<h1 style="text-align:center">Summary of the
MORGANZA TO THE GULF OF MEXICO, LOUISIANA
Final Post Authorization Change Report
May 2013</h1>

Purpose of the Post-Authorization Change Report

The Morganza to the Gulf of Mexico, Louisiana (Morganza to the Gulf) project authorized by the Water Resources Development Act (WRDA) of 2007 was developed well before Hurricane Katrina's devastating impact on the New Orleans hurricane levees in August 2005. Implementation of more robust Hurricane and Storm Damage Risk Reduction System (HSDRRS) design standards and other changes since project authorization caused the Morganza to the Gulf project to exceed the 20 percent cost increase limit specified in WRDA 1986, Section 902. The purpose of this Post Authorization Change (PAC) report is to seek re-authorization of the Morganza to the Gulf project. Once all required technical, legal, and policy reviews are complete, the report will ultimately be reviewed by the Assistant Secretary of the Army (Civil Works) and coordinated with the Office of Management and Budget as appropriate for submission to Congress.

Authority

The Morganza to the Gulf project was authorized by WRDA 2007 (PL 110-114, Sec 1001) at a total cost of $886.7 million as follows:

> "(24) MORGANZA TO THE GULF OF MEXICO, LOUISIANA.—
> (A) IN GENERAL.—The project for hurricane and storm damage reduction, Morganza to the Gulf of Mexico, Louisiana: Reports of the Chief of Engineers dated August 23, 2002, and July 22, 2003, at a total cost of $886,700,000, with an estimated Federal cost of $576,355,000 and an estimated non-Federal cost of $310,345,000.
> (B) OPERATION AND MAINTENANCE.—The operation, maintenance, repair, rehabilitation, and replacement of the Houma Navigation Canal lock complex and the Gulf Intracoastal Waterway floodgate features of the project described in subparagraph (A) that provide for inland waterway transportation shall be a Federal responsibility in accordance with section 102 of the Water Resources Development Act of 1986 (33 U.S.C. 2212)."

In accordance with the 2002 and 2003 reports of the Chief of Engineers, the Morganza project is authorized as a feature of the Mississippi River and Tributaries (MR&T).

Description of Authorized Project

The authorized MR&T project, Morganza to the Gulf, is designed to provide hurricane and storm damage reduction benefits while ensuring navigational passage and tidal exchange. The project is located about 60 miles southwest of New Orleans, LA, and includes Terrebonne Parish and the portion of Lafourche Parish between the eastern boundary of Terrebonne Parish and Bayou Lafourche. The 2002 and 2003 Chief of Engineers reports recommended a plan to reduce

