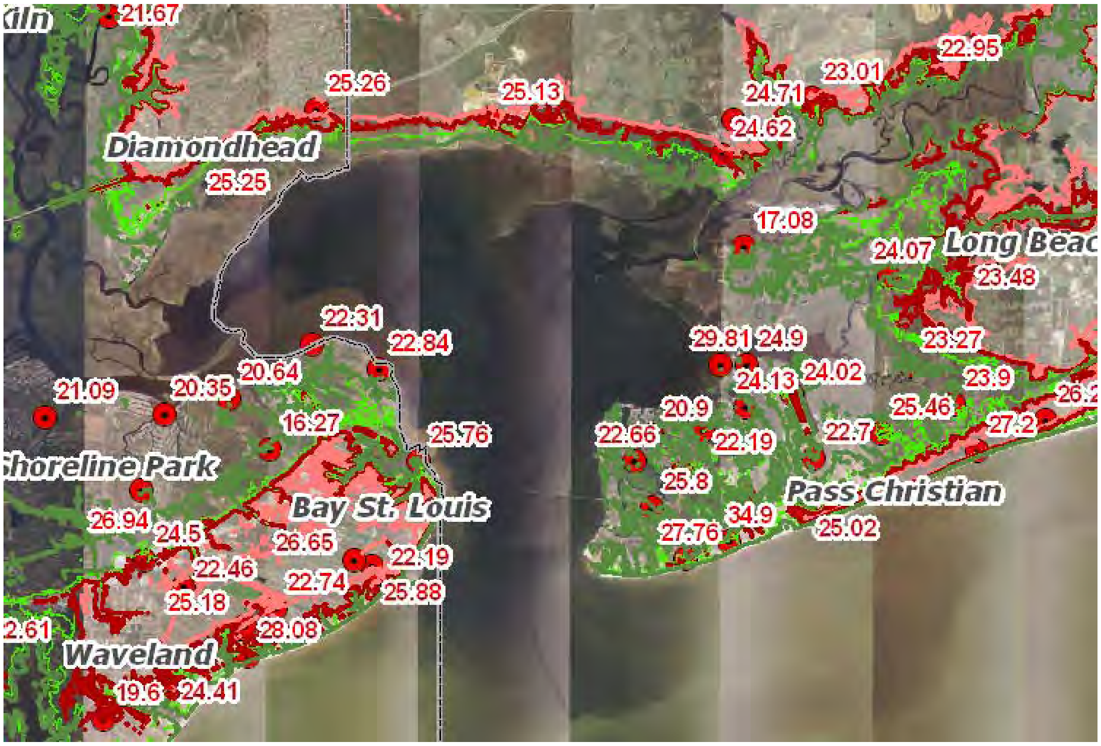


Figure 3.4.3.4-1. Ground Contours and Katrina High Water

Figure 3.4.3.4-2. Hydrodynamic Modeling Save Points near St Louis Bay

Existing Condition Stage –Frequency data for Save Point 61, near the mouth of the bay, is shown in Figure 3.4.3.4-3. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

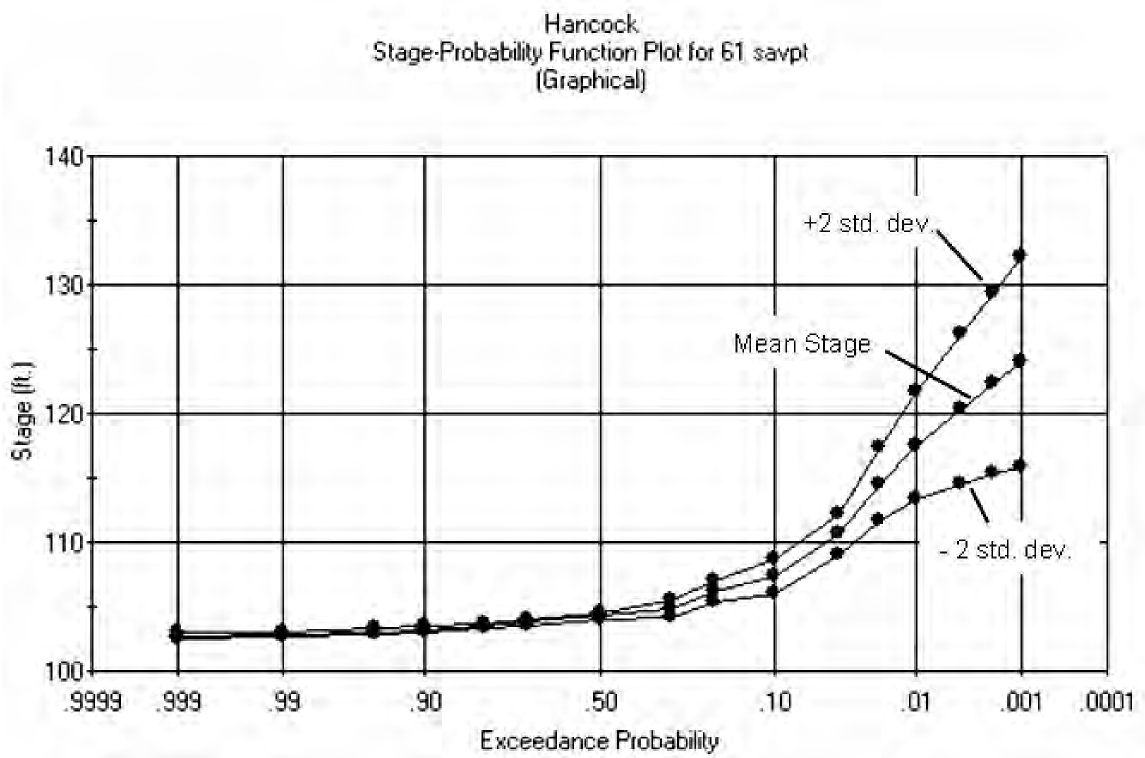


Figure 3.4.3.4-3. Existing Conditions at Save Point 61, near the Mouth of St. Louis Bay

3.4.3.5 Option A – Elevation 20.0

In order to reasonably accurately approximate the scope of the structures required to form a moveable barrier to elevation 20 a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. A system of foundation piles was then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation “Zero” on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation “zero” at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 20 on the flooded side of the structure and to elevation “zero” on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 20 resulted in gross quantities of basic construction materials as indicated in Table 3.4.3.5-1 below.

Table 3.4.3.5-1.
Gross Quantities for Saint Louis Bay Surge Barrier Elevation 20.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	38,008	Tons
Foundation Piling	20,540	Each
Concrete	493,700	Cubic Yards
Reinforcement	1,210	Tons
Rising Sector Gates (25 Each)	19,750	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.3.6 Option B – Elevation 30.0

In order to reasonably accurately approximate the scope of the structures required to form a moveable barrier to elevation 30, a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. The foundation piles were then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation “Zero” on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation “zero” at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 30 on the flooded side of the structure and to elevation “zero” on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 30 resulted in gross quantities of basic construction materials as indicated in Table 3.4.3.6-1 below.

Table 3.4.3.6-1.
Gross Quantities for Saint Louis Bay Surge Barrier Elevation 30.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	47,511	Tons
Foundation Piling	14,538	Each
Concrete	552,800	Cubic Yards
Reinforcement	1,083	Tons
Rising Sector Gates (25 Each)	24,260	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.3.7 Option C – Elevation 40.0

In order to reasonably accurately approximate the scope of The structures required to form a moveable barrier to elevation 40, a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. The foundation piles were then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation

"Zero" on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation "zero" at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 40 on the flooded side of the structure and to elevation "zero" on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 40 resulted in gross quantities of basic construction materials as indicated in Table 3.4.3.7-1 below.

Table 3.4.3.7-1.
Gross Quantities for Saint Louis Bay Surge Barrier Elevation 40.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	47,511	Tons
Foundation Piling	20,540	Each
Concrete	561,300	Cubic Yards
Reinforcement	1,061	Tons
Rising Sector Gates (25 Each)	40,291	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.3.8 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.4.3.8-1.
St Louis Bay Surge Barrier Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$1,628,000,000
Option B – Elevation 30 ft NAVD88	$1,963,600,000
Option C – Elevation 40 ft NAVD88	$2,362,200,000

Table 3.4.3.8-2.
St Louis Bay Surge Barrier O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$22,674,000
Option B – Elevation 30 ft NAVD88	$27,364,000
Option C – Elevation 40 ft NAVD88	$32,936,000

3.4.3.9 References

See 3.4.3 General discussion above for references.

3.4.4 Harrison County Inland Barrier

3.4.4.1 General

Residential and business areas along the coast in Harrison County are susceptible to storm surge damage. A damage reduction option is to construct an inland barrier to various elevations were evaluated. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center's (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented in Section 2.14 of the Engineering Appendix and in the Economic Appendix.

3.4.4.2 Location

The location of the barrier in Harrison County is shown in Figure 3.4.4-1 extending from Biloxi Bay to Pass Christian approximately 1000-3000 ft north of, and parallel to, the shoreline. This alignment is evaluated in Options A through E. For Options F through J, an alternate alignment is evaluated. This alternate alignment extends from Biloxi Bay to Menge Avenue, thence northward along Menge Avenue to high ground. Both alignments are shown on the map. They are also shown in more detail in the Option A section (Figures 3.4.4-12 through 3.4.4-14) and the Option F section (Figures 3.4.4-29 through 3.4.4-31).

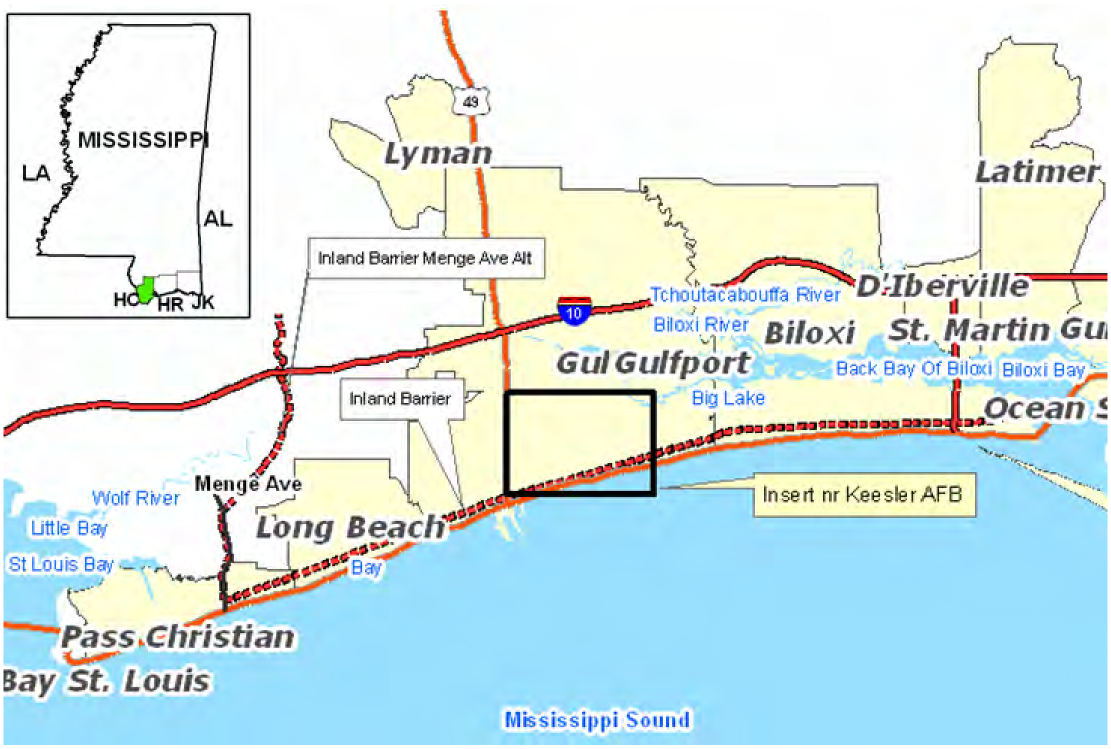


Figure 3.4.4-1. Vicinity Map Harrison County, MS

3.4.4.3 Existing Conditions

In Harrison County, ground elevations over most of the residential and business areas vary between elevation 8-12 ft NAVD88 on the coast and rising within 1000 ft to elevation 30-36 along a ridge parallel to the coast line, then decreasing to the north. The 4-ft (blue), 8-ft (green), 20-ft (pink), 30-ft (dark blue) and 34-ft (gold) ground contours show the pattern at the coastline for the county and are shown in Figure 3.4.4-2.

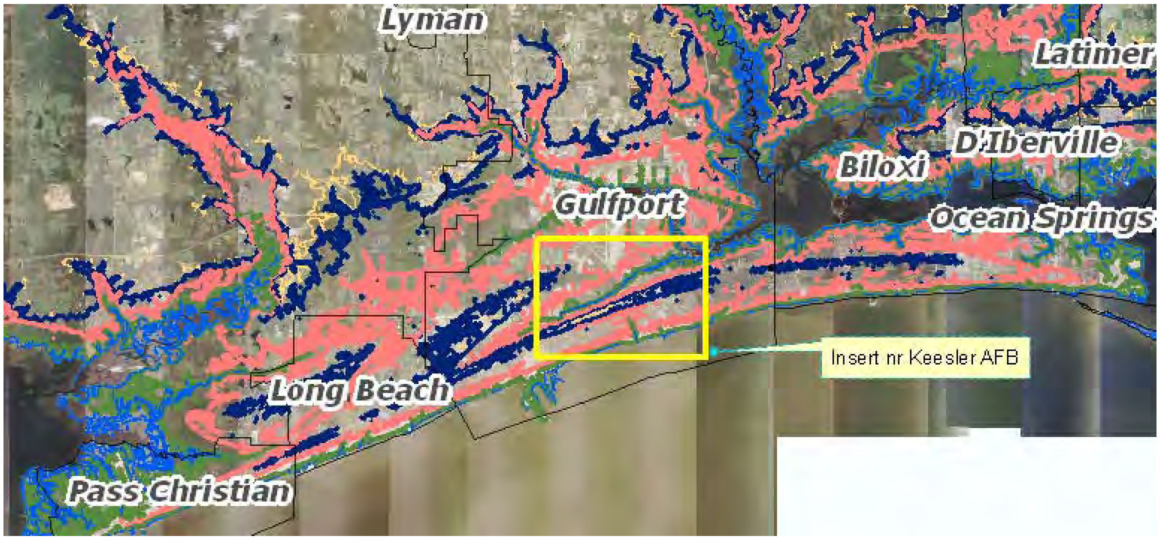


Figure 3.4.4-2. Existing Conditions Harrison County, MS

A close-up near Keesler Air Force Base is shown in Figure 3.4.4-3. The 4-ft(blue), 8-ft(dark green), 12-ft(light green), 16-ft(brown), 20-ft(pink), 24-ft(light purple), 28-ft (teal), and 32-ft (gold) ground contour lines are shown.

The area is drained by natural and some improved channels. Above the ridge water drains o the north, thence to either the Back Bay of Biloxi on the east side of the county, or to the west to the St Louis Bay. South of the ridge, the water drains to Mississippi Sound.

Drainage from ordinary rainfall is hindered on occasions when either of the rivers in the area or the gulf is high, but impacts from hurricanes are devastating.

Damage from Hurricane Katrina in August, 2005 in the Pascagoula area are shown in Figures 3.4.4-4 and 3.4.4-5. Many homes are still un-repaired, pending settlement of insurance claims.

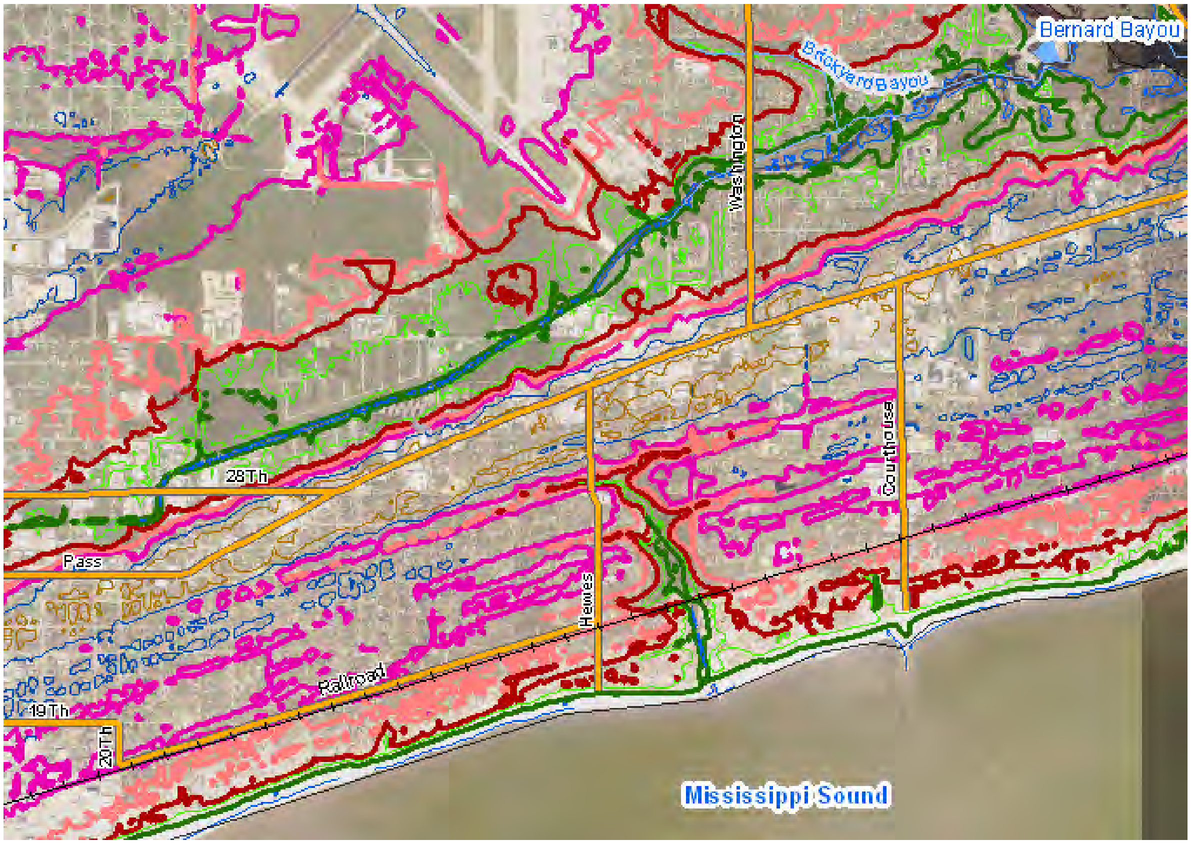
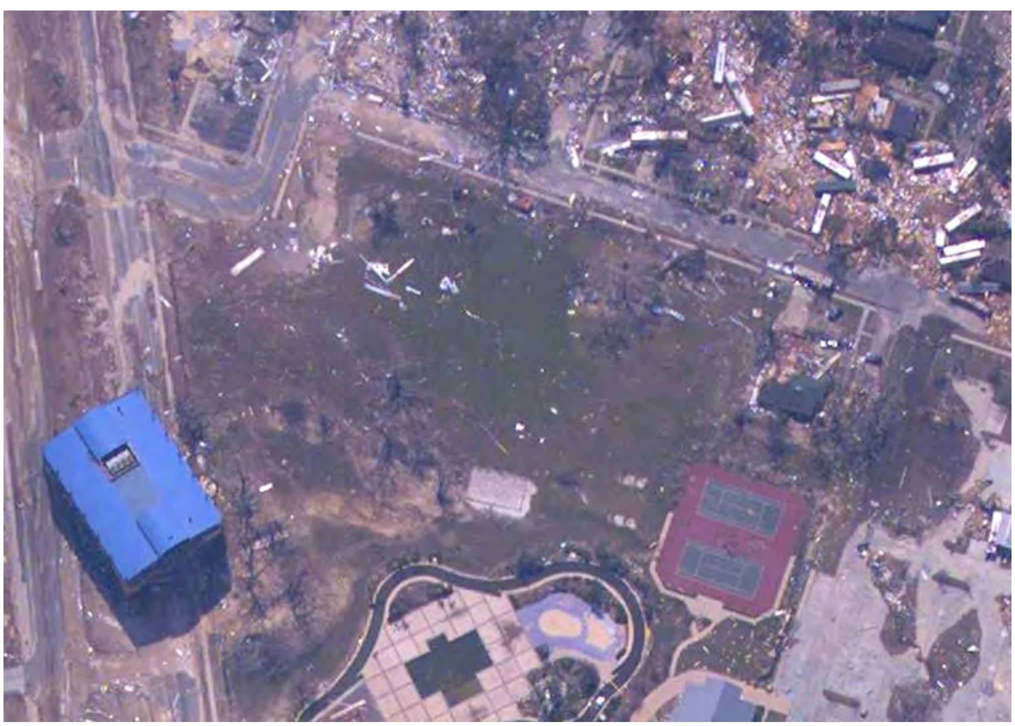


Figure 3.4.4-3. Existing Conditions Harrison County near Keesler AFB



Source: http://ngs.woc.noaa.gov/storms/katrina/24330924.jpg

Figure 3.4.4-4. Hurricane Katrina Damage Harrison County, MS

Source: danakay, http://www.flickr.com/photo_zoom.gne?id=45235550&size=m

Figure 3.4.4-5. Hurricane Katrina Damage Harrison County, MS

3.4.4.4 Coastal and Hydraulic Data

Typical coastal data is shown in Section 1.4, of this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the Katrina inundation limits are shown in Figures 3.4.4-6 and 3.4.4-7. The data indicates the Katrina high water was as high as 21 ft NAVD88 Biloxi, and 28 ft NAVD88 at Pass Christian.

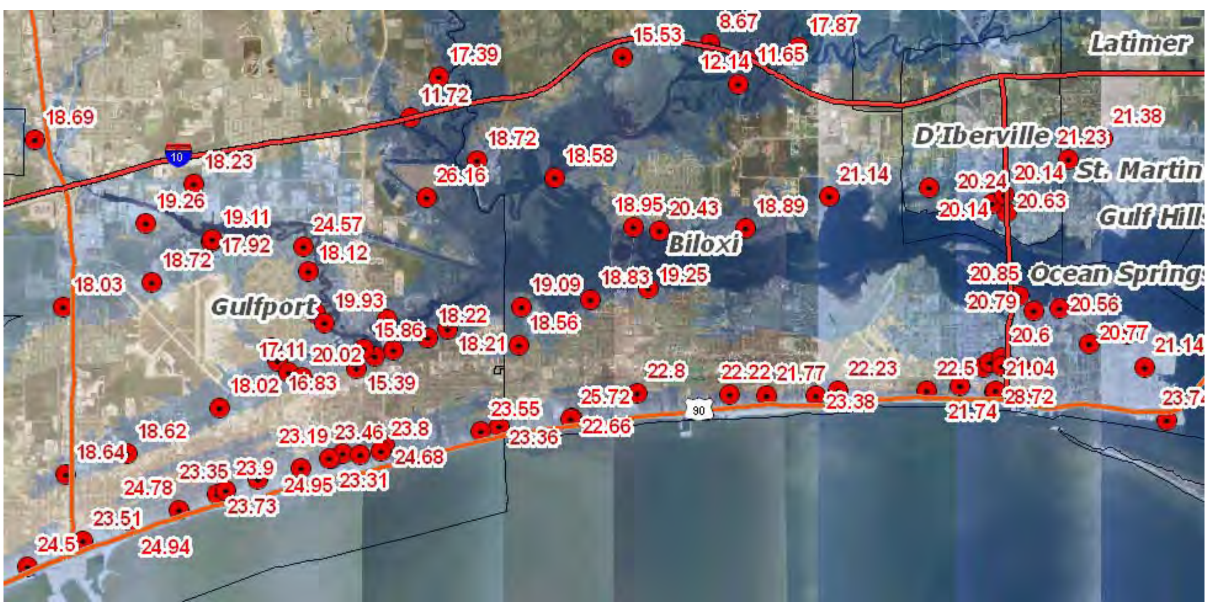


Figure 3.4.4-6. Hurricane Katrina High Water Elevations

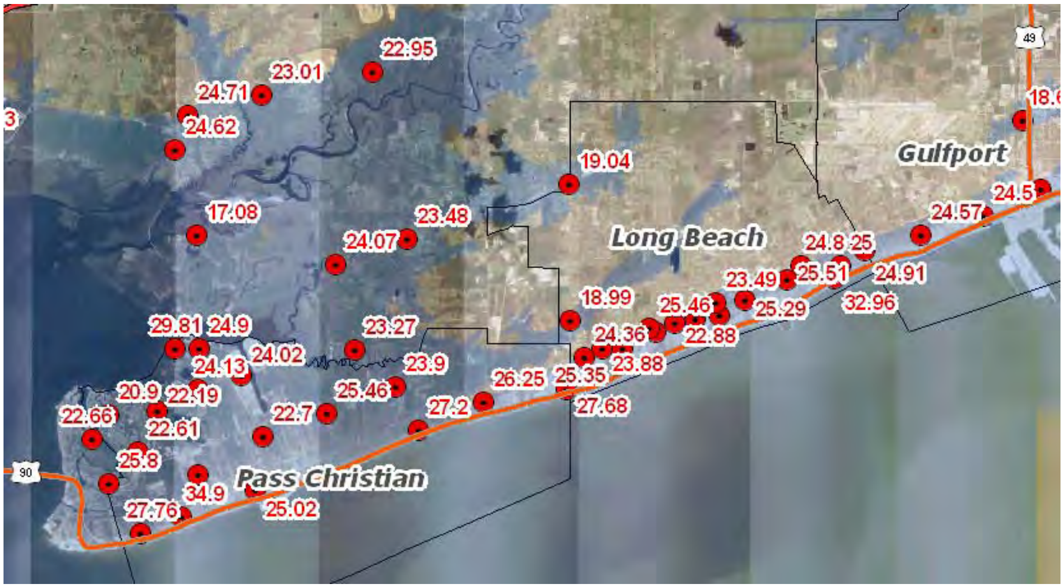


Figure 3.4.4-7. Hurricane Katrina High Water Elevations

A closer view at the intersection of Hwy 90 and US Hwy 49 in Gulfport of existing flooding potential along Harrison County is shown in Figure 3.4.4-8. Ground contours shown are 4-ft (blue), 8-ft (dark green), 12-ft (light green), 16-ft (brown), 20-ft (pink), 24-ft (light purple), 28-ft (teal), and 32-ft (gold).

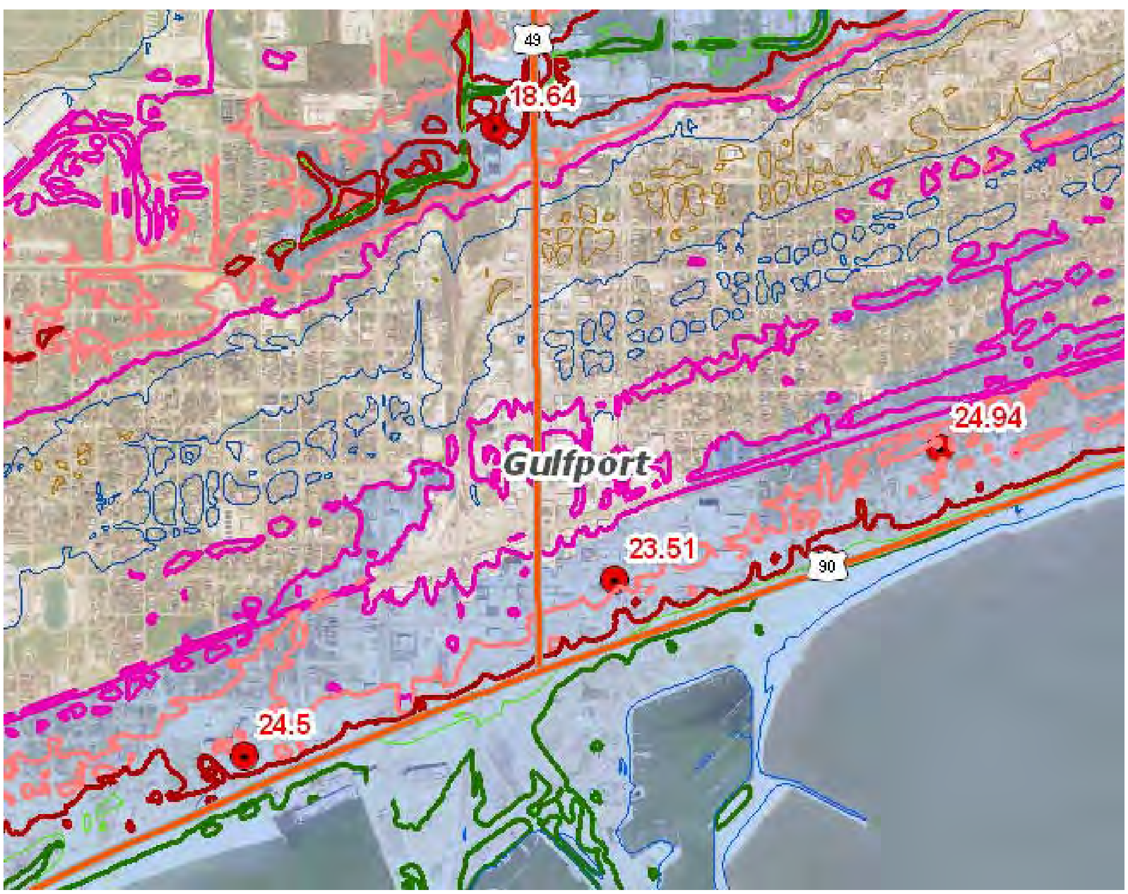


Figure 3.4.4-8. Ground Contours and Katrina High Water Elevations near Hwy 49

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown in Section 2.14 of the Engineering Appendix and in the Economic Appendix. Points near the coast in Harrison County at which data from hydrodynamic modeling was saved are shown in Figures 3.4.4-9 and 3.4.4-10.

Existing Condition Stage –Frequency data for Save Point 50, just off the coast of Harrison County, is shown in Figure 3.4.4-11. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

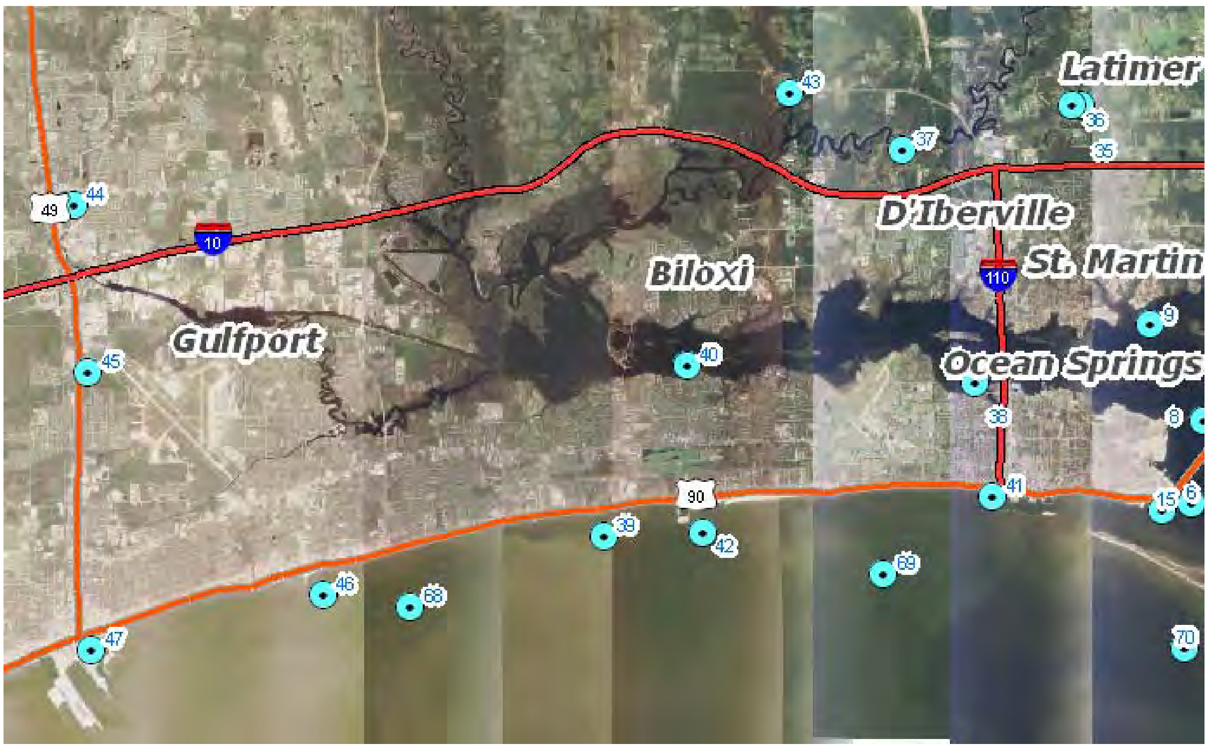


Figure 3.4.4-9. Hydrodynamic Modeling Save Points in Harrison County

Figure 3.4.4-10. Hydrodynamic Modeling Save Points in Harrison County

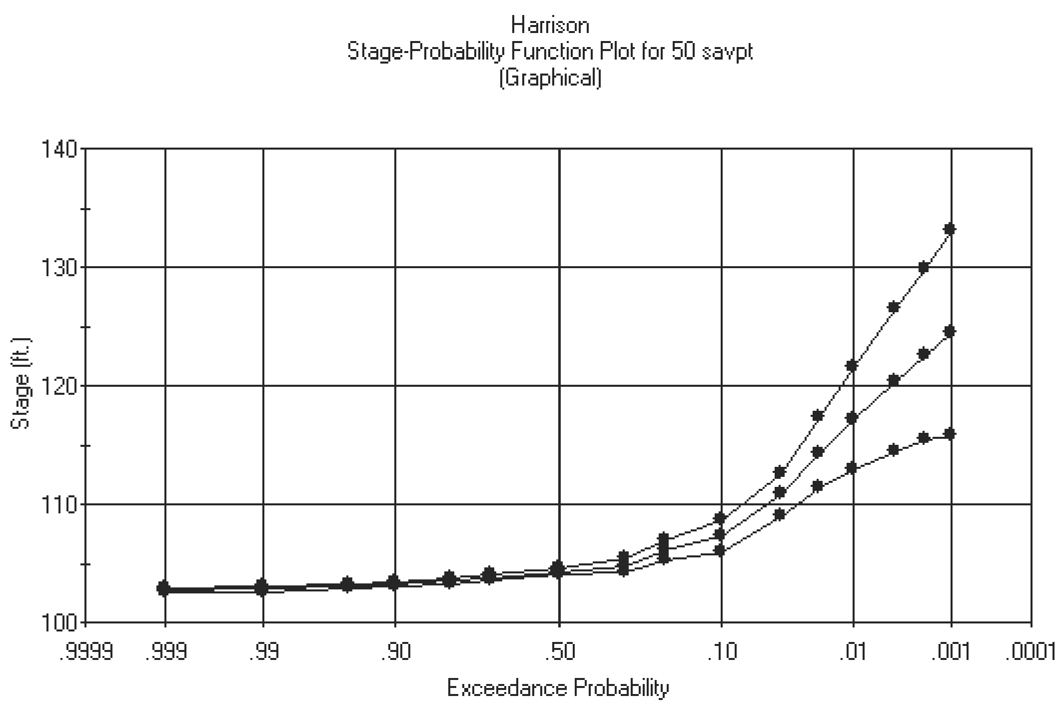


Figure 3.4.4-11. Existing Conditions at Save Point 50, near Pass Christian, MS

3.4.4.5 *Option A – Elevation 20 ft NAVD88*

This option consists of constructing a levee to elevation 20 ft NAVD88 along the coast of Harrison County as shown on Figures 3.4.4-12 through 3.4.4-14, along with the internal sub-basins and levee culvert/pump locations. Drainage basins 24 and 25 drain north against the levee. These sites will be ditched along the levee to St. Louis Bay.

Figure 3.4.4-12. Pump/Culvert/Sub-basin Site Locations

Figure 3.4.4-13. Pump/Culvert/Sub-basin Site Locations

Figure 3.4.4-14. Pump/Culvert/Sub-basin Site Locations

Drainage basin 26 drains north against the levee. This site will be ditched along the levee to Biloxi Bay.

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as shown in the Figure 3.4.4-15.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo: Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.4.4-15. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the top and back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown in Figure 3.4.4-16 was caused by approximately 1-2 ft of overtopping crest depth.

Source: ERDC, Steven Hughes

Figure 3.4.4-16. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

Revetment will be included in the levee design to prevent overtopping failure.

The levee will be protected by gabions on filter cloth as shown in Figure 3.4.4-17, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

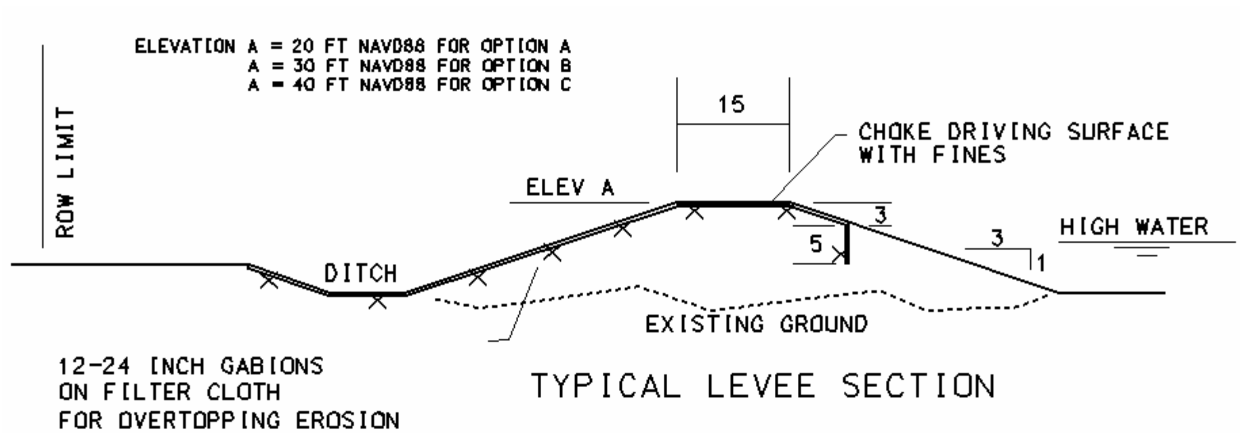


Figure 3.4.4-17. Typical Section at Levee

3.4.4.5.1 Interior Drainage

Drainage on the interior of the raised highway would be collected at the highway and channeled to culverts placed at locations shown above. The culverts would have flap gates on the seaward ends to prevent backflow when the water in Mississippi Sound is high. An additional closure gate would also be provided at every culvert for control in the event the flap gate malfunctions. A typical section is shown in Figure 3.4.4-18.

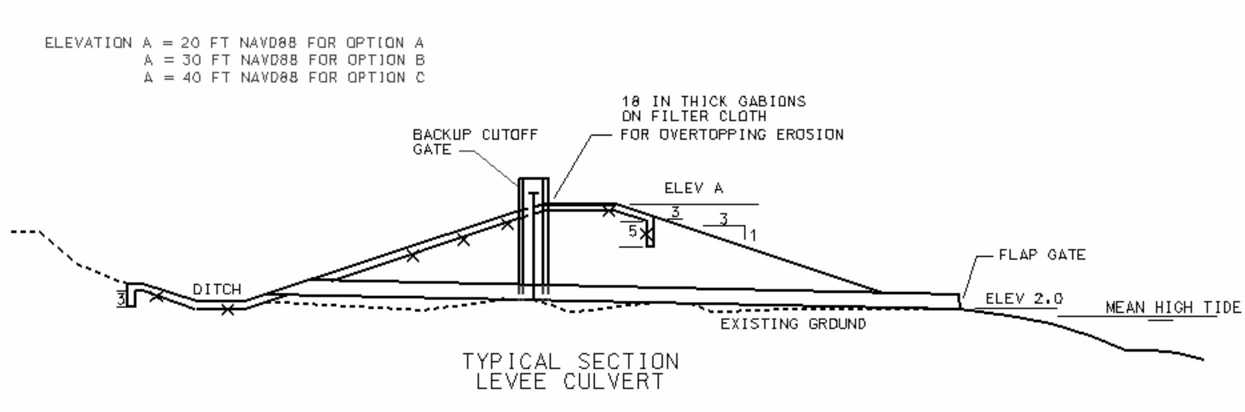


Figure 3.4.4-18. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the drainage basin into major sub-basins and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater/tailwater elevation difference was maintained at 3.0 ft or less. Drainage ditches along the toe of the highway will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is "Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico" US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is "National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)", R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report; therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

3.4.4.5.1.1 Surge Barrier

In order to prevent hurricane surges from circumventing the levee, surge barrier gates would be constructed across both Biloxi Bay and St. Louis Bay. In the event of an imminent hurricane, the gates across the Back Bay of Biloxi and St. Louis Bay would be closed, and flow from the rivers feeding these bays, as well as local runoff would pond behind the gates. The location of the barriers are shown in Figure 3.4.4-19 and 3.4.4-20.

The gates would be similar to the rising sector gates across the Thames River in London, England, shown in Figure 3.4.4-21.

The gates are described in more detail elsewhere in this report.

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) was used to model both the Biloxi Bay watershed and the St Louis Bay watershed in order to predict the maximum water elevation behind the gates in the bays under several different storm scenarios. These two basins will be described separately.

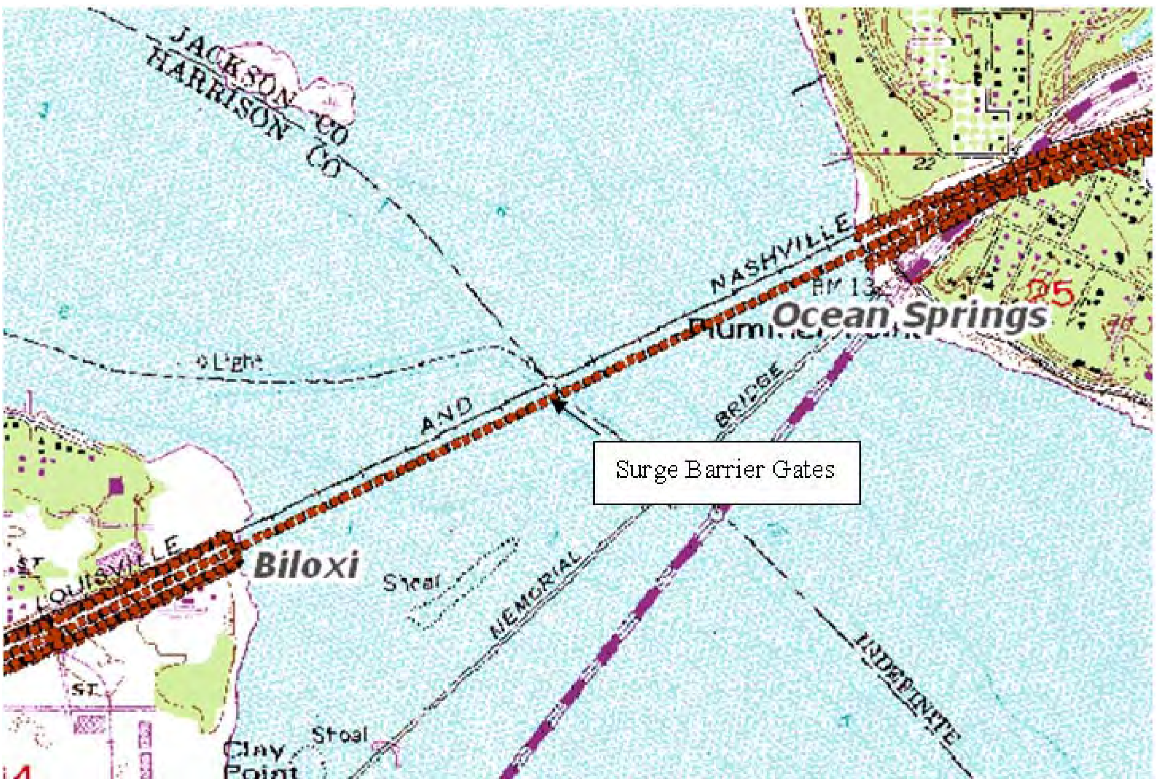


Figure 3.4.4-19. Biloxi Bay Surge Barrier Location

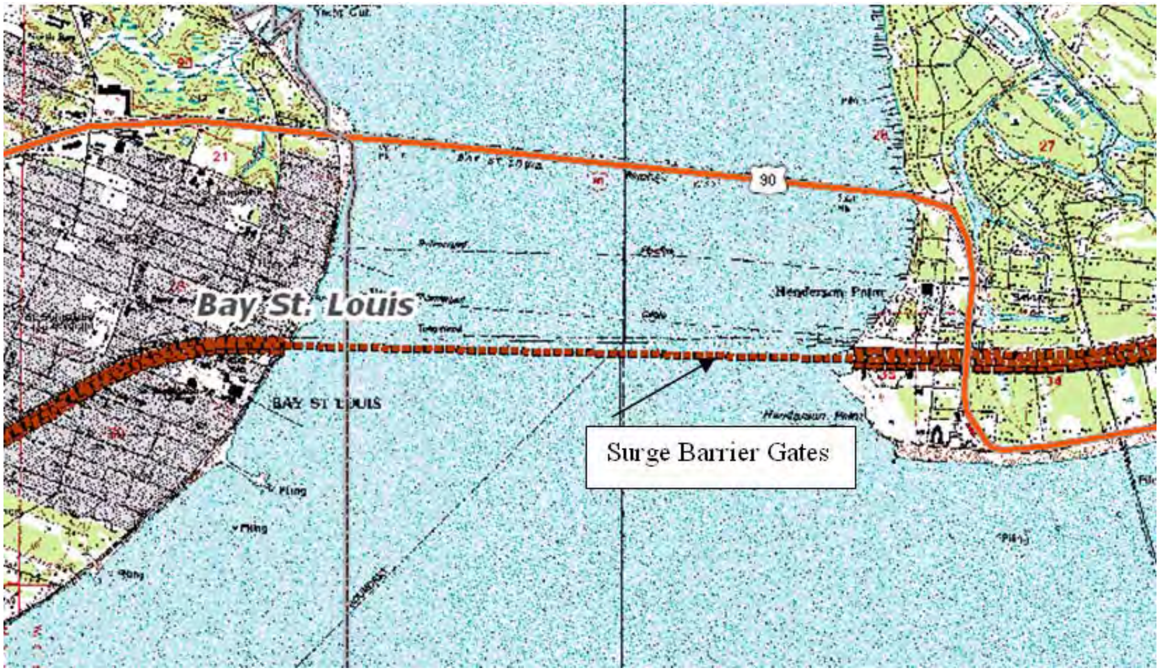


Figure 3.4.4-20. St Louis Bay Surge Barrier Location

Figure 3.4.4-21. Thames River Barrier Gates

3.4.4.5.1.2 Biloxi Bay Modeling

The Biloxi Bay watershed is an approximately 640 square mile watershed comprised of six subbasins that stretch across Harrison, Stone, and Jackson County, MS. There is one United States Geological Survey (USGS) discharge gage located in the watershed along the Biloxi River and one National Oceanic and Atmospheric Administration (NOAA) hourly precipitation gage located on the east side of the watershed. The discharge gage is USGS gage 2481000 at Wortham, MS and the precipitation gage is NOAA gage 107840 (Saucier Experimental Forest). Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering Biloxi Bay from the Biloxi Bay watershed for the 2-100 year rainfall events. The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The Biloxi Bay watershed is shown in Figure 3.4.4-22.