hurricane and storm damages by providing a 100-year, or 1 percent annual exceedance probability (1% AEP), level of risk reduction including the features shown in figure S-1.
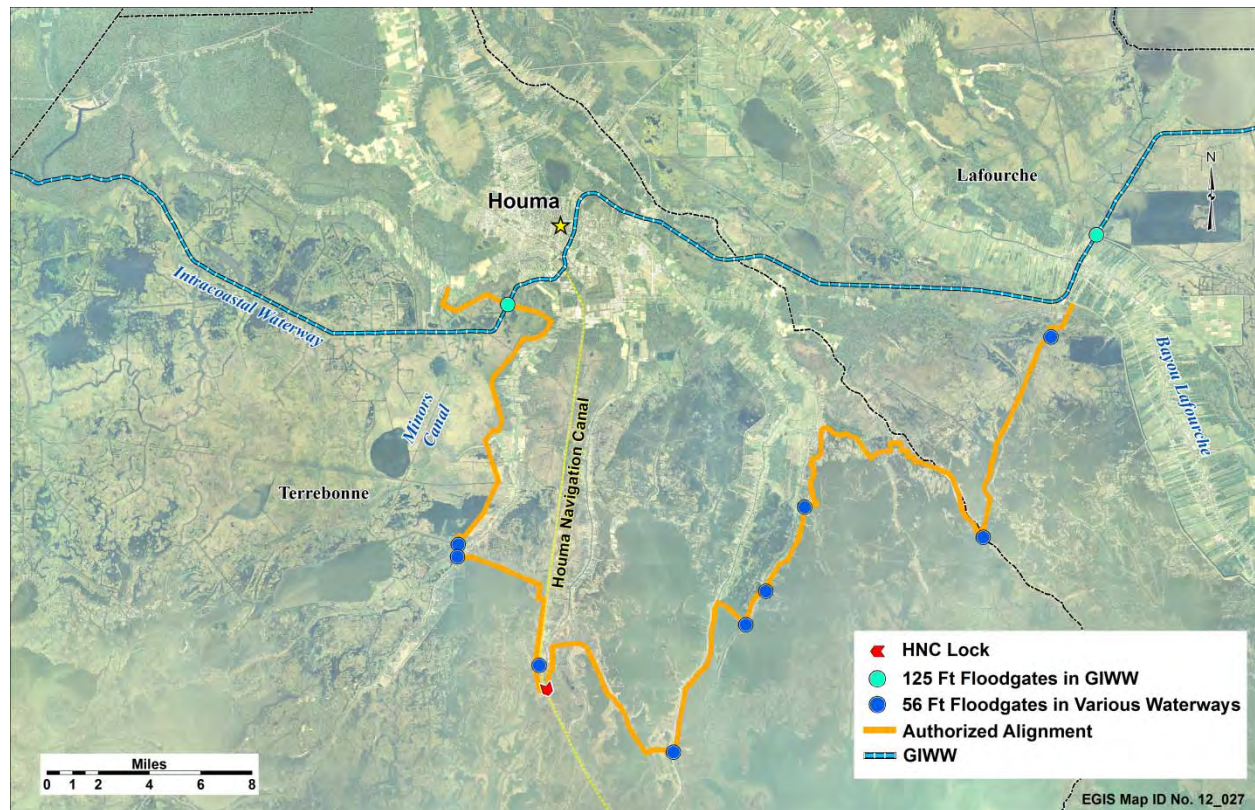

Figure S-1. Authorized Project Features

Project Purpose

The primary project purpose as described in the authorization is hurricane and storm damage reduction. The post-authorization plan does not include any changes in project purpose from the authorized plan. The purpose of the earthen levee system is to stop or slow down surge inundation. Floodgates provide storm damage reduction during tropical storms and allow currently navigable waterways to remain open to navigation during non-storm conditions. The purpose of the lock is to control saltwater intrusion at the Houma water treatment plant while allowing for navigation. Environmental control structures (box culverts) through the levee allow tidal ebb and flow.

Non-Federal Sponsor

The Louisiana Coastal Protection and Restoration Authority Board (LACPRAB) and the Terrebonne Levee and Conservation District (TLCD) have expressed their intent to be non-Federal co-sponsors for the Morganza to the Gulf project (hereafter referred to as the non-Federal sponsor). In a letter dated December 21, 2012, the non-Federal sponsor expressed commitment and understanding of non-Federal cost share responsibilities for construction and operation and maintenance, repair, replacement and rehabilitation (OMRR&R). Section 1001(24) of WRDA 2007 specifies Federal responsibility for OMRR&R of the Houma

Navigation Canal (HNC) lock complex and the Gulf Intracoastal Waterway (GIWW) floodgate features that provide for inland waterway transportation in accordance with Section 102 of WRDA 1986, as amended. The non-Federal sponsor is responsible for OMRR&R of all other project features. Additional responsibilities of the non-Federal sponsor are listed in section 8.3 of this report.

Funding Since Authorization

The Energy and Water Appropriations Act of 1998 (PL105-62) included funds to initiate design on the HNC lock feature of the Morganza to the Gulf project, which initiated the Preconstruction Engineering and Design (PED) phase for the HNC lock feature in advance of completing the Feasibility Report (2002) and signing of Chief's report (also 2002). The USACE and non-Federal sponsor signed a Design Agreement for the HNC lock in January 2000, and the non-Federal sponsor first contributed matching funds for PED in 2000. The first non-Federal contributions to the overall Morganza to the Gulf project were in September 2002. Approximately $61,650,000 has been allocated for the Morganza to the Gulf PED phase, which includes the PAC report. Most of the PED funds have been spent on engineering design and geotechnical investigations rather than on the PAC feasibility-level analysis.