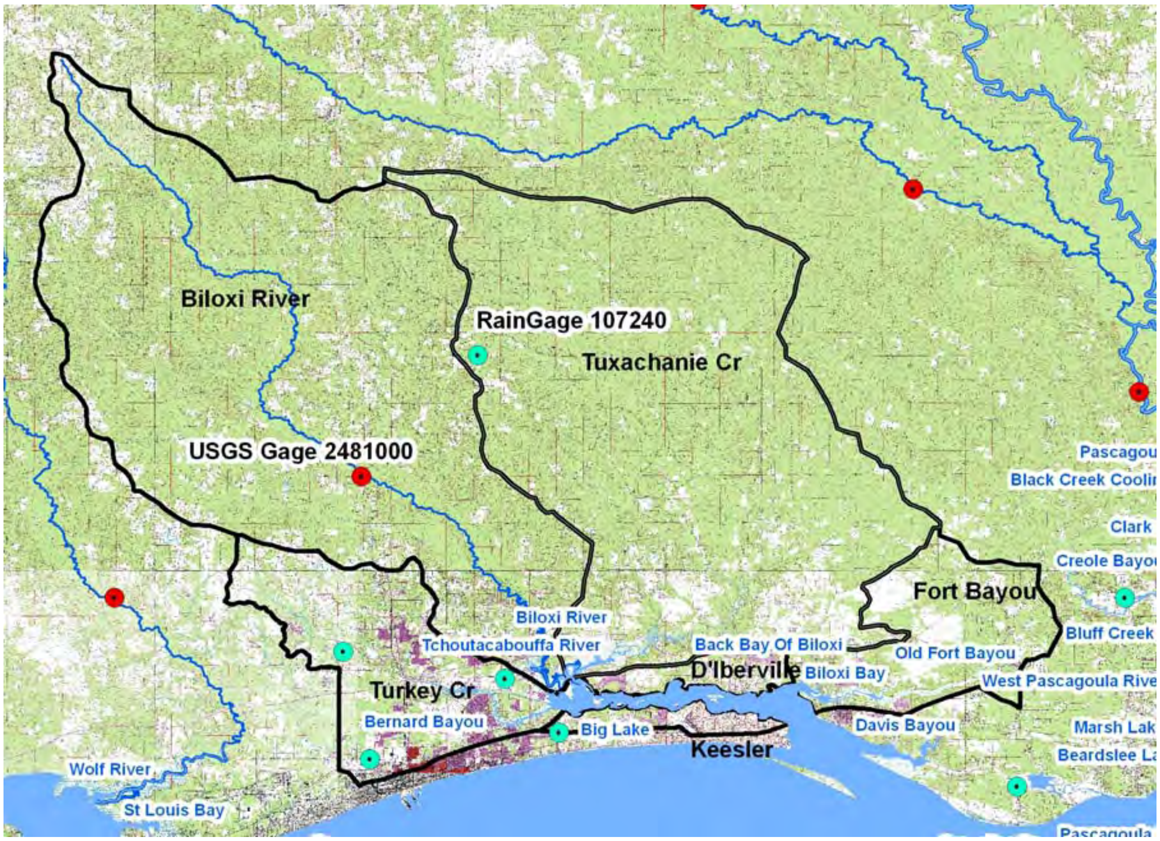


Figure 3.4.4-22. Biloxi Bay Watershed

The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gage 107840 and the 2-100 year 24-hour TP-40 rainfall events. For the loss model some basins used the initial and constant loss model and others (D'Iberville and Keesler) used SCS curve number method. For the direct runoff model, all the basins used the Snyder's unit hydrograph (UH) model. The model was calibrated to observed hourly discharge data for two events at USGS gage 2481000. The basin models used in the calibration used the initial constant loss model and Snyder's method for the direct runoff. The two calibration events (May 1991 and Jan 1993) had rainfall of about 6.4 inches and 7.6 inches each, corresponding to approximately 2-yr to 5-yr theoretical rainfall frequency.

Calibration results agree reasonable well with observed data as shown in Figures 3.4.4-23 and 3.4.4-24.

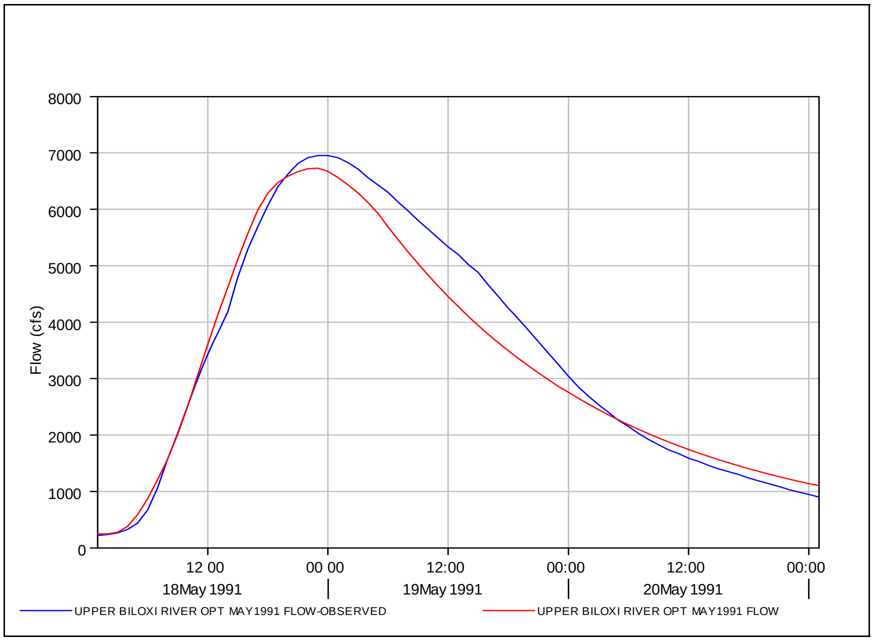


Figure 3.4.4-23. Biloxi Bay Watershed Calibration, May 18, 1991

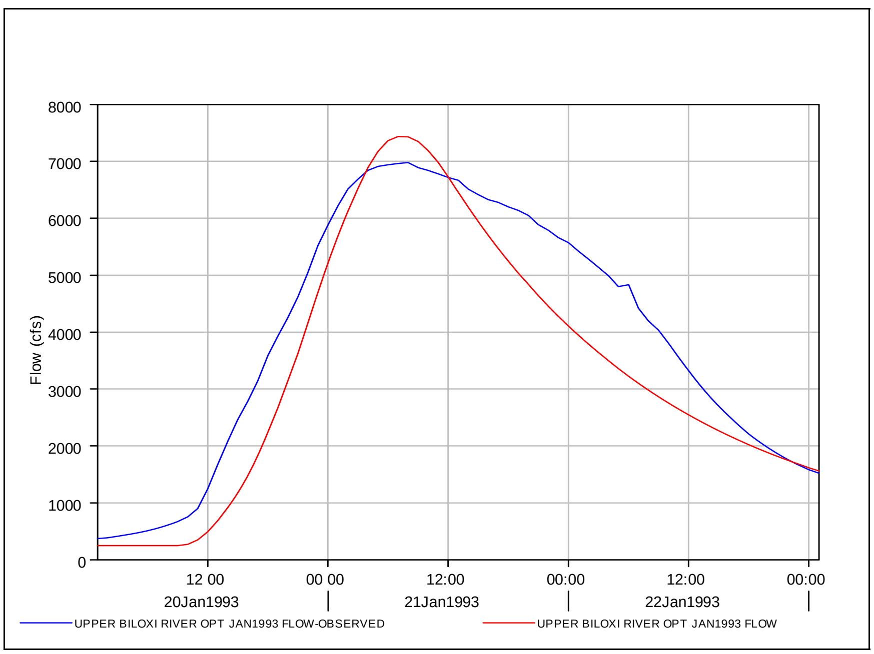


Figure 3.4.4-24. Biloxi Bay Watershed Calibration, Jan 21, 1993

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition, in accordance with studies cited above. The highest inflow period of the inflow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

Based on this method of analysis, the resulting elevations for the various storms are shown in Table 3.4.4-1, with the 10-yr elevation of 8.4 ft NAVD88 the design condition.

Table 3.4.4-1.
Biloxi Bay Ponding

Biloxi Bay 4 ft. Base Elevations	
Strom Event	**Bay Elevation (ft NAVD88)**
2-year	6.0
5-year	7.6
10-year	8.4
25-year	9.4
50-year	10.0
100-year	10.8

This ponded water area in Harrison County above the surge barrier gates at the 10-yr flood is at 8.4 ft NAVD88 and is approximated by the 8-ft ground contour line shown in Figure 3.4.4-25.

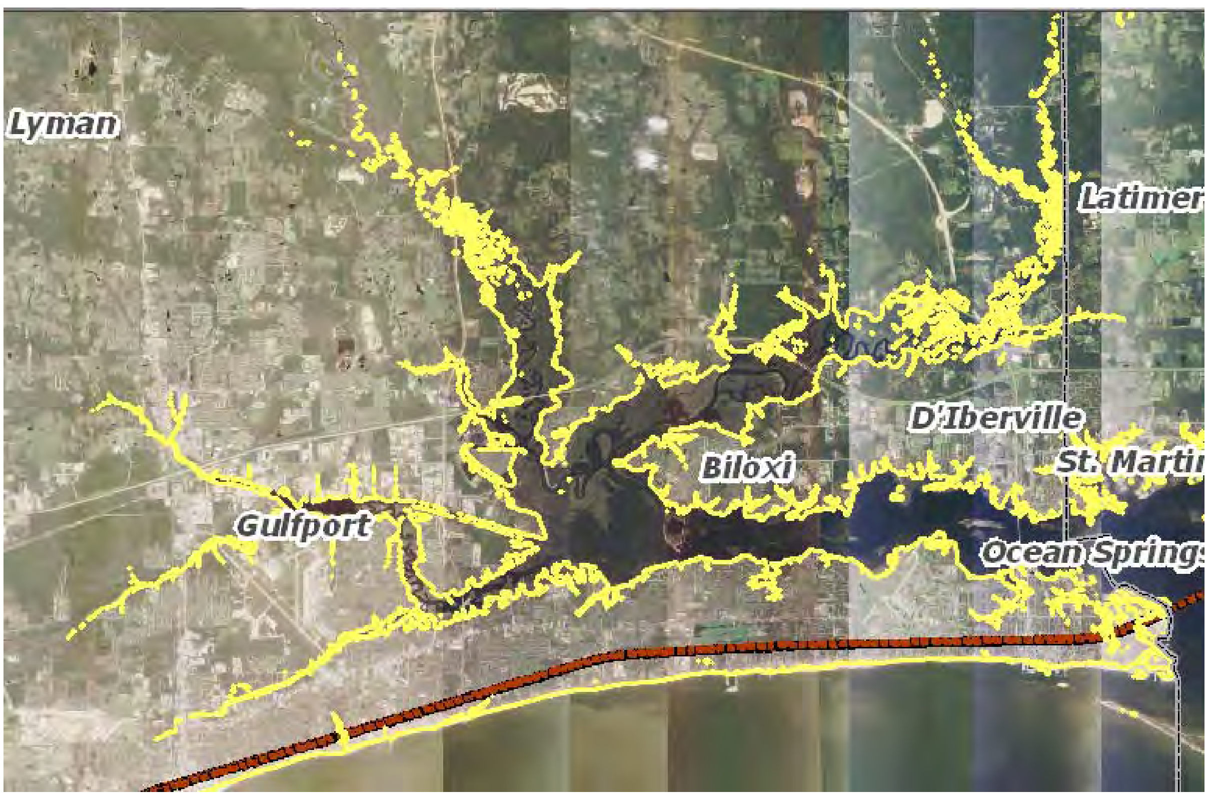


Figure 3.4.4-25. Biloxi Bay 10-yr Ponding to Elev. 8.4 ft NAVD88

3.4.4.5.1.3 St. Louis Bay Modeling

The St. Louis Bay watershed covers approximately 654 square miles and is comprised of six sub-basins that stretch across the Mississippi counties of Harrison, Hancock, Stone, and Pearl River.

There is one United States Geological Survey (USGS) discharge stream gage (#2481510) located in the watershed along the Wolf River, near Landon, Mississippi. There are three significant National Oceanic and Atmospheric Administration (NOAA) hourly precipitation gages located nearby to the watershed: #109617 White Sand located to the west, #87720 Purvis 2 N to the north, and #109617, 87720, and 107840 Saucier Experimental Forest to the east of the basin. Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering St. Louis Bay from the St. Louis Bay watershed for the 2 year, 5 year, 10 year, 25 year, 50 year and 100 year rainfall events. The St. Louis Bay watershed is shown in Figure 3.4.4-26.

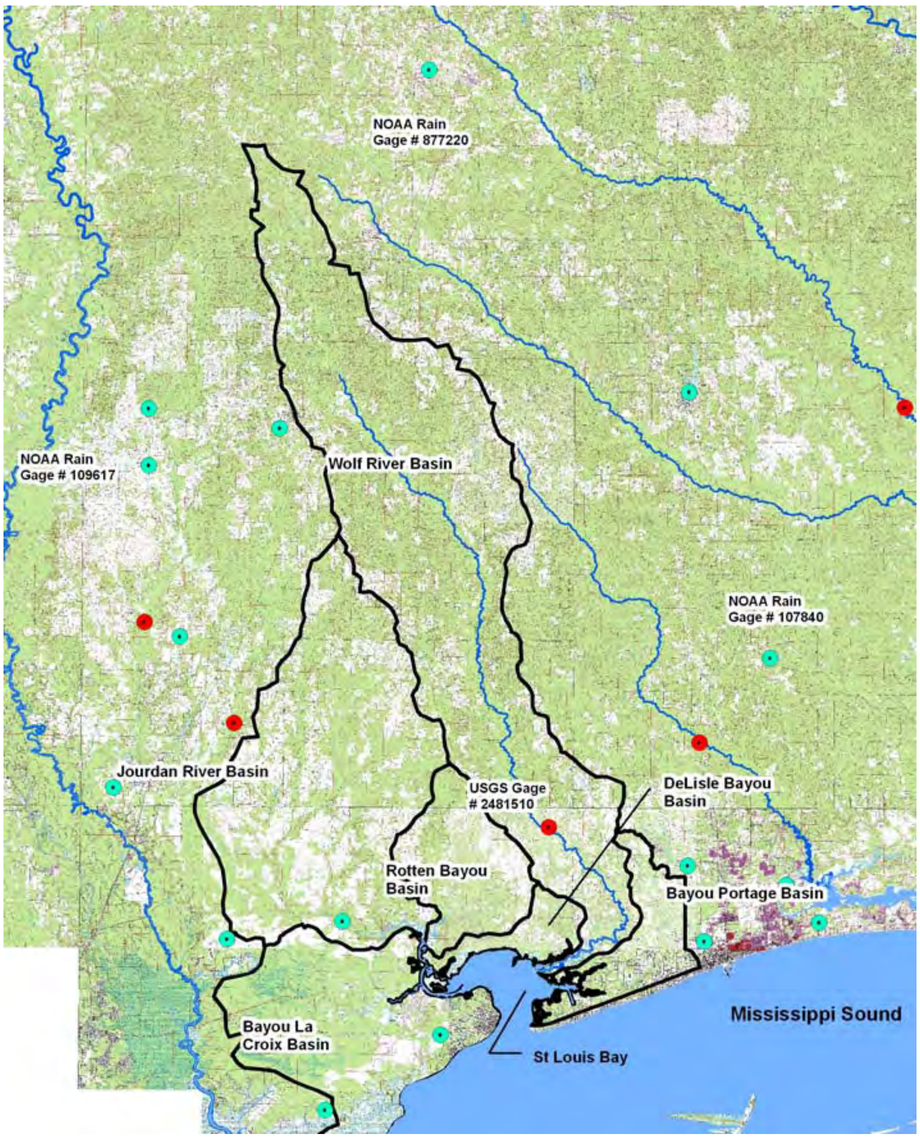


Figure 3.4.4-26. St Louis Bay Watershed

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gages 109617, 87720, and 109617, 87720, and 107840 and the 2-100 year 24-hour TP-40 rainfall events. The initial and constant loss rate method was used for the loss model while the Snyder's unit hydrograph (UH) method was used for the direct runoff model. The model was calibrated to observed hourly discharge data for one event at USGS gage 2481510. Several other events were analyzed but not used because the observed hourly precipitation for those events did not match the TP-40 rainfall.

The HEC-HMS St. Louis Bay watershed model was calibrated to the September 24-30, 2002 storm events. The model was calibrated at the Upper Wolf River sub-basin using observed precipitation data from NOAA gages 109617, 87720, and 107840 and observed discharge data from USGS gage 2481510. This event had a total rainfall of 13.75 inches and peak discharge of 17,854 cfs. This event was chosen due to the availability of both the hourly precipitation and discharge data. The observed and computed hydrographs are shown in Figure 3.4.4-27.

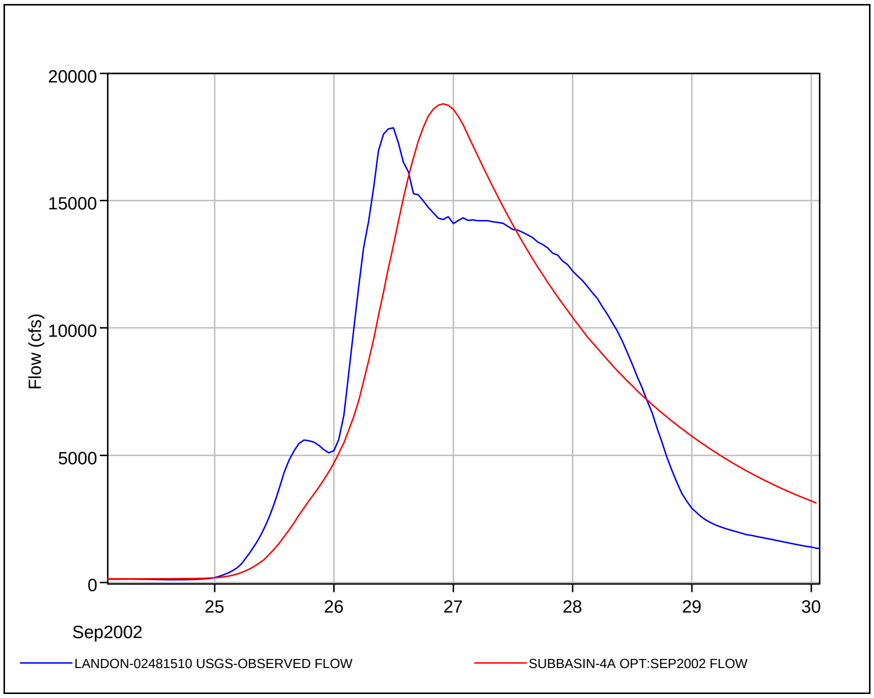


Figure 3.4.4-27. St. Louis Bay Watershed Calibration

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition, in accordance with studies cited above. The highest inflow period of the inflow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

Based on this method of analysis, the resulting elevations for the various storms are shown in Table 3.4.4-2, with the 10-yr elevation of 6.8 ft NAVD88 the design condition.

Table 3.4.4-2.
St. Louis Bay Ponding

St. Louis Bay 4 ft. Base Elevations	
Storm Event	**Bay Elevation (ft NAVD88)**
2-year	5.5
5-year	6.3
10-year	6.8
25-year	7.5
50-year	7.9
100-year	8.4

This ponded water area in Harrison County above the surge barrier gates is at the 10-yr flood elevation of 6.8 ft NAVD88, but is approximated by the 8-ft ground contour line shown in Figure 3.4.4-28.

Figure 3.4.4-28. St Louis Bay 10-yr Ponding to Elev. 8.4 ft NAVD88

3.4.4.5.2 Geotechnical Data

Geology: The Prairie formation is found southward of Interstate 10 and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

The Gulfport Formation is found along the coastline in most of Harrison County. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.4.5.3 Option A – Elevation 20 ft.NAVD88. Structural, Mechanical and Electrical

See sections 3.4.4.5.3.1 and 3.4.2.5.3.2.

3.4.4.5.3.1 Pumping Stations

Design hydraulic head derived for the one pumping facility included in the Harrison County Inland Barrier for the elevation 20 protection level was 15 feet and the corresponding flow required was 294,882 gallons per minute. The facility thus derived would consist of one plant having four, 60-inch diameter and 560 horsepower pumps.

3.4.2.5.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 20, 45 roadway intersections would have to be accommodated. For this study it was estimated that 18 roller gate structures and 27 swing gate structures would be required.

3.4.4.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.4.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.4.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred and is not applicable to this option.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied buildings and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. This security level will be applicable to this option.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.4.5.7 Operation and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.4.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.4.15, Cost Summary. Construction costs for the various options are included in Table 3.4.4-3 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.4-4. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.4.5.9 Schedule for Design and Construction

Because of the size and scope of the possible options, feasibility study level of detail could not be attained in this report. A significant additional detailed design effort will be required prior to attaining feasibility level, and construction would normally not proceed until that level is completed and an appropriate plan selected. After feasibility level design is complete and the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.4.6 Option B – Elevation 30 ft NAVD88

Option B is similar to option A except for the following items.

3.4.4.6.1 Option B – Elevation 30 ft.NAVD88. Interior Drainage

The alignment of the levee is the same as Option A, above, and is not reproduced here. Differences between the description of this option and preceding description of Option A include the height of the levee, pumping facilities (because of the increased head), and the length of the levee culverts. The methods of analysis for interior drainage and computed flows are the same as for Option A.

3.4.4.6.2 Option B – Elevation 30 ft.NAVD88. Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.4.4.6.3 Option B – Elevation 30 ft.NAVD88. Structural, Mechanical and Electrical

See sections 3.4.4.6.3.1 and 3.4.4.6.3.2.

3.4.4.6.3.1 Pumping Stations

Design hydraulic heads derived for the 7 pumping facilities included in the Harrison County Inland Barrier for the elevation 30 protection level varied from 5 and 20 feet and the corresponding flows required varied from 172,800 to 294,882 gallons per minute respectively. The facilities thus derived would consist of one plant having three, 54-inch diameter, 175 horsepower pumps and one having four, 60-inch diameter pumps each running at 750 horsepower.

3.4.4.6.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 30, 158 roadway/railway intersections would have to be accommodated. For this study it was estimated that 78 roller gate structures and 78 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.6.4 Option B – Elevation 30 ft.NAVD88. HTRW

The HTRW paragraphs for Option B are the same as for Option A, above.

3.4.4.6.5 Option B – Elevation 30 ft.NAVD88. Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option B are the same as for Option A, above.

3.4.4.6.6 Option B – Elevation 30 ft.NAVD88. Project Security

The Project Security paragraphs for Option B are the same as for Option A, above

3.4.4.6.7 Option B – Elevation 30 ft.NAVD88. Operations and Maintenance

The Operation and Maintenance paragraphs for Option B are the same as for Option A, above.

3.4.4.6.8 Option B – Elevation 30 ft.NAVD88. Cost Estimate

The Cost Estimate paragraphs for Option B are the same as for Option A, above.

3.4.4.6.9 Option B – Elevation 30 ft.NAVD88. Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option B are the same as for Option A, above.

3.4.4.7 Option C – Elevation 40 ft NAVD88

Option C is similar to option A except for the following items.

3.4.4.7.1 Interior Drainage

The alignment of the levee is the same as Option A, above, and is not reproduced here. Differences between the description of this option and preceding description of Option A include the height of the levee, pumping facilities (because of the increased head), and the length of the levee culverts. The methods of analysis for interior drainage and computed flows are the same as for Option A.

3.4.4.7.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option B are the same as for Option A, above.

3.4.4.7.3 Structural, Mechanical and Electrical

See sections 3.4.4.7.3.1 and 3.4.2.7.3.2.

3.4.4.7.3.1 Pumping Stations

Design hydraulic heads derived for the 7 pumping facilities included in the Harrison County Inland Barrier for the elevation 40 protection level varied from 15 and 30 feet and the corresponding flows required varied from 172,800 to 294,882 gallons per minute respectively. The facilities thus derived would consist of one plant having three, 54-inch diameter, 420 horsepower pumps and one having four, 60-inch diameter pumps each running at 1150 horsepower.

3.4.2.7.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 40, 161 roadway/railway intersections would have to be accommodated. For this study it was estimated that 1 roller gate structure and 158 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.7.4 HTRW

The HTRW paragraphs for Option C are the same as for Option A, above.

3.4.4.7.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Planl paragraphs for Option C are the same as for Option A, above.

3.4.4.7.6 Project Security

The Project Security paragraphs for Option C are the same as for Option A, above.

3.4.4.7.7 Operations and Maintenance

The Operations and Maintenance paragraphs for Option C are the same as for Option A, above.

3.4.4.7.8 Cost Estimate

The Cost Estimate paragraphs for Option C are the same as for Option A, above.

3.4.4.7.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option C are the same as for Option A, above.

3.4.4.8 Option D – Levee for Roadway, Elevation 20 ft NAVD88

Option D is similar to option A except for the following items.

3.4.4.8.1 Interior Drainage

The alignment of the levee is the same as Option A, above, and is not reproduced here. The difference between this option and Option A is that the width of the top of the levee in Harrison County is 75 ft for Option D and 15 ft for Option A. This will allow Hwy 90 to be relocated along the top of the levee. The methods of analysis for interior drainage and computed flows are the same as for Option A.

3.4.4.8.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option D are the same as for Option A, above.

3.4.4.8.3 Structural, Mechanical and Electrical

See sections 3.4.4.8.3.1 and 3.4.4.8.3.2.

3.4.4.8.3.1 Pumping Stations

Design hydraulic head derived for the one pumping facility included in the Harrison County Inland Barrier for the elevation 20 protection level was 15 feet and the corresponding flow required was 294,882 gallons per minute. The facility thus derived would consist of one plant having four, 60-inch diameter and 560 horsepower pumps.

3.4.4.8.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 20, 42 roadway/railway intersections would have to be accommodated. For this study it was estimated that 18 roller gate structures and 48 swing gate structures would be required at the roadway crossings.

3.4.4.8.4 HTRW

The HTRW paragraphs for Option D are the same as for Option A, above.

3.4.4.8.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option D are the same as for Option A, above.

3.4.4.8.6 Project Security

The Project Security paragraphs for Option D are the same as for Option A, above

3.4.4.8.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option D are the same as for Option A, above.

3.4.4.8.8 Cost Estimate

The Cost Estimate paragraphs for Option D are the same as for Option A, above.

3.4.4.8.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option D are the same as for Option A, above.

3.4.4.9 Option E – Levee for Roadway, Elevation 30 ft NAVD88

Option E is similar to option A except for the following items.

3.4.4.9.1 Interior Drainage

The alignment of the levee is the same as Option A, above, and is not reproduced here. The difference between this option and Option A is that the width of the top of the levee in Harrison County is 75 ft for Option E and 15 ft for Option A. In addition, the height of the levee is at 30 ft NAVD88 for Option E and 20 ft NAVD88 for Option A. The added width will allow Hwy 90 to be relocated along the top of the levee. The methods of analysis for interior drainage and computed flows are the same as for Option A.

3.4.4.9.2 Option E – Levee for Roadway, Elevation 30 ft.NAVD88. Geotechnical Data

The Geology and Geotechnical paragraphs for Option E are the same as for Option A, above.

3.4.4.9.3 Structural, Mechanical and Electrical

See setions 3.4.4.9.3.1 through 3.4.4.9.3.2.

3.4.4.9.3.1 Pumping Stations

Design hydraulic heads derived for the 7 pumping facilities included in the Harrison County Inland Barrier for the elevation 30 protection level varied from 5 and 20 feet and the corresponding flows required varied from 172,800 to 294,882 gallons per minute respectively. The facilities thus derived would consist of one plant having three, 54-inch diameter, 175 horsepower pumps and one having four, 60-inch diameter pumps each running at 750 horsepower.

3.4.4.9.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 30, 140 roadway/railway intersections would have to be accommodated. For this study it was estimated that 82 roller gate structures and 112 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.9.4 HTRW

The HTRW paragraphs for Option E are the same as for Option A, above.

3.4.4.9.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option E are the same as for Option A, above.

3.4.4.9.6 Project Security

The Project Security paragraphs for Option E are the same as for Option A, above

3.4.4.9.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option E are the same as for Option A, above.

3.4.4.9.8 Cost Estimate

The Cost Estimate paragraphs for Option E are the same as for Option A, above.

3.4.4.9.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option E are the same as for Option A, above.

3.4.4.10 Option F – Menge Avenue Alternate Route, Elevation 20 ft NAVD88

Option F is similar to Option A except for the following items.

3.4.4.10.1 Interior Drainage

The alignment of the levee is the same as Option A on the east side of Harrison County but extends to the north along Menge Avenue as shown on Figures 3.4.4-29 through 3.4.4-31 instead of continuing westward. These figures also show the pump/culvert locations and the sub-basins. For Option F, culverts are required at all sub-basins, but no pumps are required for sub-basins M3 – M8. The methods of analysis for interior drainage and computed flows are the same as for Option A, except that no surge barrier was included or evaluated for St Louis Bay.

3.4.4.10.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option F are the same as for Option A, above.

3.4.4.10.3 Structural, Mechanical and Electrical

See sections 3.4.4.10.3.1 and 3.4.4.10.3.2.

3.4.4.10.3.1 Pumping Stations

Design hydraulic head derived for the two pumping facilities included in the Harrison County Inland Barrier for the elevation 20 protection level was 16 feet, and the corresponding flow required varied from 555,626 and 772,358 gallons per minute. The facilities thus derived would consist of one plant having eleven, 42-inch diameter, 290 horsepower pumps, and one having thirteen, 48-inch diameter, 340 horsepower pumps.

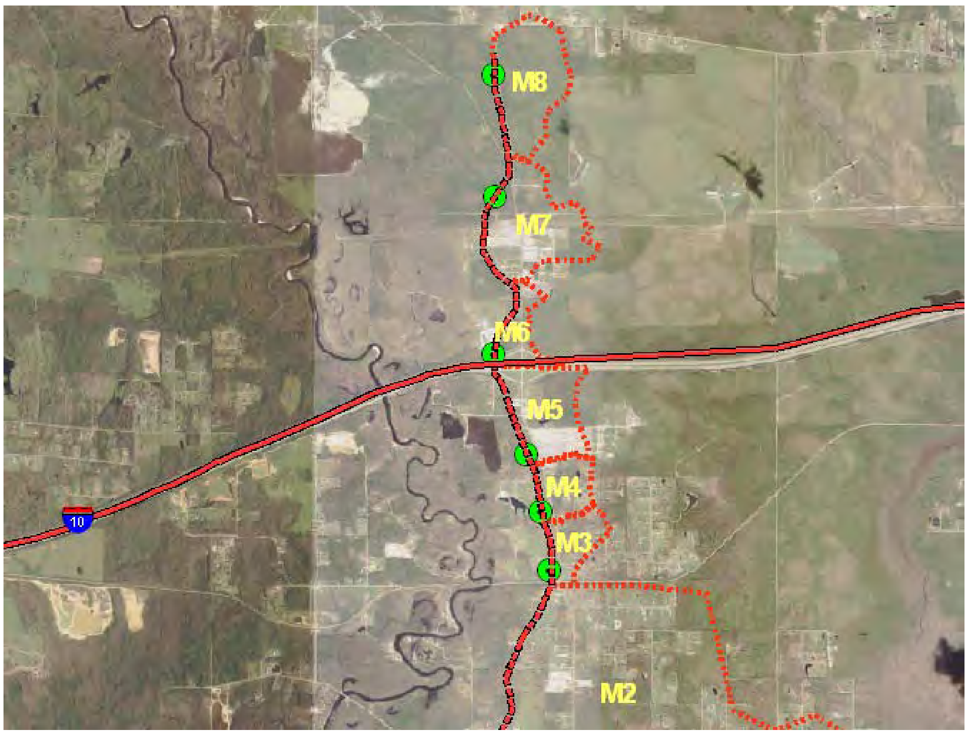


Figure 3.4.4-29. Menge Avenue Alternate Route, Pump/Culvert, Sub-basin Site Locations

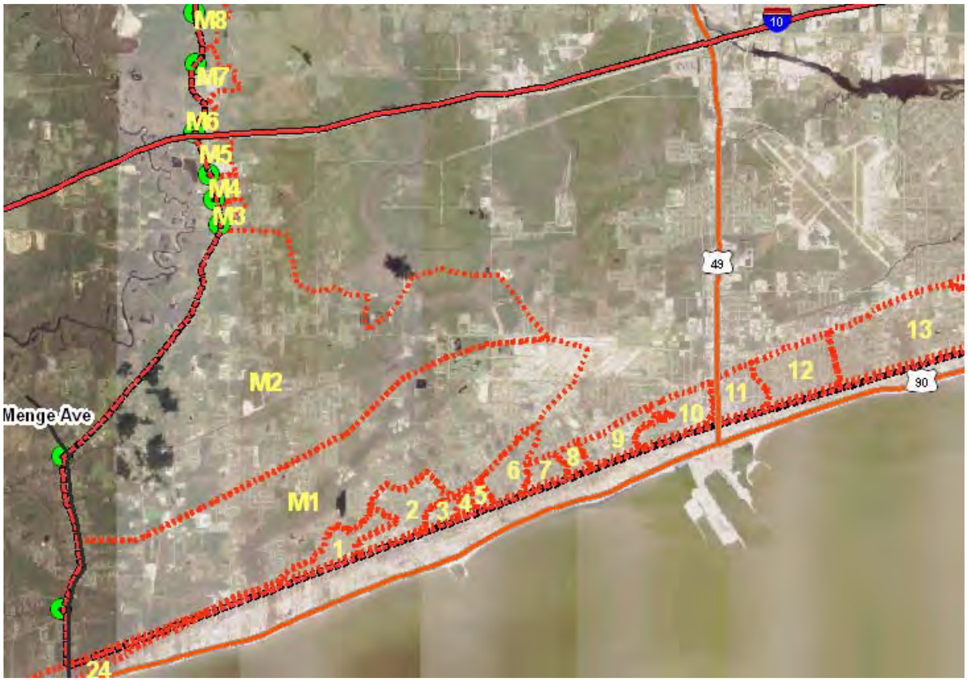


Figure 3.4.4-30. Menge Avenue Alternate Route, Pump/Culvert, Sub-basin Site Locations

Figure 3.4.4-31. Menge Avenue Alternate Route, Pump/Culvert, Sub-basin Site Locations

3.4.4.10.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 20, 21 roadway/railway intersections would have to be accommodated. For this study it was estimated that 17 roller gate structures and 4 swing gate structures would be required at the roadway crossings.

3.4.4.10.4 HTRW

The HTRW paragraphs for Option F are the same as for Option A, above.

3.4.4.10.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option F are the same as for Option A, above.

3.4.4.10.6 Project Security

The Project Security paragraphs for Option F are the same as for Option A, above

3.4.4.10.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option F are the same as for Option A, above.

3.4.4.10.8 Cost Estimate

The Cost Estimate paragraphs for Option F are the same as for Option A, above.

3.4.4.10.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option F are the same as for Option A, above.

3.4.4.11 Option G – Menge Avenue Alternate Route, Elevation 30 ft NAVD88

Option G is similar to option A except for the following items.

3.4.4.11.1 Interior Drainage

The alignment of the levee is the same as Option F, shown above in Figures 3.4.4-29 through 3.4.4-31 and is not repeated here. The primary difference between this option and Option F is the height of the levee. Option F levee height is elevation 20 ft NAVD88 and Option G levee height is elevation 30 ft NAVD88. For this option, culverts are required at all sub-basins, but no pumps are required for sub-basins M3 – M8. The methods of analysis for interior drainage and computed flows are the same as for Option A, except that no surge barrier was included or evaluated for St Louis Bay.

3.4.4.11.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option G are the same as for Option A, above.

3.4.4.11.3 Structural, Mechanical and Electrical

See sections 3.4.4.11.3.1 and 3.4.4.11.3.2.

3.4.4.11.3.1 Pumping Stations

Design hydraulic head derived for the two pumping facilities included in the Harrison County Inland Barrier for the elevation 30 protection level was 26 feet, and the corresponding flow required varied from 555,626 and 772,358 gallons per minute. The facilities thus derived would consist of one plant having eleven, 42-inch diameter, 475 horsepower pumps, and one having thirteen, 48-inch diameter, 600 horsepower pumps.

3.4.4.11.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 30, 125 roadway/railway intersections would have to be accommodated. For this study it was estimated that 86 roller gate structures and 37 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.11.4 HTRW

The HTRW paragraphs for Option G are the same as for Option A, above.

3.4.4.11.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option G are the same as for Option A, above.

3.4.4.11.6 Project Security

The Project Security paragraphs for Option G are the same as for Option A, above

3.4.4.11.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option G are the same as for Option A, above.

3.4.4.11.8 Cost Estimate

The Cost Estimate paragraphs for Option G are the same as for Option A, above.

3.4.4.11.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option G are the same as for Option A, above.

3.4.4.12 Option H – Menge Avenue Alternate Route, Elevation 40 ft NAVD88

Option H is similar to option A except for the following items.

3.4.4.12.1 Interior Drainage

The alignment of the levee is the same as Option F, shown above in Figures 3.4.4-29 through 3.4.4-31 and is not repeated here. The primary difference between this option and Option F is the height of the levee. Option F levee height is elevation 20 ft NAVD88 and Option H levee height is Elevation 40 ft NAVD88. For this option, culverts are required at all sub-basins, but no pump is required for sub-basin M8. The methods of analysis for interior drainage and computed flows are the same as for Option A, except that no surge barrier was included or evaluated for St Louis Bay.

3.4.4.12.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option H are the same as for Option A, above.

3.4.4.12.3 Structural, Mechanical and Electrical

See sections 3.4.4.12.3.1 and 3.4.4.12.3.2.

3.4.4.12.3.1 Pumping Stations

Design hydraulic head derived for the 7 pumping facilities included in the Harrison County Inland Barrier for the elevation 40 protection level varied from 10 to 36 feet, and the corresponding flow required varied from 46,225 to 772,358 gallons per minute. The facilities thus derived would consist of one plant having two, 36-inch diameter, 125 horsepower pumps, and one having thirteen, 48-inch diameter, 800 horsepower pumps.

3.4.4.12.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 40, 157 roadway/railway intersections would have to be accommodated. For this study it was estimated that 3 roller gate structures and 152 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.12.4 HTRW

The HTRW paragraphs for Option H are the same as for Option A, above.

3.4.4.12.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option H are the same as for Option A, above.

3.4.4.12.6 Project Security

The Project Security paragraphs for Option H are the same as for Option A, above

3.4.4.12.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option H are the same as for Option A, above.

3.4.4.12.8 Cost Estimate

The Cost Estimate paragraphs for Option H are the same as for Option A, above.

3.4.4.12.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option H are the same as for Option A, above.

3.4.4.13 Option I – Levee for Roadway with Menge Avenue Alternate Route, Elevation 20 ft NAVD88

Option I is similar to option A except for the following items.

3.4.4.13.1 Interior Drainage

The alignment of the levee is the same as Option F, shown above in Figures 3.4.4-29 through 3.4.4-31 and is not repeated here. The primary difference between this option and Option F is the top width of the east-west leg of the levee (Biloxi Bay to Menge Avenue). The east-west leg of Option F barrier top width is 15 ft and the east-west leg of Option I barrier top width is 75 ft. This will allow Hwy 90 to be relocated along the top of the levee. For this option, culverts are required at all sub-basins, but no pumps are required for sub-basins M3 - M8. The methods of analysis for interior drainage and computed flows are the same as for Option A, except that no surge barrier was included or evaluated for St Louis Bay.

3.4.4.13.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option I are the same as for Option A, above.

3.4.4.13.3 Structural, Mechanical and Electrical

See section 3.4.4.13.3.1 and 3.4.4.13.3.2.

3.4.4.13.3.1 Pumping Stations

Design hydraulic head derived for the fourteen pumping facilities included in the Harrison County Inland Barrier for the elevation 20 protection level varied from 8 to 18 feet, and the corresponding flow required varied from 62,388 to 490,083 gallons per minute. The facilities thus derived would consist of one plant having two, 36-inch diameter, 125 horsepower pumps, and one having seven, 54-inch diameter, 290 horsepower pumps.

3.4.4.13.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to elevation 20, 20 roadway/railway intersections would have to be accommodated. For this study it was estimated that 16 roller gate structures and 4 swing gate structures would be required at the roadway crossings.

3.4.4.13.4 HTRW

The HTRW paragraphs for Option I are the same as for Option A, above.

3.4.4.13.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option I are the same as for Option A, above.

3.4.4.13.6 Project Security

The Project Security paragraphs for Option I are the same as for Option A, above

3.4.4.13.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option I are the same as for Option A, above.

3.4.4.13.8 Cost Estimate

The Cost Estimate paragraphs for Option I are the same as for Option A, above.

3.4.4.13.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option I are the same as for Option A, above.

3.4.4.14 Option J – Levee for Roadway with Menge Avenue Alternate Route, Elevation 30 ft NAVD88

Option J is similar to option A except for the following.

3.4.4.14.1 Interior Drainage

The alignment of the levee is the same as Option F, shown above in Figures 3.4.4-29 through 3.4.4-31 and is not repeated here. The primary difference between this option and Option F is the top width of the east-west leg of the levee (Biloxi Bay to Menge Avenue). The east-west leg of Option F barrier top width is 15 ft and the east-west leg of Option J barrier top width is 75 ft. This will allow Hwy 90 to be relocated along the top of the levee. In addition, the height of this Option J is at elevation 30 ft NAVD88. For this option, culverts are required at all sub-basins, but no pumps are required for sub-basins M3 - M8. The methods of analysis for interior drainage and computed flows are the same as for Option A, except that no surge barrier was included or evaluated for St Louis Bay.

3.4.4.14.2 Geotechnical Data

The Geology and Geotechnical paragraphs for Option J are the same as for Option A, above.

3.4.4.14.3 Structural, Mechanical and Electrical

See sections 3.4.4.14.3.1 and 3.4.4.14.3.2.

3.4.4.14.3.1 Pumping Stations

Design hydraulic head derived for the two pumping facilities included in the Harrison County Inland Barrier for the elevation 30 protection level varied from 15 to 28 feet, and the corresponding flow required varied 62,388 to 490,083 gallons per minute. The facilities thus derived would consist of one plant having two, 36 inch diameter, 250 horsepower pumps, and one having five, 60-inch diameter, 1145 horsepower pumps.

3.4.4.14.3.2 Levee and Roadway/Railway Intersections

With the installation of protection to Elevation 30, 123 roadway/railway intersections would have to be accommodated. For this study it was estimated that 86 roller gate structures and 35 swing gate structures would be required at the roadway crossings. In addition, two railway closure gates would be required.

3.4.4.14.4 HTRW

The HTRW paragraphs for Option J are the same as for Option A, above.

3.4.4.14.5 Construction Procedures and Water Control Plan

The Construction Procedures and Water Control Plan paragraphs for Option J are the same as for Option A, above.

3.4.4.14.6 Project Security

The Project Security paragraphs for Option J are the same as for Option A, above

3.4.4.14.7 Operations and Maintenance

The Operation and Maintenance paragraphs for Option J are the same as for Option A, above.

3.4.4.14.8 Cost Estimate

The Cost Estimate paragraphs for Option J are the same as for Option A, above.

3.4.4.14.9 Schedule and Design for Construction

The Schedule for Design and Construction paragraphs for Option J are the same as for Option A, above.

3.4.4.15 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.4.4-3.
Harrison Co Inland Barrier Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$435,800,000
Option B – Elevation 30 ft NAVD88	$731,600,000
Option C – Elevation 40 ft NAVD88	$947,100,000
Option D – Roadway El 20 NAVD88	$205,400,000
Option E – Roadway El 30 NAVD88	$768,300,000
Option F – Menge El 20 NAVD88	$140,400,000
Option G – Menge El 30 NAVD888	$317,100,000
Option H – Menge El 40 NAVD88	$506,300,000
Option I – Road/Menge El 20 ft NAVD88	$178,600,000
Option J – Road/Menge El 30 ft NAVD88	$462,900,000

Table 3.4.4-4.
Harrison Co Inland Barrier O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$2,007,000
Option B – Elevation 30 ft NAVD88	$5,805,000
Option C – Elevation 40 ft NAVD88	$8,343,000
Option D – Roadway El 20 NAVD88	$1,868,000
Option E – Roadway El 30 NAVD88	$5,871,000
Option F – Menge El 20 NAVD88	$1,800,000
Option G – Menge El 30 NAVD888	$4,052,000
Option H – Menge El 40 NAVD88	$6,564,000
Option I – Road/Menge El 20 ft NAVD88	$2,073,000
Option J – Road/Menge El 30 ft NAVD88	$6,016,000

3.4.4.16 References

US Army Corps of Engineers (USACE) 1987. Hydrologic Analysis of Interior Areas. Engineer Manual EM 1110-2-1413. Department of the Army, US Army Corps of Engineers, Washington, D.C. 15 January 1987.

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USACE 2006. Risk Analysis for Flood Damage Reduction Studies. Engineer Regulation ER 1105-2-101. Department of the Army, US Army Corps of Engineers, Washington, D.C. 3 January 2006.

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Environmental Science Services Administration. 1968. “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968.

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3.4.5 Back Bay of Biloxi Surge Barrier

3.4.5.1 General

In order to protect the properties surrounding Biloxi Bay and along the lower portions of the various rivers and streams flowing into the bay, a barrier would be required at some point to block storm waters during major storm events.

As outlined above, a search of other similar facilities constructed world wide revealed that the structure model best satisfying both the engineering and socio-ecological necessities of this site was that used for the Thames River Barrier in London, UK. The structure tentatively chosen for incorporation into this work was thus, patterned after the Thames River Barrier with certain minor modifications to adapt to the site and environment specific conditions enumerated previously.

3.4.5.1.1 Interior Drainage

In the event of an imminent hurricane, the gates across the Back Bay of Biloxi would be closed, and flow from the rivers feeding these bays, as well as local runoff would pond behind the gates. The tentative location of the barrier chosen for this study is shown Figure 3.4.5.1-1.

The Biloxi Bay watershed is an approximately 640 square mile watershed comprised of six subbasins that stretch across Harrison, Stone, and Jackson County, MS. There is one United States Geological Survey (USGS) discharge gage located in the watershed along the Biloxi River and one National Oceanic and Atmospheric Administration (NOAA) hourly precipitation gage located on the east side of the watershed. The discharge gage is USGS gage 2481000 at Wortham, MS and the precipitation gage is NOAA gage 107840 (Saucier Experimental Forest). Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering Biloxi Bay from the Biloxi Bay watershed for the 2-100 year rainfall events. The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The Biloxi Bay watershed is shown in Figure 3.4.5.1-2.