Per U.S. Army Corps of Engineers (USACE) guidance, study costs for the PAC report are being cost shared 75 percent Federal and 25 percent non-Federal. The PAC study is cost shared under a Design Agreement originally executed on May 22, 2002 and amended on March 24, 2005 and January 11, 2011. While the Design Agreement provides for 75/25 cost share during design, WRDA 1986 (PL99-662), Section 105(c), stipulates that the non-Federal share of the cost of design is the same percentage as the non-Federal share for construction, which in this case is 35 percent. The design cost is shared per the percentage of construction cost with 25 percent being collected from the non-Federal sponsor during the Design Agreement and the remaining 10 percent collected in the first year after the Project Partnership Agreement is executed. Pending re-authorization, the construction cost share would be 65 percent Federal and 35 percent non-Federal. No Federal funds have been appropriated for construction of the Morganza to the Gulf project.

Project History Since Authorization

In 2008, a reconnaissance-level planning analysis and detailed programmatic cost estimate was completed for the purpose of determining whether or not there would still be a Federal interest in the project with post-Hurricane Katrina design criteria incorporated and whether a feasibility-level PAC report should be initiated. The 2008 analysis determined that the Morganza to the Gulf project updated with the HSDRRS criteria would still be economically justified, and the PAC re-evaluation study was initiated in early 2009.

Design Criteria Changes Since Authorization

Several policy, procedural, and design criteria changes have been made since the 2002 Morganza to the Gulf Feasibility Report was completed. Lessons learned from Hurricane Katrina and other recent storms have been incorporated into new HSDRRS design guidelines. These peer-reviewed guidelines were developed in response to recommendations made by the Interagency Performance Evaluation Task force (IPET), a team composed of members from USACE,

industry and academia that evaluated the Greater New Orleans levee system after Hurricane Katrina. In accordance with Assistant Secretary of the Army (Civil Works) direction, USACE is applying the HSDRRS guidelines to all hurricane levee system work in the New Orleans District, including the Morganza to the Gulf PAC project. The HSDRRS guidelines provide a comprehensive collection of best practices and were developed to provide redundancy, resiliency, and robustness of the interfaces between structures, materials, and members of the hurricane risk reduction system for the desired level of risk reduction.

The HSDRRS guidelines include some criteria that are more stringent than required for other USACE structures. Changes leading to larger designs and higher costs for the Morganza to the Gulf post-authorization project include the following:

- **Increase in Hydraulic Design Elevations** – Storm surge modeling in the 2002 report was based on only 17 tropical storms and did not consider relative sea level rise in the model. New storm surge modeling predicts water levels based on 115 theoretical storms and incorporates the effects of relative sea level rise within the model. In addition, the 2002 1% AEP water levels were based on the 50 percent confidence values, which have a 50 percent chance of being under-predicted. The new design guidelines require levees to be designed based on the 90 percent confidence values, which have only a 10 percent chance of being under-predicted. All of these factors result in prediction of higher surge and waves, and wave run up used to set levee elevations.

- **Change from I-Walls to T-Walls** – In the 2002 report, floodwalls could be based on I-wall designs. Under the new guidelines, I-walls are not permitted in most cases and have been replaced with more robust and more expensive T-walls.

- **Increase in Geotechnical Stability Factor of Safety** – The analysis method for global stability changed, leading to a higher factor of safety, resulting in taller and wider levees and cost increases.

- **Addition of Structural Superiority** – All new structures that are difficult to construct because of disruptions to navigation or traffic, large utility crossings, or requiring cofferdams must be designed with a minimum of 2 ft of additional wall height resulting in cost increases.

These increases in project size and costs resulted in the project exceeding the WRDA 1986 Section 902 limit.

Post-Authorization Change Alternatives

The PAC study considered two primary hurricane and storm damage reduction alternatives in detail: a 3% AEP system (pre-Katrina 100-yr alternative) and a 1% AEP system (post-Katrina 100-yr alternative). The 3% AEP and 1% AEP alternatives both follow the same levee alignment, which is based on the authorized alignment, but with some modifications that have occurred since authorization. Of the two alternatives, the 1% AEP alternative has the greater net benefits, lower residual risk, and greater adaptability to future sea level rise.

Description of the 1% AEP Post-Authorization Project

The post-authorization 1% AEP Morganza to the Gulf hurricane and storm damage risk reduction system based on post-Katrina HSDRRS criteria consists of 98 miles of grass-covered earthen levees tying into US 90 near the town of Gibson in Terrebonne Parish and Hwy 1 near Lockport, LA in Lafourche Parish (see figure S-2). Levee elevations for base conditions (2035) range from 10.5 to 24 ft NAVD88, and final levee elevations (2085) range from 15 to 26.5 ft NAVD88 with final levee widths from 282 to 725 ft.

Structures include a lock on the HNC, 22 floodgates on navigable waterways (3 on Federally-maintained navigation channels and 19 on other canals and bayous), 23 environmental water control structures, 9 road gates, and fronting protection for 4 existing pumping stations. Structures on Federally-maintained navigation channels include the HNC lock and floodgate (250-ft sector gate) and two floodgates on the GIWW (i.e. GIWW East, a 125-ft sector gate east of Bayou Lafourche, and GIWW West, a 125-ft sector gate west of Houma). Fourteen 56-ft sector gates and five 20- to 30-ft stop log gates are located on various waterways that cross the levee system. Structure elevations range from 17 to 33 ft NAVD88.

Levees would be covered in grass to increase resilience in the case of wave overtopping. All of the transitions between levees and floodwalls would be armored with reinforced concrete scour protection.

Changes in Location of Project

Figure S-2 shows the location of both the authorized and post-authorization (current) alignments. A few reaches in the authorized project were refined during PED (reaches A, G, H, J, and L), and the original alignment had to be extended to the west (Barrier Reach) and to the east (Larose reaches) because surge modeling now indicates that with projected sea level rise, the 1% AEP surge may be able to cross the Bayou Black and Lafourche ridges. Surge modeling for the 2002 feasibility report produced lower stages, which indicated that surge would not cross the ridges. Some levee reach footprints are also wider because of the higher post-Katrina design elevations and the HSDRRS increase in Geotechnical Stability Factor of Safety.