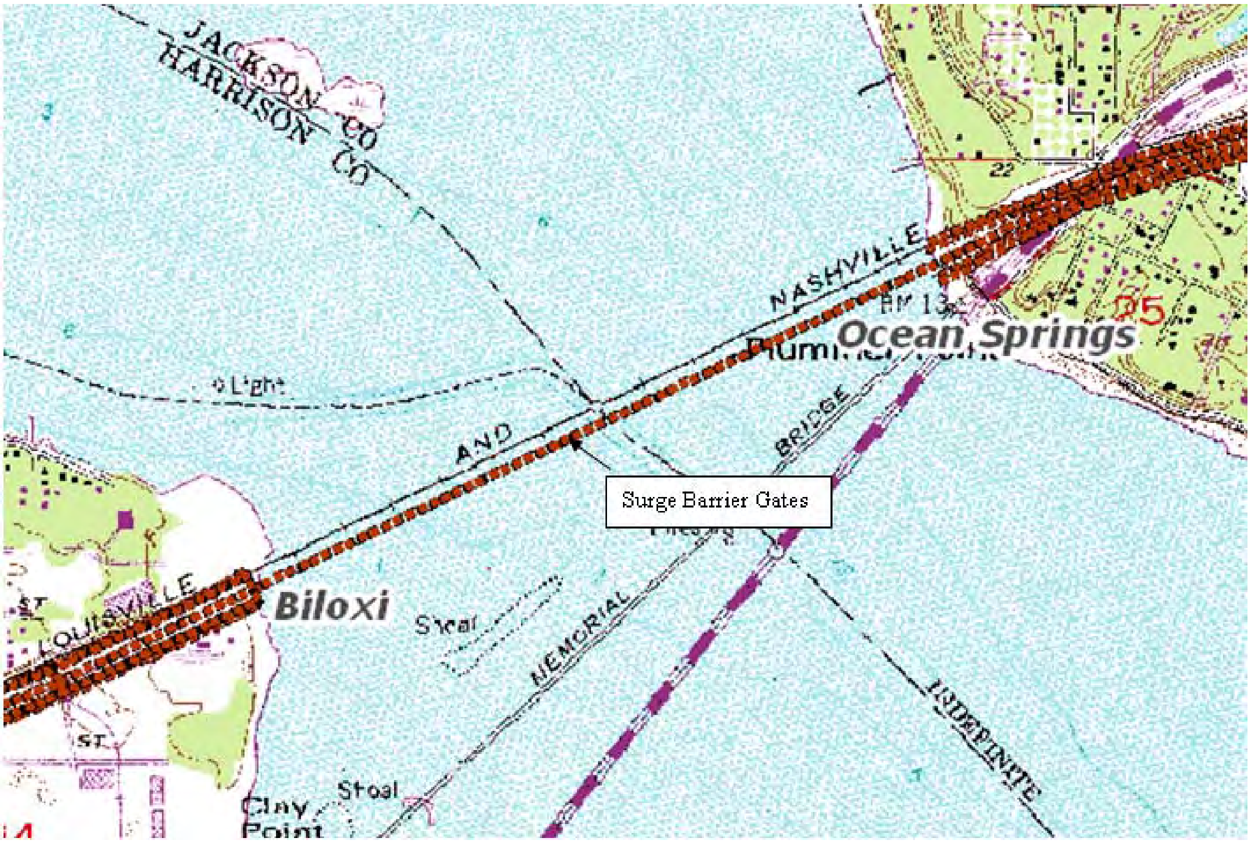

Figure 3.4.5.1-1. Biloxi Bay Surge Barrier Location

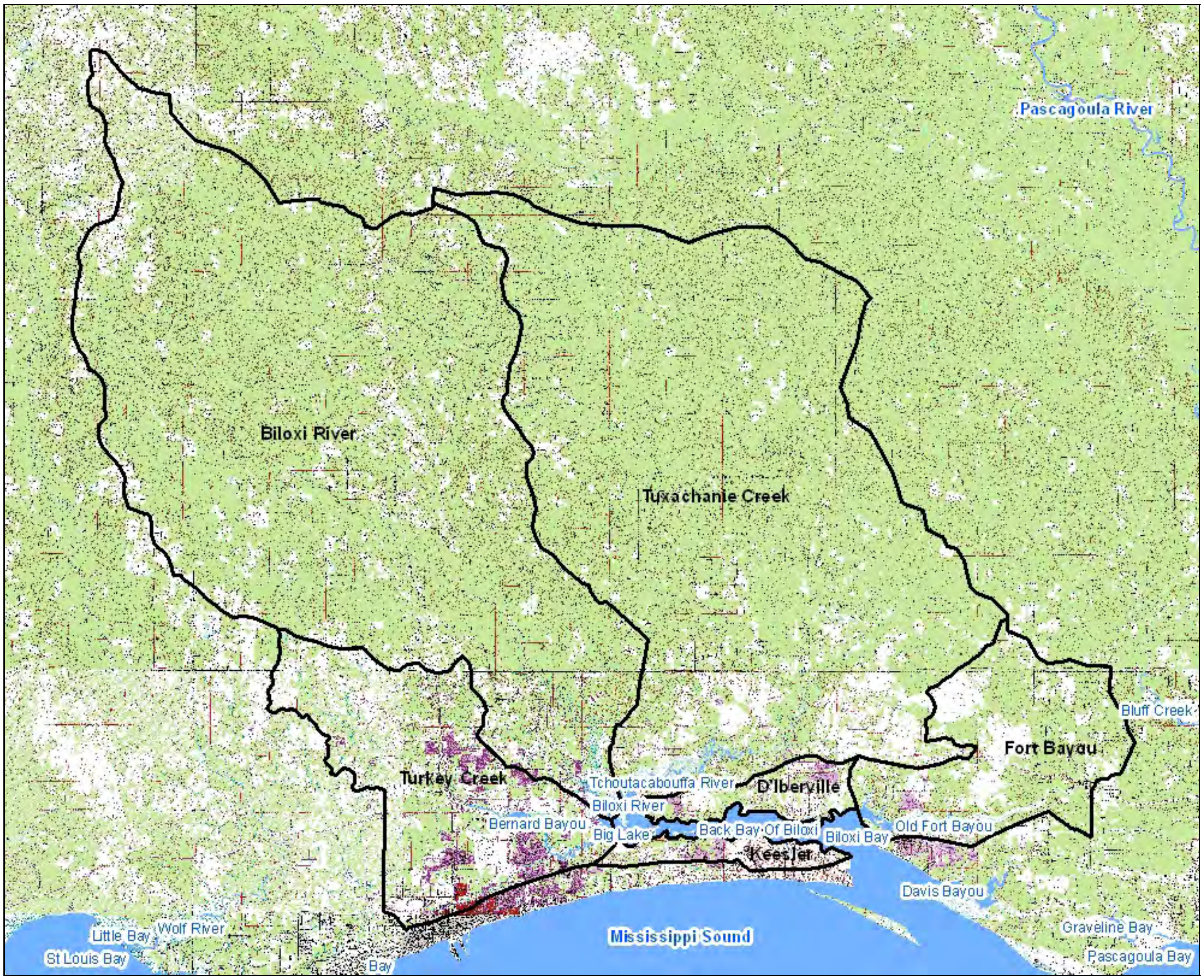

Figure 3.4.5.1-2. Biloxi Bay Watershed

The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gage 107840 and the 2-100 year 24-hour TP-40 rainfall events. The initial and constant loss rate and SCS curve number methods were used for the loss model while the Snyder's unit hydrograph (UH) and SCS UH methods were used for the direct runoff model. The model was calibrated to observed hourly discharge data for two events at USGS gage 2481000.

Calibration results agree reasonable well with observed data as shown in Figures 3.4.5.1-3 and 3.4.5.1-4.

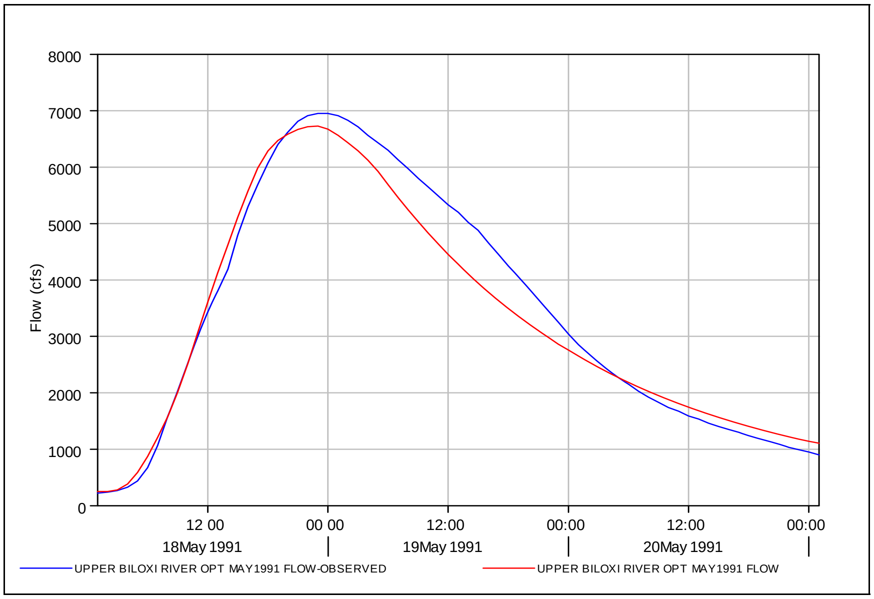

Figure 3.4.5.1-3. Biloxi Bay Watershed Calibration

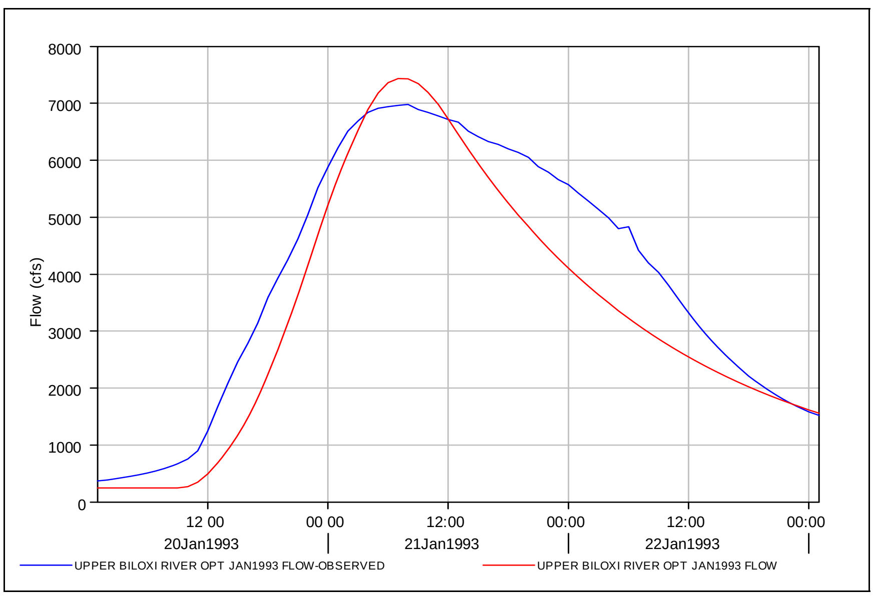

Figure 3.4.5.1-4. Biloxi Bay Watershed Calibration

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Areal Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V. Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District, U.S. Army Corps of Engineers.

The 24-hour period of highest inflow from the flow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

Based on this method of analysis, the resulting elevations for the various storms are shown in Figure 3.4.5.1-1, with the 10-yr elevation of 8.4 ft NAVD88 the design condition.

Table 3.4.5.1-1.
Biloxi Bay Ponding

Biloxi Bay 4 ft. Base Elevations	
Strom Event	**Bay Elevation (ft NAVD88)**
2-year	6.0
5-year	7.6
10-year	8.4
25-year	9.4
50-year	10.0
100-year	10.8

This ponded water area in Jackson County above the surge barrier gates at the 10-yr flood is at 8.4 ft NAVD88 and is approximated by the 8-ft ground contour line shown in Figure 3.4.5.1-5.

Figure 3.4.5.1-5. Biloxi Bay 10-yr Ponding to Elev 8.4 ft NAVD88

This ponded water area in Harrison County above the surge barrier gates at the 10-yr flood is at 8.4 ft NAVD88 and is approximated by the 8-ft ground contour line shown in Figure 3.4.5.1-6.

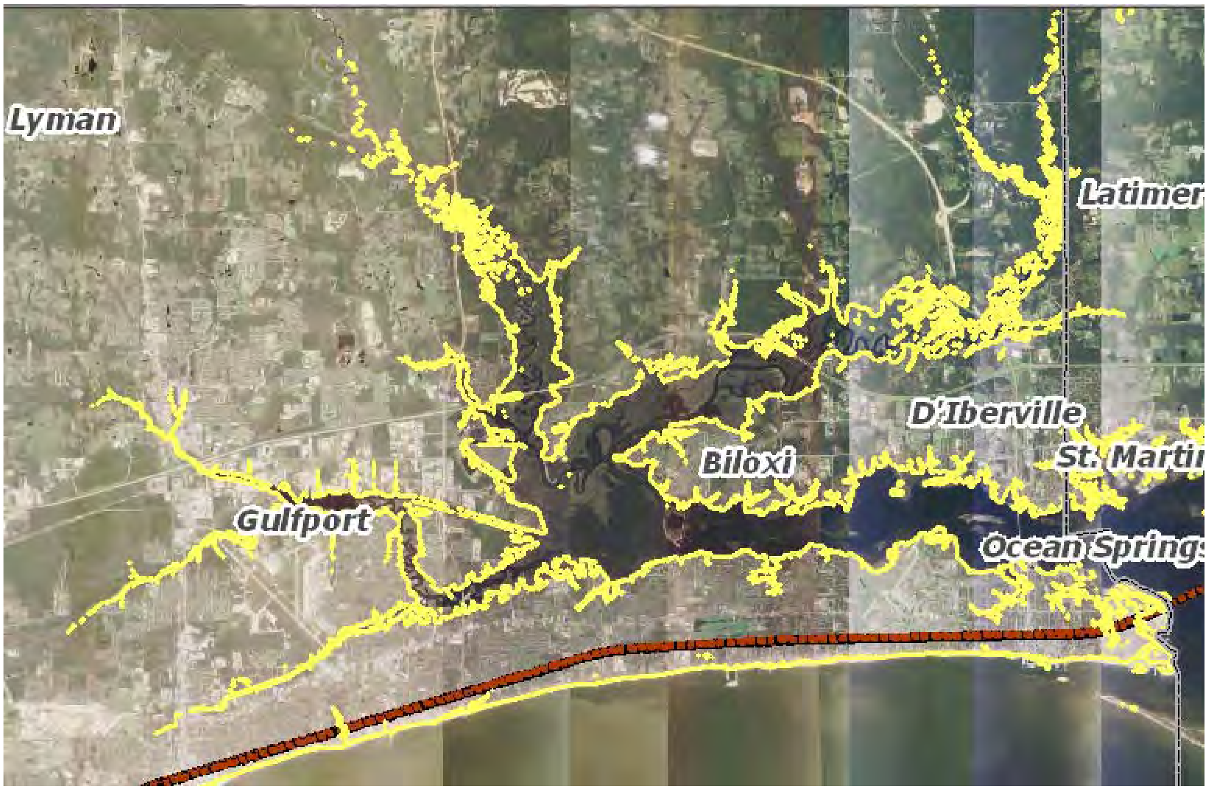

Figure 3.4.5.1-6. Biloxi Bay 10-yr Ponding to Elev 8.4 ft NAVD88

3.4.5.1.2 Geotechnical Data

The available mapping covering the bay bottom is very sketchy consisting mostly of quad maps. This data indicates that the existing bay bottom elevation along the study alignment would vary from a maximum of about (-)12 feet at the maintained channel (the nominal channel depth) to approximately (-)3 feet near, and for some distance out from, each bank. Information gathered from the Mississippi Department of Transportation subsequent to their emergency replacement of the U.S. Highway 90 Bridge indicates that the bay bottom materials are very loose and unstable to a significant depth below the bay bottom, indicating that a significant amount of undercutting would be required for any structure that might be installed, and that structures of the magnitude under consideration would require very deep pile foundations.

3.4.5.1.3 Structural, Mechanical and Electrical

See sections 3.4.5.1.3.1 through 3.4.5.1.3.3.

3.4.5.1.3.1 Structural

Structurally, the Barrier as configured for this study would consist of a series of 25 large stainless steel clad, structural steel framed gates called rising sector gates. Each of these would be supported on reinforced concrete piers resting on large continuous concrete sills with pile foundations. The tentative layout used to estimate the scope of the structure was configured having gates 132 feet long mounted on 28-foot wide piers. The number of gates was determined by the extent of water having depth sufficient to support their operation. To facilitate as nearly as possible the normal ebb and flow of tide waters through the barrier, the concrete connector wall and rock fill portions of the barrier either side of the gated structure would be fitted with a series of closely spaced low level gated culverts. The gate and pier heights were varied to accommodate the “level of protection” under consideration. The three elevations selected for this study were 20, 30, and 40 NAVD88. In each case the gate heights were set to match the protection level elevations with pier heights set approximately 3 feet higher to provide minor wave clearance for protection of operating equipment. Atop each pier an operating machinery block would be mounted to house the operating equipment. No lateral access over the tops of the piers was envisioned because of the long spans and the desire to keep the vista across the structure as clear as possible. Operating and utility access would be provided through two continuous tunnels passing through the sill section and the rock fill, to operating facilities located on each bank.

3.4.5.1.3.2 Mechanical

The mechanical equipment and appurtenances required for operation of these facilities would include very large steel gate linkages and hydraulic rams and pivot pins for operation of each gate. Each gate would rotate on large bearings and pivot hubs at each end of the gate. Various operating hydraulic and lubrication oil systems would also be required. Each gate would have an opening/closing time of approximately 15 minutes.

3.4.5.1.3.3 Electrical

Primary electrical power for operating the gates would be provided using dedicated, standard transformers with emergency back-up generators. The size of the generators would be greatly reduced by minimizing the wattage output through reduction of the demand on the facility. The demand would be minimized by phasing the operation of the gates to the greatest extent possible. For this study it was determined that this could possibly be done by operating a maximum of eight gates at a time, with the last eight gates being left open until the storm threat was definite and eminent. The operation would require that a maximum of four gates be started at one time, with the remaining four gates sequenced to start 1 minute later. It was determined that this would allow the entire closure and subsequent opening operation to be done over a period of 4 to 6 hours. The supplemental generation aspect was considered to be a vital component of the design because of the very high cost of commercial standby power and because commercial electric power would almost certainly be unavailable during and immediately following a storm event.

3.4.5.1.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.5.1.5 Construction Procedures and Water Control Plan

Following is a very tentative description of a sequence of construction by which the barrier structure and embankments might be built. There are admittedly myriad other means by which this could be accomplished as demonstrated by the construction methods used in construction of the Thames River Barrier and various structures in The Netherlands and elsewhere, any one of which might result in more economical and expeditious construction of the barrier. However, at this juncture, in the interest of clarity and brevity, it was considered expedient to describe this work using customary construction techniques common to most of our large civil works projects constructed to date.

3.4.5.1.5.1 Construction Procedure

As configured for this study, the physical construction of the barrier would begin with installation of the first of what would likely be a two stage cellular cofferdam. The arrangement assumed for this study consisted of a series of circular sheet pile cells and connecting arcs measuring approximately 60 feet in diameter and extending 100 feet from the top of the cell to the pile tip elevation. These cells would encompass either the east side or west side transition monoliths and the portion of the gated structure extending to the center of the boat channel. The second phase cofferdam would encompass the remainder of the gated structure and the remaining non-overflow concrete section. It was assumed that for structures designed to provide the highest protection level (Elevation 40 NAVD88) the top of cells could be placed at elevation 35 with reasonable degree of safety. This would provide cell embedment of approximately 30 feet below the lowest structure foundation elevation. This configuration was, naturally, modified to fit the lower levels of protection but in each case the configuration was made to provide the same relative of protection during construction. With the cofferdam in place the interior would be dewatered using hydraulic pumps, and excavation for the concrete structures would begin. Once the excavation in a given area is brought to the required grade work would continue in this area with the installation of foundation piles. Prior to completion of the first phase of the concrete work, installation would begin on the next phase of the cofferdam.

Once the first phase of the concrete structure is completed and the first phase cofferdam removed, installation of the gates and operating machinery would begin. Fabrication of the gates would have been done on land in an outfitting yard and the gates transported by water to the proper installation site. Note that this would likely require dredging of a temporary construction channel parallel to the barrier for a portion of its length.

Construction of the rock fill embankments would require surcharging and pre-consolidation of the bay bottom materials. (See section 3.4.3.1.2 above for discussion of the Geotechnical aspects of this site.)

3.4.5.1.5.2 Water Control Plan

As this work progresses the flow into and out of Biloxi Bay would be somewhat restricted for practically the entire construction time. This restriction could be minimized by removal of the cofferdams immediately upon completion of the concrete piers to some point above the normal high tide level thus allowing flow over the completed sill sections as construction continues on the piers and as the gates are being installed. It is estimated the maximum flow restriction at any time would be approximately 50% of the inlet width and that this restriction could endure for as much as three to five years using the methods and approximate sequence of construction indicated above.

3.4.5.1.6 Physical Security

During the construction of the project the contractor would be responsible for maintaining security of all his work sites. This would be done in accordance with guidance noted under Section 3.4.1.7 General, above, in addition to the normal site security requirements.

Upon completion of the project the facilities security responsibilities would pass to the U.S. Army Corps of Engineers and the state, county and municipal law enforcement entities, all of whom would coordinate a program of oversight under which the facilities would be operated and maintained and under which specific security responsibilities would be defined and allocated. These agreements would also be required to reflect the provisions of the guidance noted in Section 3.4.1.7 General, above, in addition to normal security criteria.

3.4.5.1.7 Operations and Maintenance

In order to assure proper functioning of the facilities once they are placed in service a program of Operations and Maintenance would be developed by the U.S. Army Corps of Engineers, in conjunction and cooperation with the affected state and local entities. This O & M Plan would address specific responsibilities as to daily operation of the facilities, the periodic testing and maintenance of the operating machinery, maintenance of specified stocks of replacement parts, security of the facilities, and maintenance of any buildings and grounds associated with the operation and maintenance of the facilities. As presently envisioned, this O & M responsibility would remain under control of the U.S. Army Corps of Engineers and would be administered under its Operations mission.

3.4.5.1.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.5.8 Cost Summary. Construction costs for the various options are included in Table 3.4.5.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.5.8-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates exclude project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.5.1.9 Schedule and Design for Construction

The scheduling for events following this conceptual study would of necessity include extensive further study to ascertain in greater detail the specific requirements of the project and the most feasible means by which to fulfill these requirements. The Sequence of events would include but not be limited to the following:

a. The alignment and extent of the proposed barrier should be subjected to detailed study to determine the most feasible routing. This study should address, among other factors, the exact location of utilities features crossing the bay inlet, the present and projected future needs of the boat channel passing through the barrier, and how best to minimize the effects that the barrier could have on the existing marine environment.

b. Detailed deep geotechnical investigation should be made to determine as accurately as possible the engineering capabilities of the soils making up the bay bottom along the alignment (or alignments) under consideration.

c. A more thorough and painstaking investigation of various types of gate structures should be undertaken to confirm the choice of the rising sector gate for this application, or to replace this type gate with another perhaps more appropriate to the circumstances.

d. Once this search and these investigations and analyses have been completed a thorough design of the structures to be included in the final facility would be undertaken addressing the full range of hydraulic events that the structure might see, and making certain that all pertinent design considerations are accounted for.

e. A thorough analysis of the power required to operate the gates in a timely manner in time of storm must be made and the very best, most dependable means of providing this power determined.

f. The link between the operation of the gates and the best available storm forecasting system(s) would be designed and its operating features and equipment detailed.

3.4.5.2 Location

The alignment suggested herein for the barrier structure would run parallel with and south of the Railroad Bridge crossing Biloxi Bay. This would approximate the shortest route across the inlet leading from the Mississippi Sound into the bay. As the preliminary layout of the barrier was developed it became apparent that, because of the excavation required, a significant amount of separation would be required between the railroad bridge and the ultimate location of the structures included in the barrier. For this study the centerline of the barrier was positioned approximately 260 feet from the center of the railroad bridge. This was left unaltered for all protection levels. The entire barrier would be approximately 6,100 feet in length from water's edge to water's edge, and would consist of rock fill levees extending from the overland levee at each bank for some distance into the bay and enveloping the mass concrete non-overflow wall sections leading to each end of the gated structure.

3.4.5.3 Existing Conditions

The points at which the barrier would come ashore in Jackson County on the east and Harrison County on the west, are in urban areas with extensive residential and commercial development. Several structures would need to be relocated and it is uncertain the extent to which existing utilities might have to be relocated to clear the way for this facility.

3.4.5.4 Coastal and Hydraulic Data

Historic coastal data is shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as the 6-ft(blue), 12-ft(green), 16-ft(brown), and 20-ft(pink) ground contour lines are shown in Figure 3.4.5.4-1. The data indicates the Katrina high water was as high as 21 ft NAVD88 at the mouth of the bay.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is shown elsewhere in this report. Points near the mouth of the bay at which data from hydrodynamic modeling was saved are shown in Figure 3.4.5.4-2.