Figure S-2. Post-Authorization Morganza to the Gulf Project Map
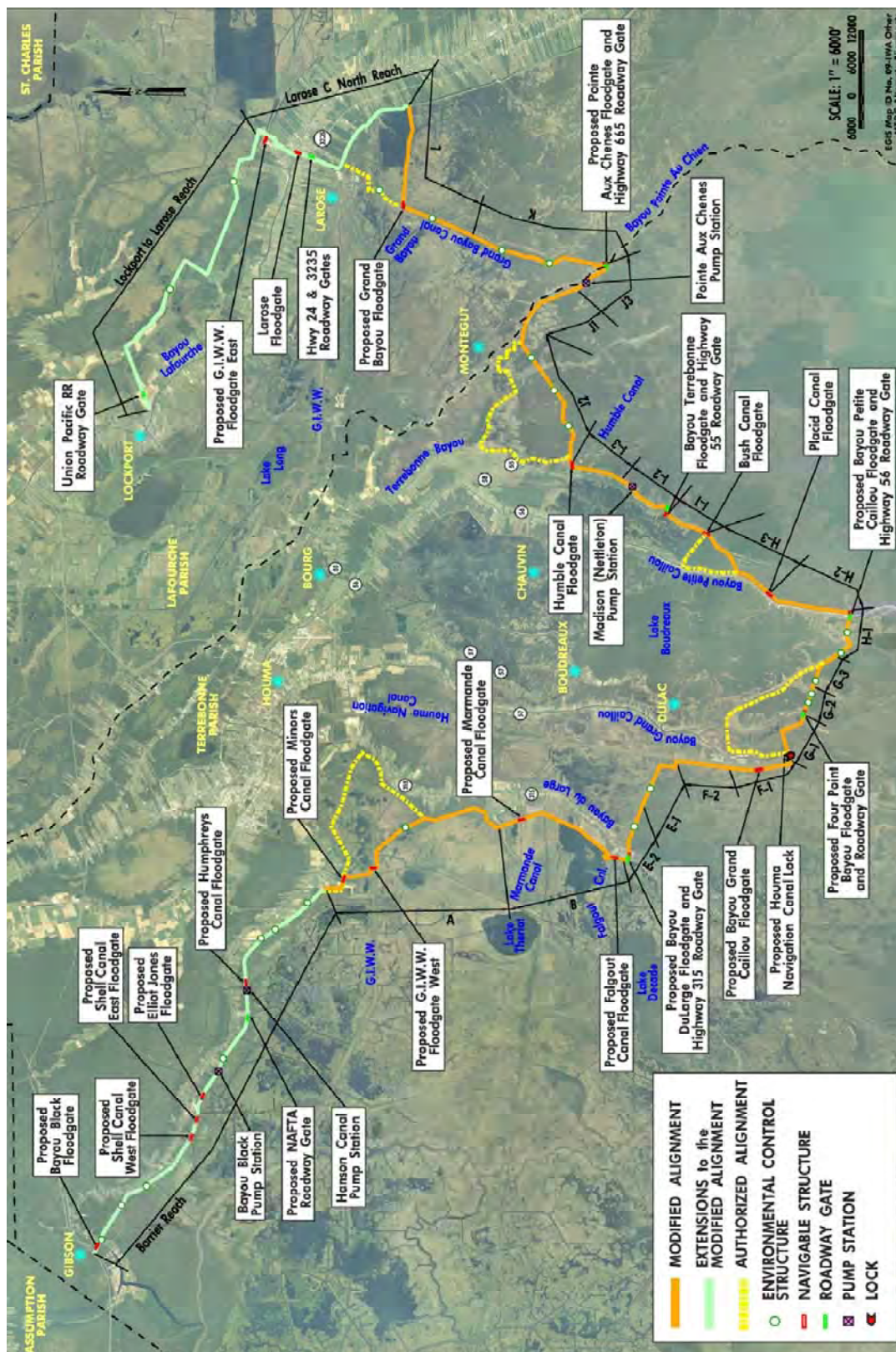
Changes in Scope of Authorized Project

The post-authorization plan has the same target level of risk reduction as the authorized plan (1% AEP). More rigorous storm surge modeling and more robust post-Katrina HSDRRS standards expanded the scope of the authorized project as follows:

- **Total levee length** increased from 72 miles to 98 miles. Post-authorization refinements initially reduced the length of the 72-mile authorized alignment by approximately 11 miles, however, post-Katrina surge modeling, which considered higher rates of relative sea level rise and higher surge and waves in the future, demonstrated that the authorized project could potentially be flanked at either end. The proposed levees were also extended to address potential costs to complete the Morganza to the Gulf system in the event that other previously proposed hurricane and storm damage reduction projects in the area are never authorized and/or constructed. The alignment was extended 16 miles to the west and 21 miles to the east to complete the system.

- **Levee and structure elevations** increased by several feet in all reaches and more than doubled in some reaches. Authorized levee and structure elevations along the authorized alignment vary from a minimum elevation of 9 ft National Geodetic Vertical Datum (NGVD) to a maximum elevation of 15 ft NGVD. Post-authorization levee elevations (for future conditions at year 2085) range from 15 ft North American Vertical Datum (NAVD88 epoch 2004.65) at the lowest point to 26.5 ft NAVD88 at the highest point, and structure elevations range from 17 ft NAVD88 at the lowest structure to 33 ft NAVD88 at the highest structure. Note the different datum for the authorized (NGVD) and current (NAVD88) elevations. The change in elevation due to datum differences varies by location, and is around 0.5 to 1.5 ft. Most of the increase is attributable to higher predicted surge and waves and post-Katrina HSDRRS design criteria. For the structures, 2 ft of the increase is attributable to the HSDRRS structural superiority requirement.