Existing Condition Stage –Frequency data for Save Point 9, near the mouth of the bay, is shown in Figure 3.4.5.4-3. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

Figure 3.4.5.4-1. Ground Contours and Katrina High Water

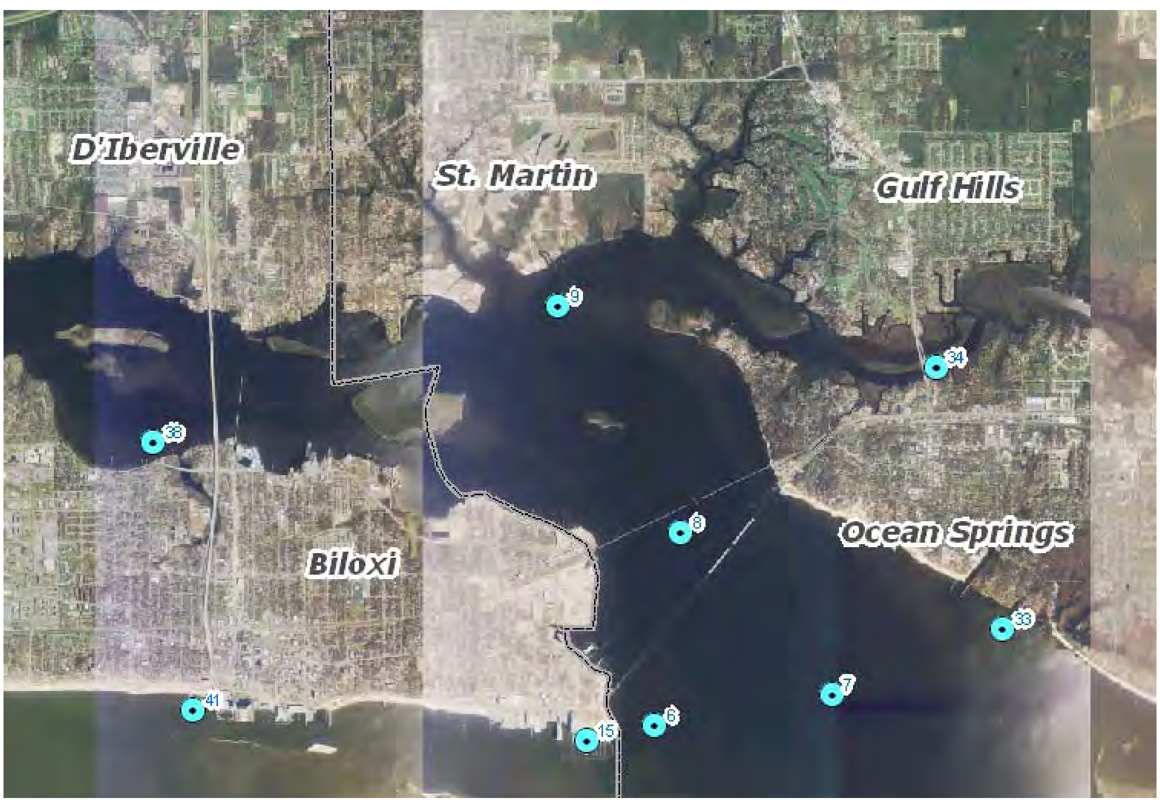

Figure 3.4.5.4-2. Hydrodynamic Modeling Save Points near Biloxi Bay

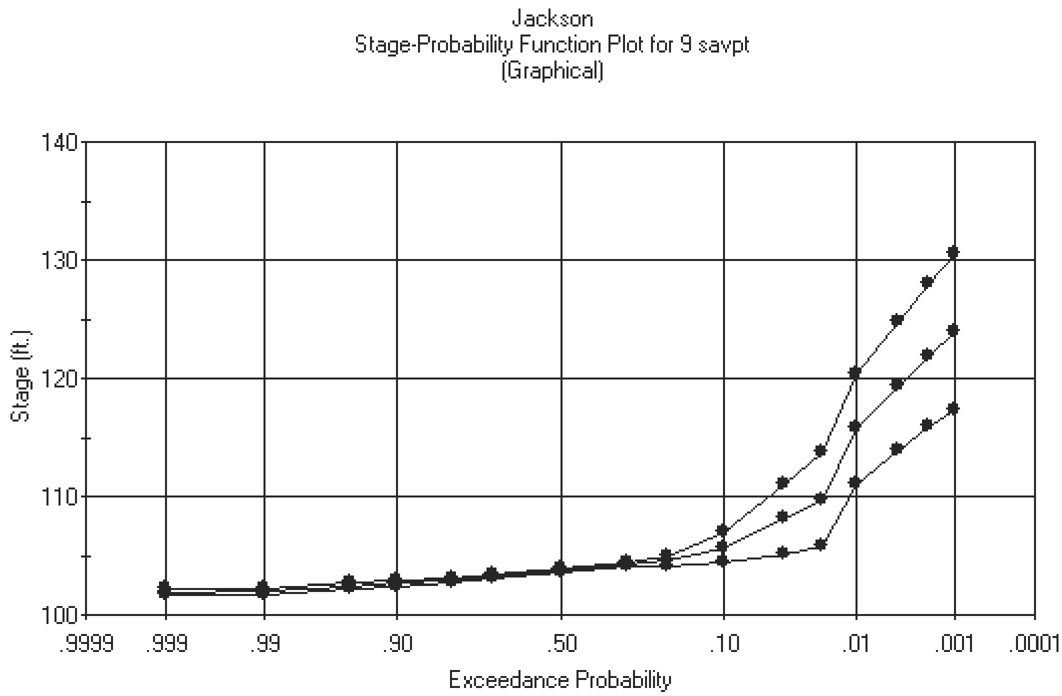

Figure 3.4.5.4-3. Existing Conditions at Save Point 8, near the Mouth of Biloxi Bay

3.4.5.5 Option A – Elevation 20.0

3.4.5.5.1 Structural

In order to reasonably accurately approximate the scope of the structures required to form a moveable barrier to elevation 20 a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. A system of foundation piles was then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation "Zero" on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation "zero" at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 20 on the flooded side of the structure and to elevation "zero" on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 20 resulted in gross quantities of basic construction materials as indicated in Table 3.4.5.5-1.

Table 3.4.5.5-1.
Gross Quantities for Biloxi Bay Surge Barrier Elevation 20.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	23,294	Tons
Foundation Piling	50,540	Each
Concrete	493,700	Cubic Yards
Reinforcement	1,210	Tons
Rising Sector Gates (25 Each)	19,750	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.5.6 Option B – Elevation 30.0

3.4.5.6.1 Structural

In order to reasonably accurately approximate the scope of the structures required to form a moveable barrier to elevation 30, a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. The foundation piles were then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation "Zero" on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation "zero" at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 30 on the flooded side of the structure and to elevation "zero" on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 30 resulted in gross quantities of basic construction materials as indicated in Table 3.4.5.6-1 below.

Table 3.4.5.6-1.
Gross Quantities for Biloxi Bay Surge Barrier Elevation 30.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	31,837	Tons
Foundation Piling	14,538	Each
Concrete	552,800	Cubic Yards
Reinforcement	1,083	Tons
Rising Sector Gates (25 Each)	24,260	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.5.7 Option C – Elevation 40.0

3.4.5.7.1 Structural

In order to reasonably accurately approximate the scope of The structures required to form a moveable barrier to elevation 40, a very preliminary rising sector gate design was made for the gate and its operating disks, and the piers and foundations were approximated on a proportional basis. The foundation piles were then estimated from a stability analysis made for the most stringent hydraulic situation, water with wave impact to the top of the gates on the flood side and at elevation "Zero" on the protected side of the gate. Uplift for the situation described was assumed to vary from full static water head at the flood side edge of the sill to static water pressure equivalent to the embedment of the sill below elevation "zero" at the protected side edge of the sill. Static lateral water forces were derived for static water pressure to elevation 40 on the flooded side of the structure and to elevation "zero" on the protected side. Wave impact data from model testing was not yet available when these analyses were made. Therefore an approximation of the wave impact loading was made by applying 25% of the flooded side static head pressure at the top of the gate and allowing this to taper to zero at the base of the monolith. The force and moment resulting from this inverted triangular load was then added to that derived for the static head situation.

The preliminary design for a gated structure providing protection up to elevation 40 resulted in gross quantities of basic construction materials as indicated in Table 3.4.5.7-1 below.

Table 3.4.5.7-1.
Gross Quantities for Biloxi Bay Surge Barrier Elevation 40.0 NAVD88

Item	Quantity	Units
Cofferdam Piling	31,837	Tons
Foundation Piling	20,540	Each
Concrete	561,300	Cubic Yards
Reinforcement	1,061	Tons
Rising Sector Gates (25 Each)	40,291	Tons
Gate Operating Machinery (Steel, 25 sets)	10,100	Tons

Note: Quantities taken from preliminary stability and other design computations.

3.4.5.8 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown below. Estimates are comparative-Level "Parametric Type" and are based on Historical Data, Recent Pricing, and Estimator's Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.4.5.8-1.
Back Bay of Biloxi Surge Barrier Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$989,800,000
Option B – Elevation 30 ft NAVD88	$1,267,100,000
Option C – Elevation 40 ft NAVD88	$1,810,700,000

Table 3.4.5.8-2.
Back Bay of Biloxi Surge Barrier O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$13,770,000
Option B – Elevation 30 ft NAVD88	$17,646,000
Option C – Elevation 40 ft NAVD88	$25,243,000

3.4.5.9 References

See 3.4.3 General discussion above for references.

3.4.6 Jackson County Inland Barrier

3.4.6.1 General

Several high density residential and business areas are located in Jackson County. These are subject to damage from storm surges associated with hurricanes. Earthen levees were evaluated for protection of these areas. The levees were evaluated at elevations 20 ft NAVD88 and 30 ft NAVD88 and 40 ft NAVD88. The top width was assumed 15 ft with sideslopes of 1 vertical to 3 horizontal. Each of the levees is presented separately in this report. Storm surge gates across Biloxi Bay are also included to prevent flooding from hurricanes. Additional options not evaluated in detail are described elsewhere in this report.

Evaluation of this option was done by comparing benefits computed by Hydrologic Engineering Center’s (HEC) Flood Damage Analysis (FDA) computer application HEC-FDA and costs computed. HEC-FDA modeling was done comparing the study reaches using variations in expected sea-level rise and development. Details regarding the methodology are presented elsewhere in this report.

3.4.6.2 Location

The location of the levee in Jackson County is shown in Figures 3.4.6-1 through 3.4.6-4 parallel to the CSX railroad, Hwy 57 and Hwy 90.

Figure 3.4.6-1. Vicinity Map Jackson County, MS

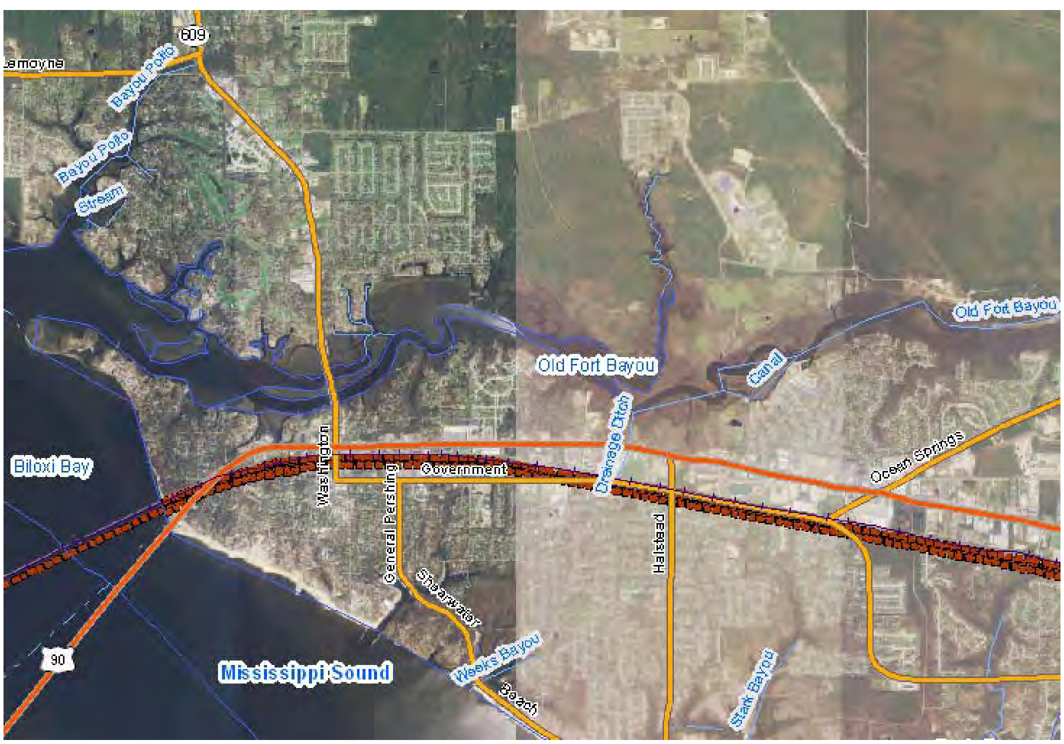

Figure 3.4.6-2. Jackson County Inland Barrier

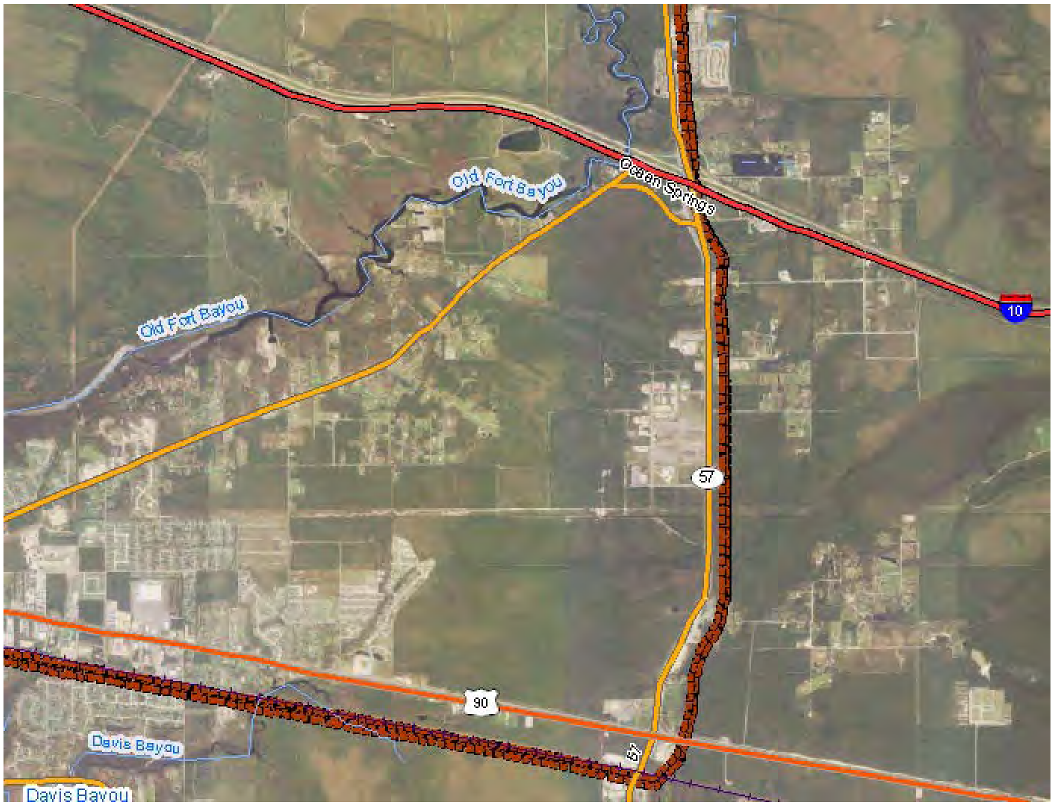

Figure 3.4.6-3. Jackson County Inland Barrier

Figure 3.4.6-4. Jackson County Inland Barrier

3.4.6.3 Existing Conditions

Jackson County is located on the east side of the Mississippi at the Mississippi Sound coast. The main residential and business area is at Ocean Springs, which is mostly south of the levee. Ground elevations over the areas behind the levee vary between Elevations 10-20 ft NAVD88 at low areas to as high as 50 ft NAVD88. The area is drained by Old Fort Bayou. The 4-ft (blue), 10-ft (green), 20-ft (pink), 30-ft (dark Blue), 40-ft (purple), and 50-ft (gray) ground contour lines are shown in Figure 3.4.6-5.

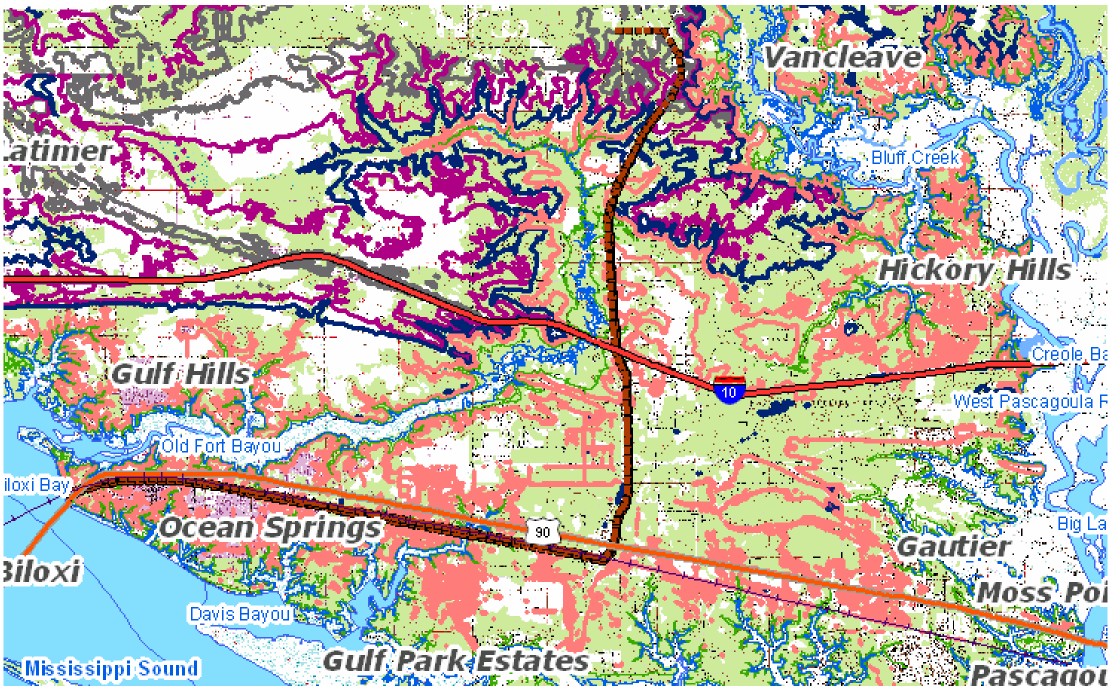

Figure 3.4.6-5. Existing Conditions Jackson County, MS

Drainage from ordinary rainfall is hindered on occasions when either of the rivers or the gulf is high, but impacts from hurricanes are devastating.

Recent damage from Hurricane Katrina in August, 2005 near the mouth of the Old Fort Bayou area are shown in Figures 3.4.6-6 and 3.4.6-7.

3.4.6.4 Coastal and Hydraulic Data

Historic coastal data is shown in Paragraph 1.4, elsewhere in this report. High water marks taken by FEMA after Hurricane Katrina in 2005 as well as 4-ft(blue), 10-ft(green),20-ft(pink), 30-ft(dark Blue), 40-ft(purple), and 50-ft(gray) ground contour lines are shown in Figure 3.4.6-8 below. The data indicates the Katrina high water was as high as 21-22 ft NAVD88 in the Old Fort Bayou area north of Ocean Springs.

Stage-Frequency data for a suite of severe storms using Joint Probability Method (JPM) and hydrodynamic modeling were developed by the Engineer Research and Development Center (ERDC) for 80 locations along the study area. These data were combined with historical gage frequencies for smaller storms to establish stage-frequency curves at 54 economic reaches in the study area which were entered into Hydrologic Engineering Center's (HEC) Flood Damage Analysis (HEC-FDA) application to evaluate benefits. An expanded description of the procedure is presented elsewhere in this report. Points near Gautier at which data from hydrodynamic modeling was saved are shown in Figure 3.4.6-9.

Source: http://ngs.woc.noaa.gov/storms/katrina/24806787.jpg

Figure 3.4.6-6. Hurricane Katrina Damage Near mouth of Old Fort Bayou, MS

Source: http://www.flickr.com/photos/cbsnaps/53488199/, cbatesteach

Figure 3.4.6-7. Hurricane Katrina Damage in St Martin (nr Ocean Springs), MS

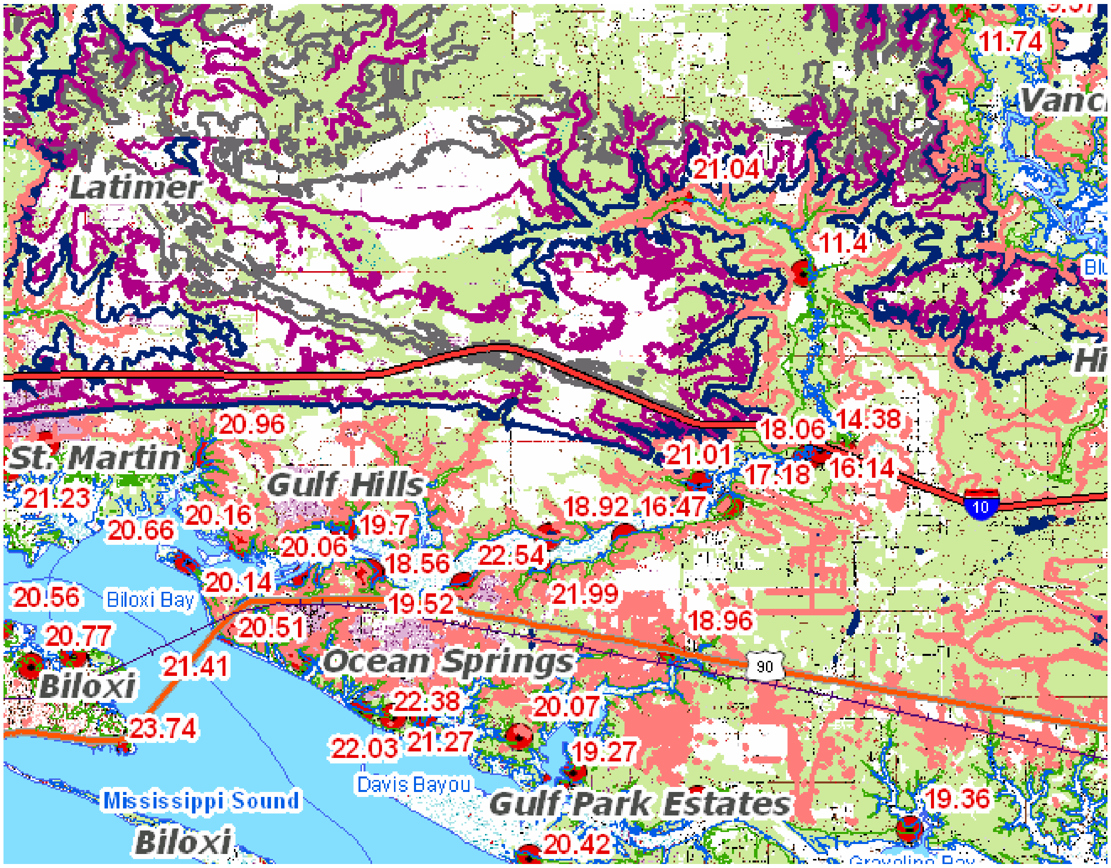

Figure 3.4.6-8. Ground Contours and Katrina High Water Elevations

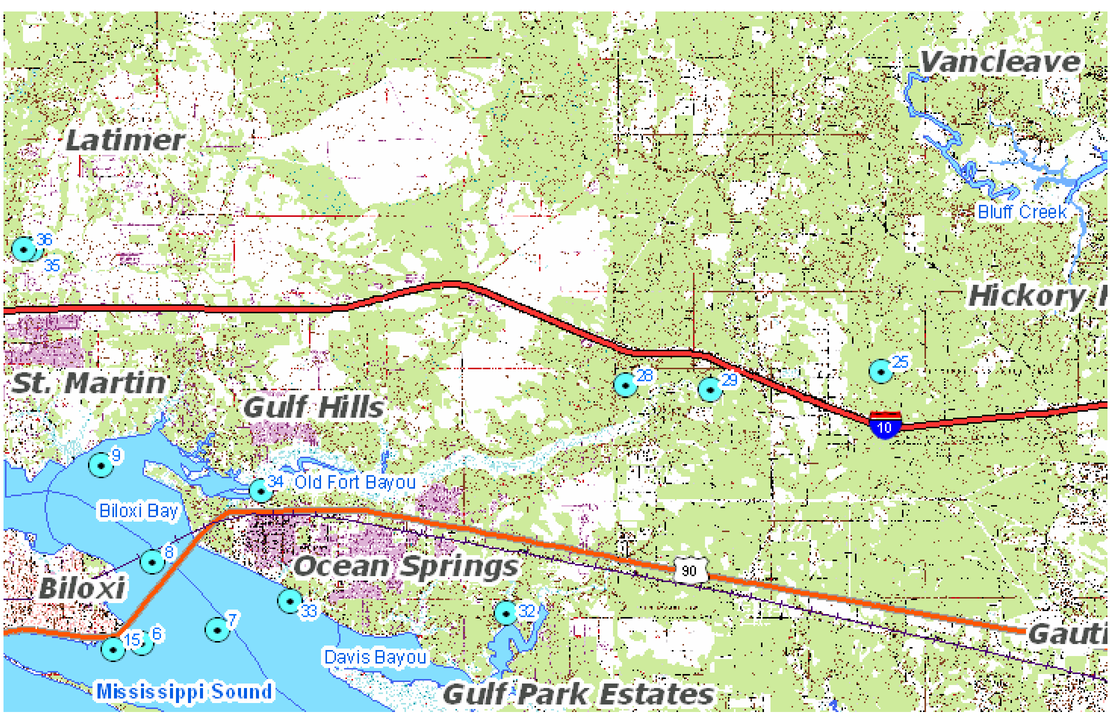

Figure 3.4.6-9. Hydrodynamic Modeling Save Points near Gautier, MS

Existing Condition Stage –Frequency data for Save Point 33, near the Ocean Springs, is shown in Figure 3.4.6-10 as an example. The 95% confidence limits, approximately equally to plus and minus two standard deviations, are shown bounding the median curve. The elevations are presented at 100 ft higher than actual to facilitate HEC-FDA computations.