- **Levee widths** increased by several hundred feet and are now four to eight times wider. Authorized levee widths range from 40- to 200-ft wide; post-authorization levee widths range from 282- to 725-ft wide. The increases in levee widths are attributable to the increases in levee heights and the HSDRRS increase in Geotechnical Stability Factor of Safety.

- **The HNC lock complex and GIWW floodgate features**, which are located on Federally-maintained navigation channels, are generally the same except for the following changes: The GIWW West floodgate near Houma was re-designed to eliminate one of the two sector gates. Both the GIWW West and GIWW East floodgate sizes were reduced from 175-ft to 125-ft sector gates consistent with the authorized channel width and as a cost-saving measure. The HNC floodgate width increased from 200 to 250 ft. The reason the HNC floodgate had to be widened is that the floodgate's pre-Katrina arrangement is no longer technically feasible given the increase in design height. HNC structure sill depths may increase by 5 ft as part of a requested sponsor funded additional work item.

- **The number of floodgates on other canals and bayous** increased from 9 to 19 for two reasons. One reason is that during PED, several bayous were identified as being used for

navigation, but were not noted as such in the original feasibility study. Another reason is that the western and eastern levee extensions contain several navigable bayous. The assumption for the PAC report was that all currently navigable bayous must remain navigable in the future; the number/sizes of gates may be reduced during PED with additional data on navigation.

- **The number of environmental flow control structures** increased from 12 to 23 sets of concrete box culverts with sluice gates. In the 2002 feasibility report, a single design criterion stated that the environmental control structures should be sized to "return the specified wetland storage area elevations to pre-storm elevations within 14 days." The PAC report includes a more refined set of criteria, including precipitation event conditions, water level, velocity, and box culvert design criteria.

- **The number of road gates and modifications to existing pump stations** also increased because of the western and eastern levee extensions.

- **Environmental mitigation features** for the project authorized in WRDA 2007 included creation of 1,352 acres of marsh habitat. At this time, mitigation features for the post-authorization project have only been identified for the constructible features (i.e. levee reaches F-1, F-2, and G-1; the HNC lock complex; and the Bayou Grand Caillou floodgate). Mitigation for direct and indirect impacts of the constructible features includes approximately 394 acres intermediate marsh restoration, 358 acres brackish marsh restoration, and 883 acres saline marsh restoration. For the remaining programmatic features, mitigation costs and land requirements were estimated, but the exact number of acres will be determined in the future as more specific designs are completed and impacts are assessed in future supplemental NEPA documents.

- **A preliminary nonstructural buyout plan** has been developed for high risk areas outside the proposed levee system. Hydrologic modeling indicates that the levee could potentially increase storm surge flooding in these areas; however, additional modeling and analysis would need to be conducted during PED. For purposes of this report, the worst case scenario (most expensive option) has been assumed, which would be a 100 percent buyout of all structures in the impacted areas (approximately 1,000 structures). Should this scenario prove to be the appropriate mitigation method, approximately 2,500 people would need to be relocated to areas behind the Federal levee system.

Changes in Project Costs and Benefits

Both project costs and benefits have increased significantly since authorization. As shown in table S-1, project first costs have increased by an order of magnitude. As described in the previous section, the primary reasons for the cost increases are changes in predicted surge elevations and more robust post-Katrina HSDRRS guidelines. As a result, levee lengths, levee and structure heights, and levee widths have increased significantly.

As shown in table S-2, benefits have increased proportionately to costs, because the same, updated storm surge modeling indicates that more structures have a higher probability of getting flooded. The 1% AEP surge elevations have increased from 4 to 6 ft, to 12 to 14 ft, so the 1% AEP floodplain is now larger and incorporates more structures. The 2009/2010 PAC inventory

included approximately 53,000 structures, which is over twice the number of structures in the original 1997/1998 feasibility study inventory, which included approximately 26,000 structures.