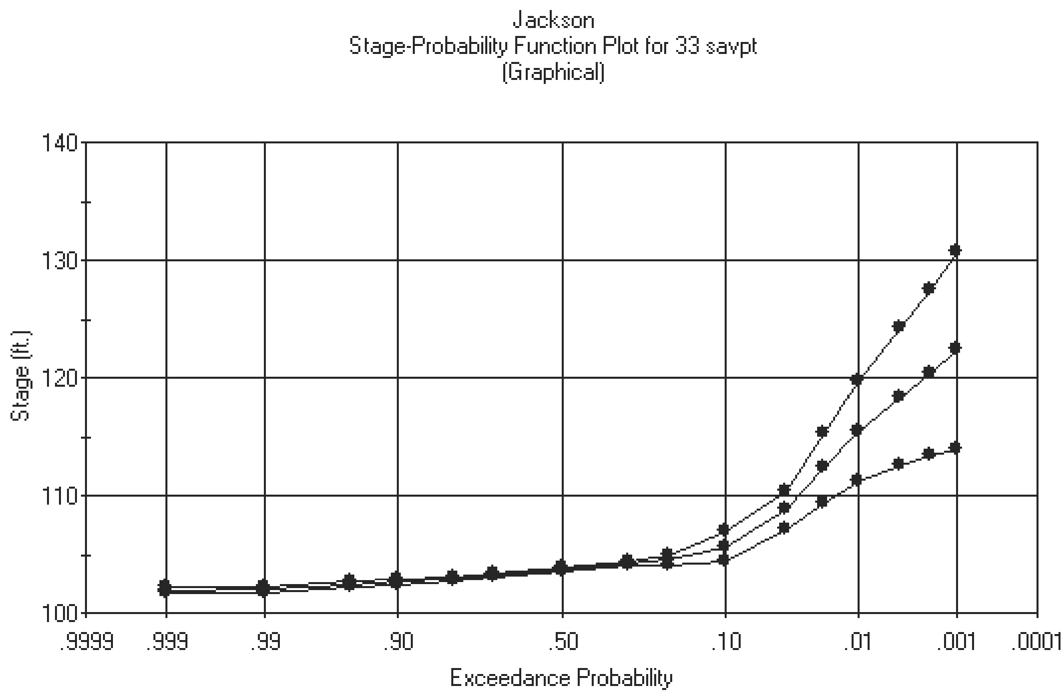

Figure 3.4.6-10. Existing Conditions at Save Point 33, near Ocean Springs, MS

3.4.6.5 Option A – Elevation 20 ft NAVD88

This option consists of an earthen dike around the areas north of Hwy 90 as shown on Figure 3.4.6-11, along with the internal sub-basins and levee culvert/pump locations. The levee would have a top width of 15 ft and slopes of 1 vertical to 3 horizontal. The levee is located mostly along high ground so ponding at the levee would be minimal. The levee surfaces will be armored with a layer of gabions to prevent scour during overtopping. Ponding will occur on the outside of the levee which would require ditching to other drainage basins. The ditch locations are shown in Figure 3.4.6-11 in dark blue.

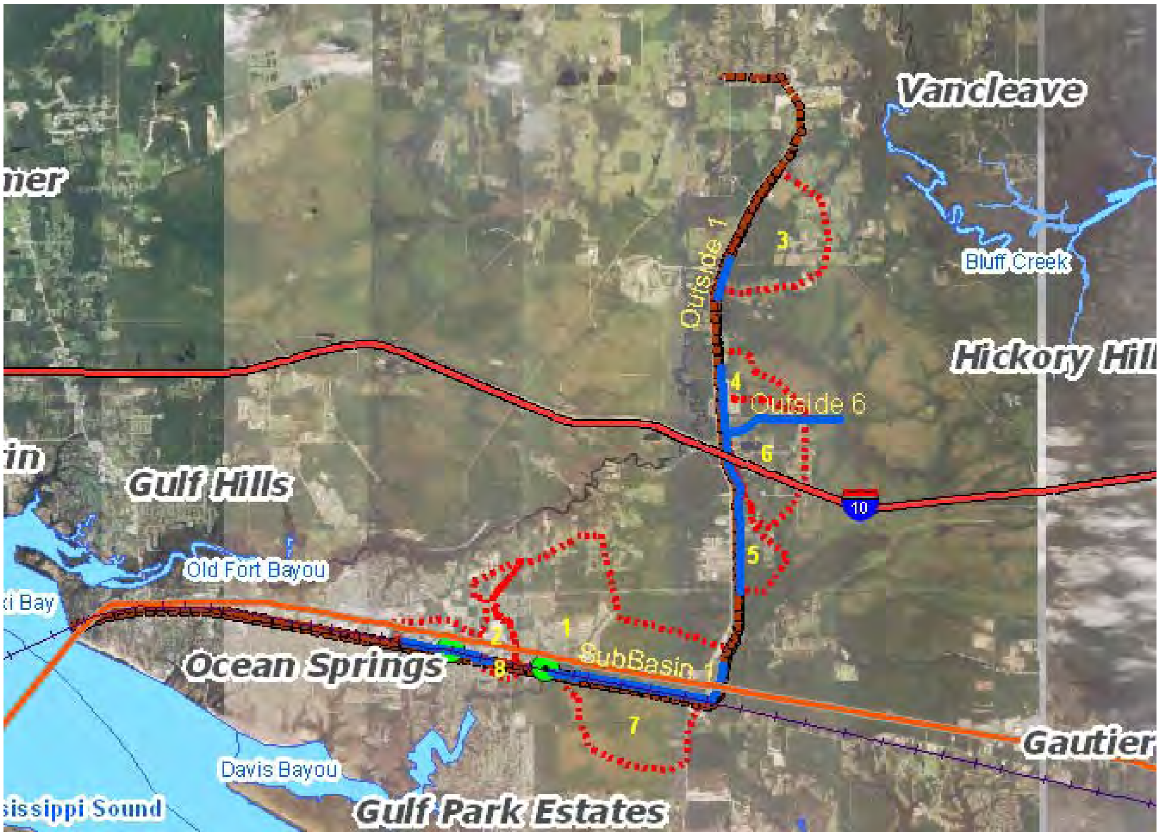

Figure 3.4.6-11. Pump/Culvert/Sub-basin Site Locations

Damage and failure by overtopping of levees could be caused by storms surges greater than the levee crest as depicted in Figure 3.4.6-12.

Source: Wave Overtopping Flow on Seadikes, Experimental and Theoretical Investigations, Holger Schüttrumpf, (Photo: Leichtweiss-Institute) http://kfki.baw.de/fileadmin/projects/E_35_134_Lit.pdf

Figure 3.4.6-12. North Sea, Germany, March 1976

Overtopping failures are caused by the high velocity of flow on the back side of the levee. Although significant wave attack on the seaward side of some of the New Orleans levees occurred during Hurricane Katrina, the duration of the wave attack was for such a short time that major damage did not occur from wave action. The erosion shown in Figure 3.4.6-13 was caused by approximately 1-2 ft of overtopping crest depth.

Revetment would be included in the levee design to prevent overtopping failure.

Source: ERDC, Steven Hughes

Figure 3.4.6-13. Crown Scour from Hurricane Katrina at Mississippi River Gulf Outlet (MRGO) Levee in St. Bernard Parish, New Orleans, LA

The levee would be protected by gabions on filter cloth as shown in Figure 3.4.6-14, extending across a drainage ditch which carries water to nearby culverts and which would also serve to dissipate some of the supercritical flow energy during overtopping conditions.

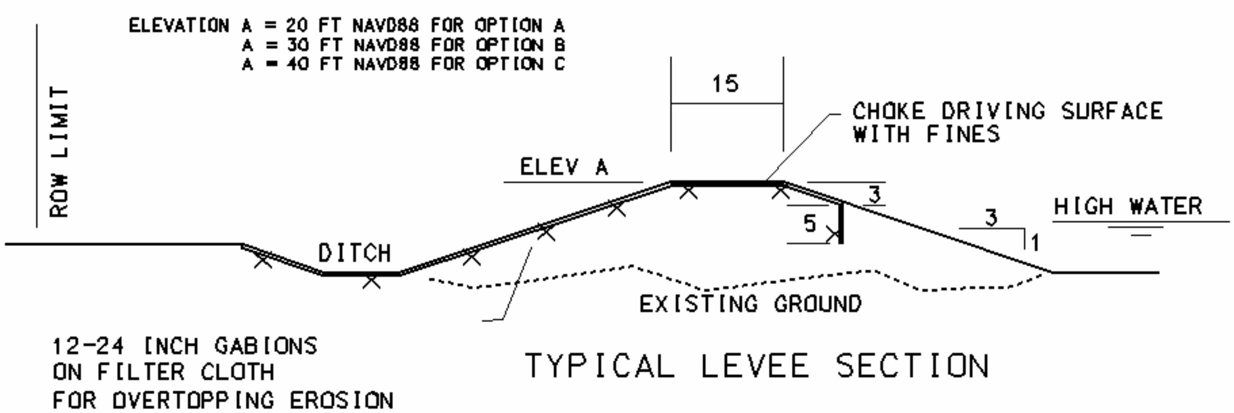

Figure 3.4.6-14. Typical Section at Inland Barrier

3.4.6.5.1 Interior Drainage

For smaller drainage areas, drainage on the interior of the inland barrier would be collected at the levee and channeled to culverts placed in the levee at the locations shown in Figure 3.4.6-11. The culverts would have tidal gates on the seaward ends to prevent backflow when the water in

Mississippi Sound is high. An additional closure gate would also be provided at the upstream end at every culvert in the levee for manual control in the event the tidal gate malfunctions. A typical section is shown in Figure 3.4.6-15.

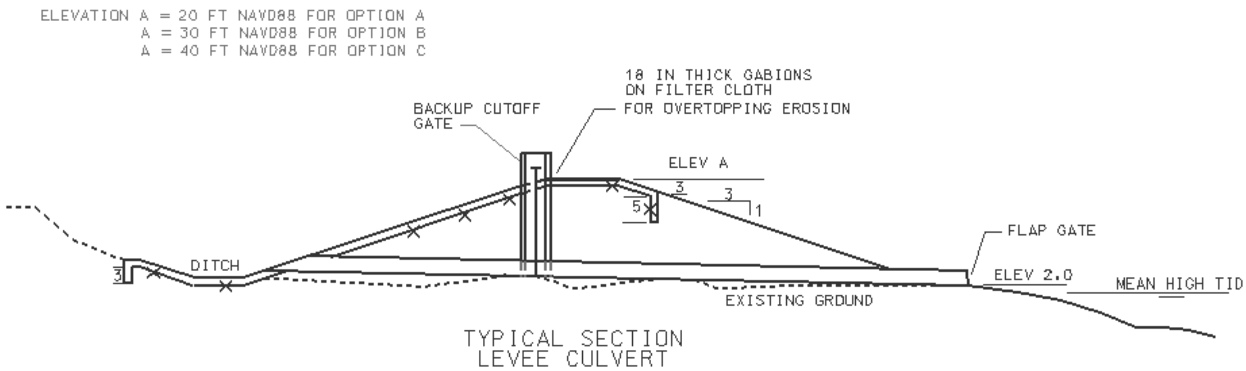

Figure 3.4.6-15. Typical Section at Culvert

In addition, pumps would be constructed near the outflow points to remove water from the interior during storm events occurring when the culverts were closed because of high water in the sound.

Flow within the levee interior was determined by subdividing the interior of the inland barrier into major sub-basins as shown in Figure 3.4.6-11 and computing flow for each sub-basin by USGS computer application WinTR55. The method incorporates soil type and land use to determine a run-off curve number.

Peak flows for the 1-yr to 100-yr storms were computed. Levee culverts were then sized to evacuate the peak flow from a 25-year rain in accordance with practice for new construction in the area using Bentley CulvertMaster application. For the culvert design, headwater elevations at the culverts were maintained at an elevation no greater than 5 ft NAVD88 with a tailwater elevation of 2.0 ft NAVD88 assumed. Drainage ditches along the toe of the levee will be required to assure that smaller basins can be drained to a culvert/pump site. These ditches were sized using a normal depth flow computation. Curve numbers, pump, and culvert capacity tables are not included in the report beyond that necessary to obtain a cost estimate. The data is considered beyond the level of detail required for this report.

During periods of high water in Mississippi Sound, pumps would be required to evacuate rainfall. Pump sizes were determined for the peak flow resulting from a 10-yr rainfall. This decision was based on an evaluation of rainfall observed during hurricane and tropical storm events as presented in two sources. The first is “Frequency and Aerial Distributions of Tropical Storm Rainfall in the US Coastal Region on the Gulf of Mexico” US Dept of Commerce, Environmental Science Services Administration, ESSA Technical Report WB-7, Hugo V Goodyear, Office Hydrology, July 1968. The second is “National Hurricane Research Project Report No. 3, Rainfall Associated with Hurricanes (And Other Tropical Disturbances)”, R.W. Schoner and S. Molansky, 1956, Weather Bureau and Corps of Engineers. This decision was also based on coordination with the New Orleans District.

During some hurricane events, when the gates are shut, and rainfall exceeds the average 10-yr intensity over the basin, some ponding from rainfall will occur. Detailed modeling of all the interior sub-basins for all the areas was not possible for this report; therefore the exact extent of the ponding for extreme events is not precisely defined. However, in some of the areas, existing storage could be adequate to pond water without causing damage, even without pumps. In other areas that do have pumps, some rise in interior water during interior events greater than the 10-yr rain could occur, but may not cause damage. Designing the pumps for the peak 10-yr flow provides a significant pumping capacity. Further studies will detail the requirement for the appropriate ponding areas, pump sizes, or buyouts in the affected areas.

During non-hurricane periods of low water in the sound, when rainfall greater than the 25-yr event occurs, the pumps could also be used to augment the flow capacity of the levee culverts.

In addition to the local drainage outlets at the levee described above, in the event of an imminent hurricane, barrier gates across the Back Bay of Biloxi would be closed, and flow from the Biloxi and Tchoutacabouffa Rivers, as well as local runoff would pond behind the gates. The location of the barrier is shown in Figure 3.4.6-16.

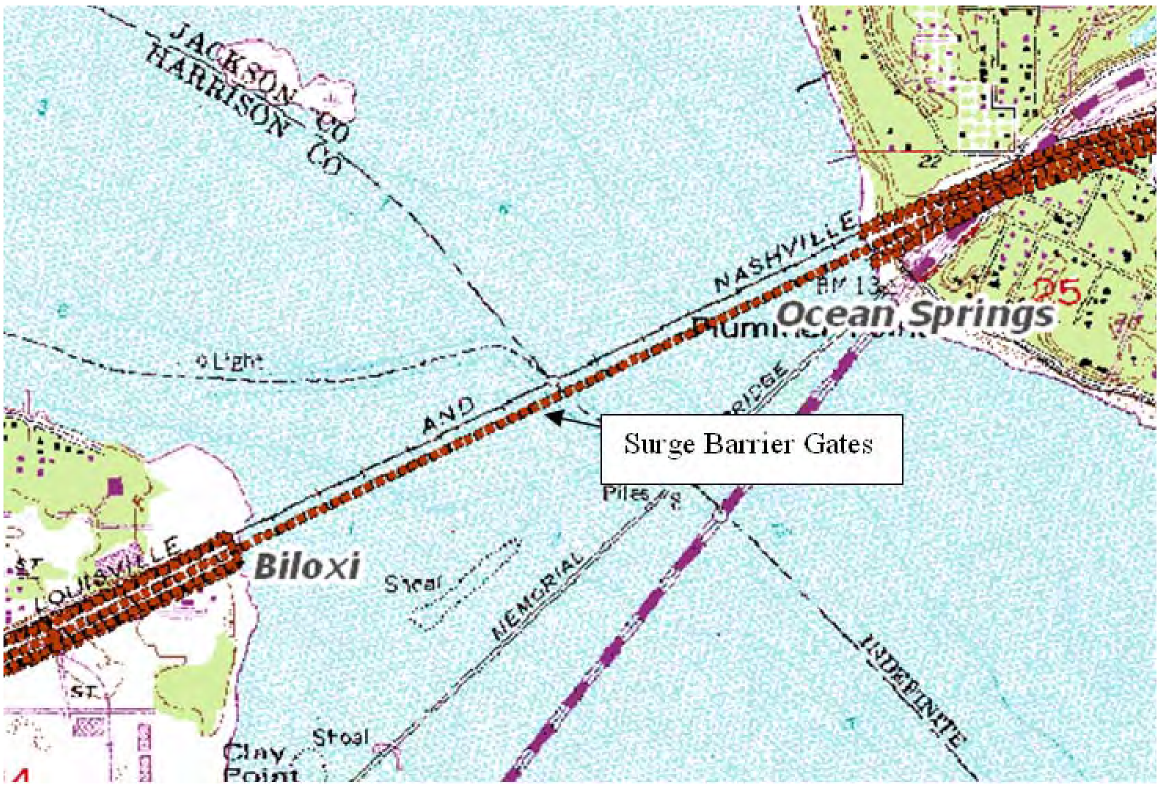

Figure 3.4.6-16. Biloxi Bay Surge Barrier Location

The gates would be similar to the gates across the Thames River in London, England, shown in Figure 3.4.6-17.

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS) was used to model the Biloxi Bay watershed in order to predict the maximum water elevation behind the gates in the bay under several different scenarios.

Figure 3.4.6-17. Thames River Barrier Gates

The Biloxi Bay watershed is an approximately 640 square mile watershed comprised of six subbasins that stretch across Harrison, Stone, and Jackson County, MS. There is one United States Geological Survey (USGS) discharge gage located in the watershed along the Biloxi River and one National Oceanic and Atmospheric Administration (NOAA) hourly precipitation gage located on the east side of the watershed. The discharge gage is USGS gage 2481000 at Wortham, MS and the precipitation gage is NOAA gage 107840 (Saucier Experimental Forest). Data from these gages, along with soils data from the National Cooperative Soil Survey and Technical Paper 40 (TP-40) synthetic rainfall events were used to determine the peak discharge and total run-off volume entering Biloxi Bay from the Biloxi Bay watershed for the 2-100 year rainfall events. The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used for the modeling effort. The Biloxi Bay watershed is shown in Figure 3.4.6-18.

The components of the model include the precipitation specification, the loss model, the direct runoff model, and observed discharge data. Precipitation data used in the modeling process included hourly precipitation from NOAA gage 107840 and the 2-100 year TP-40 rainfall events. The initial and constant loss rate and SCS curve number methods were used for the loss model while the Snyder's unit hydrograph (UH) and SCS UH methods were used for the direct runoff model. The model was calibrated to observed hourly discharge data for two events at USGS gage 2481000.

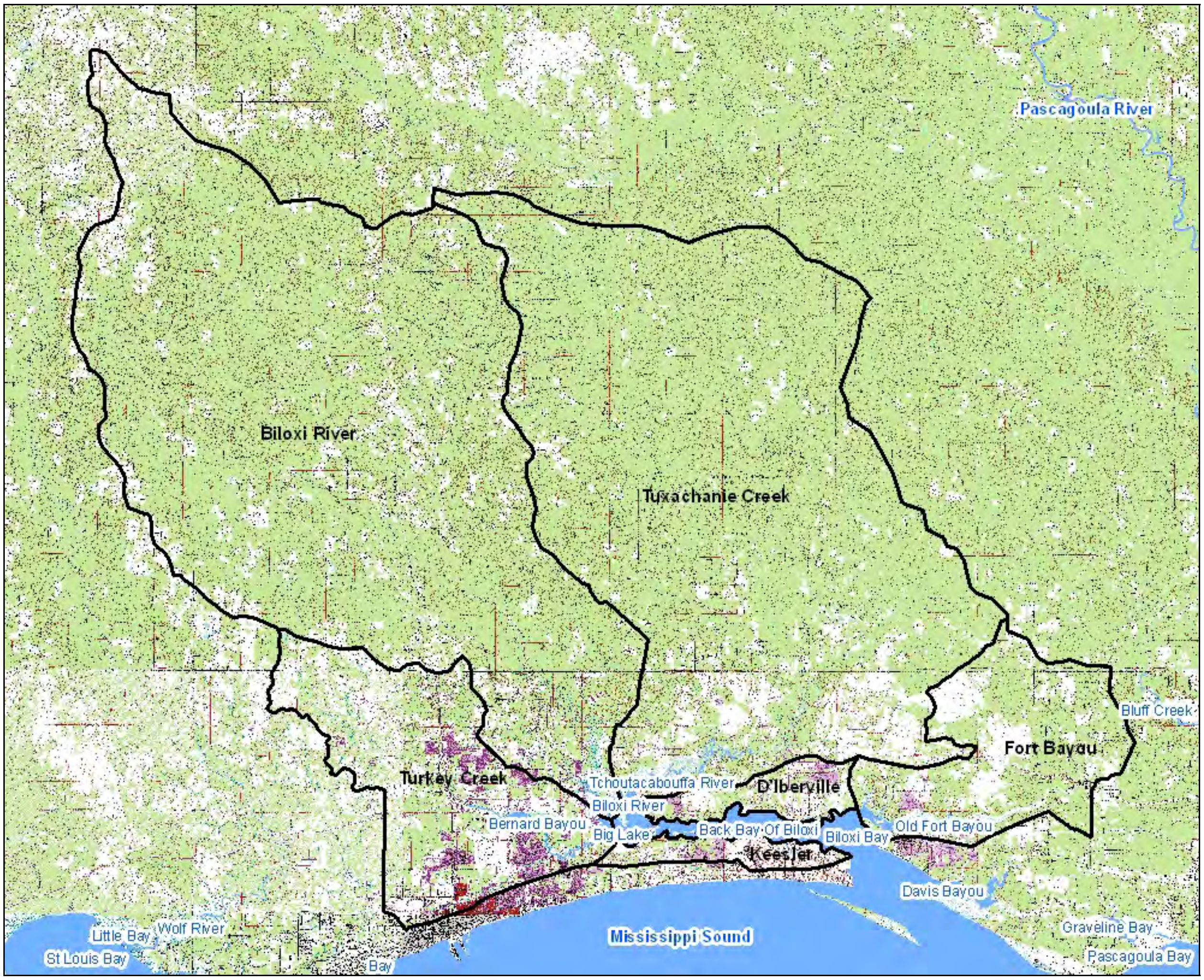

Figure 3.4.6-18. Biloxi Bay Watershed

Calibration results agree reasonable well with observed data as shown in Figures 3.4.6-19 and 3.4.6-20.

Ponding from the interior rivers behind the gates will depend partially on the elevation of the gulf when the gates are closed. Several historical stage hydrographs of hurricanes were reviewed to determine the duration of various stages along the gulf. From this review, it was determined that storms generally reach 4 ft NAVD88 and recede to that elevation within 24 hours. Using this information, various theoretical coincident rainfall events taken from T.P 40 were modeled to determine the resulting water surface elevations behind the barrier during the 24-hour period the gates are to be closed. A 10-yr rain was selected for the design condition, in accordance with studies cited above. The highest inflow period of the inflow hydrograph was used to compute changes in bay elevations in the 24-hour gate closure period.

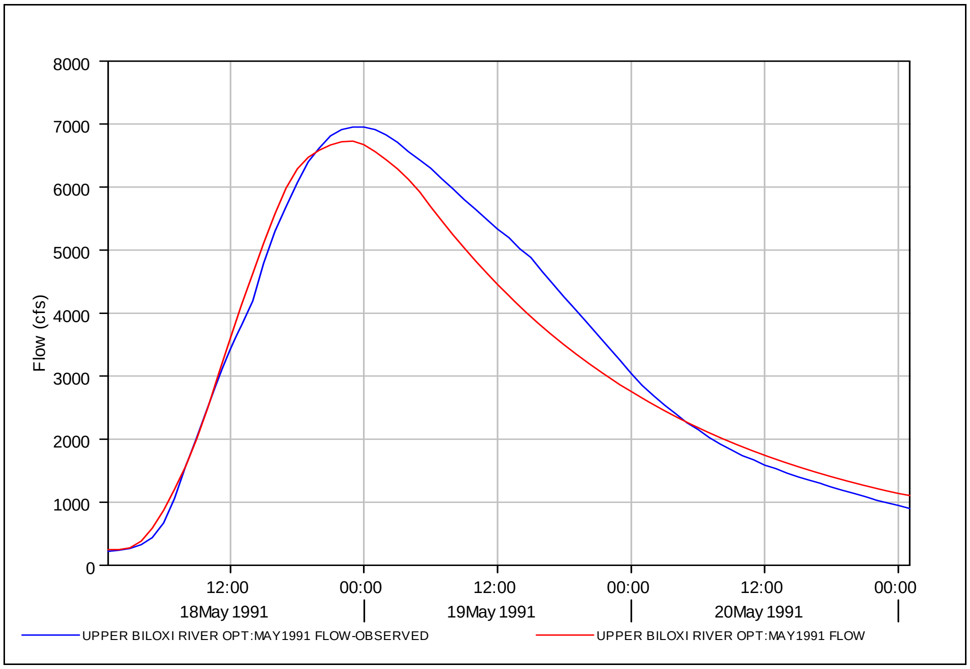

Figure 3.4.6-19. Biloxi Bay Watershed Calibration, 19 May 1991

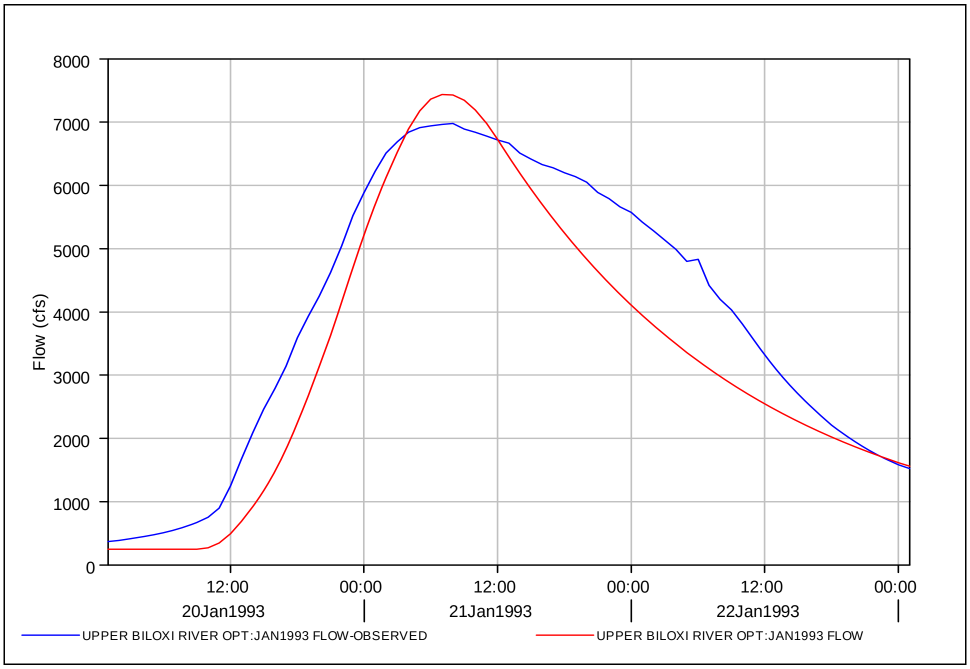

Figure 3.4.6-20. Biloxi Bay Watershed Calibration, 21 Jan 1993

Based on this method of analysis, the resulting elevations for the various storms are shown in Table 3.4.6.1-1, with the 10-yr elevation of 8.4 ft NAVD88 the design condition.

Table 3.4.6.1-1.
Biloxi Bay Ponding

Biloxi Bay 4 ft. Base Elevations	
Strom Event	**Bay Elevation (ft NAVD88)**
2-year	6.0
5-year	7.6
10-year	8.4
25-year	9.4
50-year	10.0
100-year	10.8

This area in Jackson County is approximated by the 8-ft ground contour line shown in Figure 3.4.6-21.

Figure 3.4.6-21. Biloxi Bay 10-yr Ponding to Elev. 8.4 ft NAVD88

3.4.6.5.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 24 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.6.5.3 Structural, Mechanical and Electrical

See sections 3.4.6.5.3.1 and 3.4.6.5.3.2.

3.4.6.5.3.1 Culverts

Reinforced concrete box culverts would be required at 2 locations, as described above, with the culvert requirement ranging from seven 7’ wide by 3’ high, to eleven 10’ wide by 4’ high water passages. Each of these culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a manually operated vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.6.5.3.2 Pumping Stations

Design hydraulic heads derived for the 2 pumping facilities included in the Jackson County Inland Barrier for the elevation 20 protection level were 15 and 10 feet and the corresponding flows required were 567,772 and 213,195 gallons per minute respectively. The facilities thus derived would consist of one plant having six, 60-inch diameter, 560 horsepower pumps and one having four, 54-inch diameter pumps each running at 290 horsepower.

3.4.6.5.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.6.5.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.6.5.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

3.4.6.5.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.6.5.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.6.8 Cost Summary. Construction costs for the various options are included in Table 3.4.6.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.6.8-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.6.5.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.6.6 Option B – Elevation 30 ft NAVD88

This option consists of an earthen levee around the most populated areas of Gautier The alignment of the levee is the same as Option A, above, and is not reproduced here. The only difference between the description of this option and preceding description of Option A is the height of the levee, pumping facilities, and the length of the levee culverts. Other features and methods of analysis are the same.

3.4.6.6.1 Interior Drainage

Interior drainage analysis and culverts are the same as those for Option A, above, except that the culvert lengths through the levees would be longer.

3.4.6.6.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into a friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will be armored by the placement of 12 inch thick gabion mattress filled with small stone for erosion protection during an event that overtops the levee. The armoring will be anchored on the front face by trenching and extend across the downstream slope and the 25 foot easement area beyond the toe. The front side of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.6.6.3 Structural, Mechanical and Electrical

See sections 3.4.6.6.3.1 through 3.4.6.6.3.3.

3.4.6.6.3.1 Culverts

Reinforced concrete box culverts would be required at 2 locations, as described above, with the culvert requirement ranging from seven 7’ wide by 3’ high, to eleven 10’ wide by 4’ high water passages. Each of these culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a manually vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.6.6.3.2 Pumping Stations

Design hydraulic heads derived for the 2 pumping facilities included in the Jackson County Inland Barrier for the elevation 30 protection level were 25 and 20 feet and the corresponding flows required were 567,772 and 213,195 gallons per minute respectively. The facilities thus derived would consist of one plant having six, 60-inch diameter, 1000 horsepower pumps, and one having four, 54-inch diameter pumps each running at 560 horsepower.

3.4.6.6.3.3 Dedicated Flood Barriers

There are two sites in Jackson County that would require special flood protection with the flood protection level set at elevation 40, the court facilities located immediately south of the protection line in downtown Biloxi and similar governmental facilities in downtown Moss Point.

The Biloxi facilities would require a three sided Tee Wall structure approximately 1410 feet long originating and terminating in the levee at its northwest and northeast ends. It would be fitted with four face sealing roller gates to close off the required street and driveway access points in time of flood.

The Moss Point Tee Wall would be similarly configured and would extend approximately 1552 feet. It would require two roadway closure gates.

3.4.6.6.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report

therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.6.6.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.6.6.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied building and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. Level 2 Security is the level to be applied to this option.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.6.6.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.6.6.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.6.8 Cost Summary. Construction costs for the various options are included in Table 3.4.6.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.6.8.-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.6.6.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.6.7 Option C – Elevation 40 ft NAVD88

3.4.6.7.1 Interior Drainage

The alignment of the levee is the same as Option A, above, and is not reproduced here. Differences between the description of this option and preceding description of Option A include the height of the levee, pumping facilities (because of the increased head), and the length of the levee culverts. The methods of analysis for interior drainage and computed flows are the same.

3.4.6.7.2 Geotechnical Data

Geology: Citronelle formation is found above the Interstate 10 alignment and is a relatively thin unit of fluvial deposits of Plio-Pleistocene age consisting of gravelly sand and silty sand layers. Typically the formation is 30 to 80 feet thick, except where it has filled eroded channels in the underlying formations. The sand in the formation has a variety of colors, often associated with the presence of iron oxides in the form of hematite or goethite. Thin discontinuous clay layers are found in some areas. The iron oxide has occasionally cemented the sand into friable sandstone, usually occurring only as a localized layer. Within the study area, this formation outcrops north of Interstate 10 and will not be encountered at project sites other than any levees that might extend northward to higher ground elevations.

Prairie formation is found along the rest of the Line 4 alignment within Jackson County. The Prairie formation is found southward of the Citronelle formation and is of Pleistocene age. This formation consists of fluvial and floodplain sediments that extend southward from the outcrop of the Citronelle formation to or near the mainland coastline. Sand found within this formation has an economic value as beach fill due to its color and quality. Southward from its outcrop area, the formation extends under the overlying Holocene deposits out into the Mississippi Sound.

Gulfport Formation is found along the coastline in most of western Jackson County at Belle Fontaine Beach. This formation of Pleistocene age overlies the Prairie formation and is present as well sorted sands that mark the edge of the coastline during the last high sea level stage of the Sangamonian Interglacial period. It does not extend under the Mississippi Sound.

Geotechnical: The inland barrier earthen levee section will have one vertical to three horizontal side slopes with a fifteen foot crest width. All work areas to receive the fill shall be cleared and grubbed of all trees and surface organics and all existing foundations, streets, utilities, etc. will be removed and the subsequent cavities backfilled and compacted. The levee will be constructed of sand clay materials obtained from off site commercial sources, trucked to the work area, placed in thin lifts and compacted to 95 percent of the maximum modified density. The final surface will not be armored for this option since the elevation of this option will not permit overtopping. The surface of the levee and all non critical surface areas will be subsequently covered by grassing. Road crossings will incorporate small gate structures or ramping over the embankment where the surface elevation is near that of the crest elevation. The elevation relationship of the crest and the adjacent railroad will be a governing factor. The surfaces will be paved with asphalt and the corresponding drainage will be accommodated. Those areas where the subgrade geology primarily consists of clean sands, seepage underneath the levee and the potential for erosion and instability must be considered. Final designs may require the installation of a bentonite concrete cutoff wall deep within the foundation. This condition will be investigated during any design phase and its requirement will be incorporated.

3.4.6.7.3 Structural, Mechanical and Electrical

See sections 3.4.6.7.3.1 through 3.4.6.7.3.4.

3.4.6.7.3.1 Culverts

Reinforced concrete box culverts would be required at 2 locations, as described above, with the culvert requirement ranging from seven 7’ wide by 3’ high, to eleven 10’ wide by 4’ high water passages. Each of these culverts was configured having nominally sized and reinforced structure walls and top and bottom slabs. Each water passage would be fitted with both a flap gate at the outlet end and a sluice gate placed near the center of the culvert with a vertical operator stem extending through an access shaft to the top of levee elevation.

3.4.6.7.3.2 Pumping Stations

Design hydraulic heads derived for the 2 pumping facilities included in the Jackson County Inland Barrier for the elevation 30 protection level were 35 and 30 feet and the corresponding flows required were 567,772 and 213,195 gallons per minute respectively. The facilities thus derived would consist of one plant having eight, 54-inch diameter, 1000 horsepower pumps, and one having seven, 42-inch diameter pumps each running at 500 horsepower.

3.4.6.7.3.3 Levee and Roadway/Railway Intersections

With the installation of Line 4 protection to elevation 40, three roadway intersections would have to be accommodated. It was determined that roller gate structures would suffice for all three of these locations.

3.4.6.7.3.4 Dedicated Flood Barriers

There are two sites in Jackson County that would require special flood protection with the flood protection level set at elevation 40, the court facilities located immediately south of the protection line in downtown Biloxi and similar governmental facilities in downtown Moss Point.

The Biloxi facilities would require a three sided Tee Wall structure approximately 1410 feet long originating and terminating in the levee at its northwest and northeast ends. It would be fitted with four face sealing roller gates to close off the required street and driveway access points in time of flood.

The Moss Point Tee Wall would be similarly configured and would extend approximately 1552 feet. It would require two roadway closure gates.

3.4.6.7.4 HTRW

Due to the extent and large number of real estate parcels along with the potential for re-alignment of the structural aspects of this project, no preliminary assessment was performed to identify the possibility of hazardous waste on the sites. These studies will be conducted during the next phase of work after the final siting of the various structures. The real estate costs appearing in this report therefore will not reflect any costs for remediation design and/or treatment and/or removal or disposal of these materials in the baseline cost estimate.

3.4.6.7.5 Construction Procedures and Water Control Plan

The construction procedures required for this option are similar to general construction in many respects in that the easement limits must be established and staked in the field, the work area cleared of all structures, pavements, utilities, trees, organics, etc. and the foundation prepared for the new work. Where the levee alignment crosses the existing streams or narrow bays, the alignment base shall be created by displacement with layers of crushed stone pushed ahead and compacted by the placement equipment and repeated until a stable platform is created. The required drainage culverts or other ancillary structures can then be constructed. The control of any surface water will be handled by temporary sheetpile cofferdams and pumping. The control of groundwater will be a series of wellpoints systems designed to keep the excavations dry to a depth and width sufficient to install the new work.

3.4.6.7.6 Project Security

The Protocol for security measures for this study has been performed in general accordance with the Risk Assessment Methodology for Dams (RAM-D) developed by the Interagency Forum for Infrastructure Protection (IFIP). This methodology has been used for physical security for the critical infrastructure throughout the Corps of Engineers. The determination of the level of physical security provided for each facility is based on the following critical elements: 1) threat assessment of the likelihood that an adversary will attack a critical asset, 2) consequence assessment should an adversary be successful in disrupting, disabling or destroying the asset and 3) effectiveness to prevent a successful attack against an operational component.

Three levels of physical security were selected for use in this study:

Level 1 Security provides no improved security for the selected asset. This security level would be applied to the barrier islands and the sand dunes. These features present a very low threat level of attack and basically no consequence if an attack occurred.

Level 2 Security applies standard security measures such as road barricades, perimeter fencing, and intrusion detection systems for unoccupied building and vertical structures and security lighting. The intrusion detection systems will be connected to the local law enforcement office for response during an emergency. Facilities requiring this level of security would possess a higher threat level than those in Level 1 and would include assets such as levees, access roads and pumping stations. This option will be best supported by the Level 2 Security.

Level 3 Security includes all of the features of Level 2 plus enhanced security measures such as the use of video cameras for real-time monitoring of the facility, monitors, motion detectors and alarm sound system in the occupied control buildings. Facilities requiring this level of security would

possess the highest threat level of all the critical assets. The surge barriers located in the bays, manned control buildings, and power plants would require this level of security.

3.4.6.7.7 Operations and Maintenance

The features that require periodic operations will be the exercising of the pumps and emergency generators at the various pump stations, the testing of the gate structures at the various road crossings, grass cutting of the levee slopes and toe areas and the filling of rilled areas within the embankment due to surface erosion. Scheduled maintenance should include periodic greasing of all gears and coupled joints, maintaining any battery backup systems, and replacement of standby fuel supplies.

3.4.6.7.8 Cost Estimate

The costs for the various options included in this measure are presented in Section 3.4.6.8 Cost Summary. Construction costs for the various options are included in Table 3.4.6.8-1 and costs for the annualized Operation and Maintenance of the options are included in Table 3.4.6.8-2. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost. The construction costs include real estate, engineering design (E&D), construction management, and contingencies. The E&D cost for preparation of construction contract plans and specifications includes a detailed contract survey, preparation of contract specifications and plan drawings, estimating bid quantities, preparation of bid estimate, preparation of final submittal and contract advertisement package, project engineering and coordination, supervision technical review, computer costs and reproduction. Construction Contingency developed and assigned at 25% to cover the Cost Growth of the project.

3.4.6.7.9 Schedule and Design for Construction

After the authority for the design has been issued and funds have been provided, the design of these structures will require approximately 12 months including comprehensive plans and specifications, independent reviews and subsequent revisions. The construction of this option should require in excess of two years.

3.4.6.8 Cost Estimate Summary

The costs for construction and for operations and maintenance of all options are shown below. Estimates are comparative-Level “Parametric Type” and are based on Historical Data, Recent Pricing, and Estimator’s Judgment. Quantities listed within the estimates represent Major Elements of the Project Scope and were furnished by the Project Delivery Team. Price Level of Estimate is April 07. Estimates excludes project Escalation and HTRW Cost.

Table 3.4.6.8-1.
Jackson Co Inland Barrier Construction Cost Summary

Option	Total project cost
Option A – Elevation 20 ft NAVD88	$126,900,000
Option B – Elevation 30 ft NAVD88	$224,800,000
Option C – Elevation 40 ft NAVD88	$266,000,000

Table 3.4.6.8-2.
Jackson Co Inland Barrier O & M Cost Summary

Option	O&M Costs
Option A – Elevation 20 ft NAVD88	$819,000
Option B – Elevation 30 ft NAVD88	$2,028,000
Option C – Elevation 40 ft NAVD88	$2,438,000

3.4.6.9 References

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3.5 Line of Defense 5 – Retreat and/or Relocation of Critical Facilities

3.5.1 General

Hurricanes are a naturally occurring phenomena that wreak havoc on natural and man-made environments through three different but related mechanisms: torrential rainfall, high winds, and storm surge. While each of these can produce costly outcomes in their own right, storm surge is typically the most damaging and particularly deadly. It is also the most difficult and costly to provide enduring and confident protection against. However, if one cannot be reached by storm surge by virtue of being on ground at elevation higher than any storm surge might reach, one cannot be directly damaged by it. The limit of storm surge represents the first line of avoidance to hurricane related damages. It therefore makes sense to identify the potential inland limit of storm surge so that prudent choices might be made by any and all regarding their exposure to damage by storm surge.

The primary measures identified for the project area include permanent acquisitions, floodproofing by elevation and other means, relocations of public buildings, flood preparedness and evacuation planning, public education, changes in the current municipal and county NFIP and building codes, implementation of either a transfer of development rights or purchase of development rights program, potential changes in zoning ordinances, development impact fees, and redirection of new development. These measures have been combined into several plans that can be implemented by either agencies of the Federal government or collaboratively by those agencies and state, county and local governmental units. In several cases, only local jurisdictions can implement some of the measures identified.

3.5.1.1 Existing Conditions

Computer simulations have predicted[1] how far inland storm surge will extend if the worse-case hurricane or maximum possible intensity (MPI) event hits the Mississippi coast.

This line of defense is shown in Figure 3.5-1. This line represents a line of safety where homes, facilities or transportation routes north of this line should not be directly damaged by storm surge. This would be an area where hospitals, schools, emergency response and management facilities, power stations, water supply facilities, or other critical infrastructure might be located. It would also represent an area whereby future development (commercial, industrial, or residential) might be redirected. The maximum water level along the Mississippi coastline was determined to be approximately 30 ft along the entire western half of the state and east of Pascagoula. The landward extent of the inundation indicates the storm surge reaches Interstate 10 for much of the western portion of the state. Lower peaks near Biloxi and Mobile Bay (24-27 ft) may be attributed to the protection afforded by the barrier islands. The line of defense accordingly approximates the 24 to 30 ft. (NAVD '88 datum) contours.

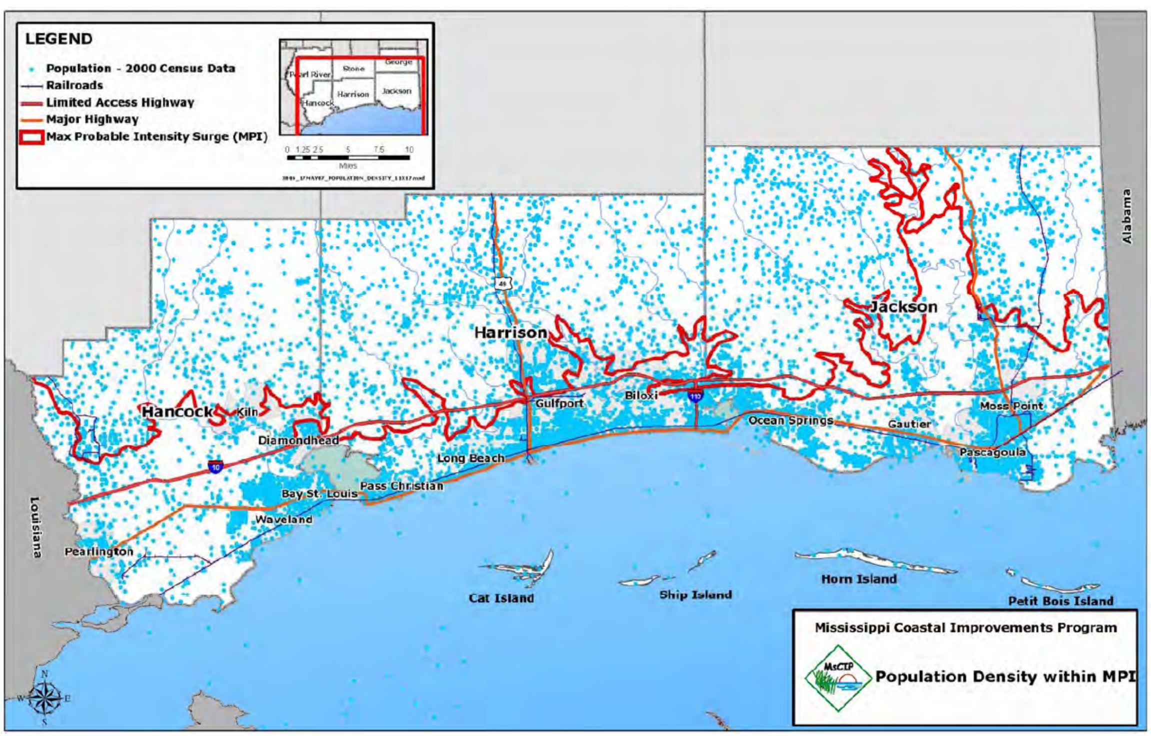

Figure 3.5-1. Maximum Probable Intensity Storm Surge Limits

[1] Storm surge modeling is described in Chapters 2.2 through 2.8.

This ‘line of defense’ is a naturally occurring measure against storm surge. This line of defense is not intended to suggest preferential protection against hurricane force winds. The line of defense is located based on storm surge only and is best considered jointly with riverine flood inundation maps published by FEMA for the purposes of promulgating the National Flood Insurance Program. FEMA is currently revising inland riverine regulatory flood maps. In keeping with historic hydrologic engineering practice, no probability of occurrence has been assigned to the MPI storm related surge, though in the future, USACE may adopt methods targeted at assigning risk to the occurrence of maximum probable storm events.

The area seaward of the line of defense is occupied by natural, rural, suburban, and urban environments and residential, commercial, and industrial development. Approximately 1/3 (visually estimated) of the coastal county areas fall within the estimated surge limits. With the exceptions of seawalls fronting Harrison County, Bay St. Louis, and the city of Pascagoula, there are no hurricane storm damage reduction structures in place. These structures provide little inundation protection over what the natural ground elevation would provide for and do not provide hurricane protection for surge events approaching or exceeding the 1 in 100 annual chance event.

3.5.1.2 Coastal and Hydraulic Data

The line of defense shown on Figure 3.5-1 is resultant of hydrodynamic modeling of six maximum possible intensity (MPI) storms with landfall points along the Mississippi coast were simulated to determine inundation limits for the Mississippi coastline. The six MPI storms made landfall at various points along the Mississippi Coast. All MPI storms were defined at their most intense point as having a minimum central pressure of 880 mb, radius to maximum winds of 36 n mi, and a forward speed of 11 kt. Peak water level envelopes from each of the six MPI simulations were computed. The six peak water level envelopes were then compared to compute the “peak of peaks”, which is considered the inundation limit along the entire Mississippi coastline.

3.5.1.3 Alternative Plans

There are no alternative alignments to this line of defense. The line of defense alignment could be changed or modified due to any of the following: (a) revised hydrodynamic modeling results; (b) the construction of storm damage reduction measures, such as levees and/or storm surge barriers; (c) sea level rise; (d) construction of other infrastructure (e.g. roadway embankments) that might materially obstruct or alter surge flow pathways.

A thorough discussion of non-structural alternative measures is provided in the Non-Structural Formulation Appendix.