Table S-1. Changes in Project First Costs ($ Millions) by Project Feature

Work Breakdown Structure No. & Civil Works Feature Description	Project as Authorized by Congress (WRDA 2007)	Authorized Project (Updated)	3% AEP PAC Alternative	1% AEP PAC Alternative
Effective Price Level	Oct 2006	Oct 2012	Oct 2012	Oct 2012
02 Relocations	43	51	274	291
05 Locks	169	228	530	622
06 Fish & Wildlife Facilities	55	66	619	941
11 Levees & Floodwalls	253	314	2,467	5,351
15 Floodway Control & Diversion Structures	219	262	763	1,068
Construction Totals:	$739	$921	$4,652	$8,273
01 Lands and Damages	10	11	339	355
30 Planning, Engineering & Design	87	115	574	1,006
31 Construction Management	50	67	385	631
Project Cost Totals:	**$887**	**$1,113**	**$5,950**	**$10,265**

Note: Authorized project costs in 2006 and 2012 price levels developed using Form 17 (project cost estimates used to support budget submissions).

Table S-2. Changes in Annual Costs and Benefits ($ Millions)

(All costs and benefits in $millions)	Project as Authorized by Congress in WRDA 2007	Authorized Project (Updated)	3% AEP PAC Alternative	1% AEP PAC Alternative
Effective Price Level, Interest Rate:	Oct 2006, 5.125%	Oct 2012, 3.75%	Oct 2012, 3.75%	Oct 2012, 3.75%
Structures, Contents and Vehicles	91	98	609	959
Emergency Costs	9.6	10	37	54
Boats	1.6	2	<1	<1
Agricultural	2	2	N/A	N/A
Water Supply	0.3	0.3	<1	<1
Avoided Structure Raising Costs	N/A	N/A	10	10
Total Equivalent Annual Benefits	104	113	657	1,023
Annual Costs	49	36	442	716
Benefit-Cost Ratio	2.12	3.16	1.48	1.43
Net Benefits	55	77	214	307

Note: Benefits for boats in 2006 price levels updated to 2012 price levels using the Energy Information Administration Diesel Fuel Price on Gulf Coast; Agricultural benefits updated using the National Agricultural Statistical Service ratio of prices received and paid; all other benefit categories updated using the Composite Index Civil Works Construction Cost Index System. Agricultural benefits were calculated for the 2002 authorized plan, but not for the PAC because a certified model was not available to incorporate risk analysis, and the agricultural benefits were a small percentage of the total PAC benefits.

Emergency cost reductions for the authorized project were based on pre-Hurricane Katrina/Rita information, and the emergency cost reductions for the post-authorization project were based on post-Hurricane Katrina/Rita information. Also, the emergency cost reductions for the post-authorization project include damages to transportation infrastructure, while these damages were not included in the emergency cost reductions for the authorized project.

The project benefit-to-cost ratio in the 2002 feasibility report was 1.43 based on 2000 price levels and an interest rate of 6.625 percent. The benefit-to-cost ratio of the 1% AEP post-authorization project is 1.43 based on 2012 price levels and a 3.75 percent interest rate. The benefit-to-cost ratio based on 2012 price levels and a 7.0 percent interest rate is 0.69 (required by ER 1105-2-100, Appendix H, Exhibit H-11).

Changes in Cost Allocation and Apportionment

No changes in cost allocation have occurred since authorization; all costs are for hurricane and storm damage reduction. The post-authorization project does not include any changes in the local cooperation requirements or changes in Federal/non-Federal cost share percentages. The cost apportionment would be 65 percent Federal and 35 percent non-Federal for construction.

The non-Federal sponsor requests that the entire non-Federal share be provided as work-in-kind rather than cash. The non-Federal sponsor would focus their effort on earthen levee construction (multiple lifts) concentrated between Reach E-2 and Reach L. The non-Federal sponsor would also construct floodgates on some bayous within the same geographical area, such as Bush Canal, Placid Canal, Bayou Pointe aux Chenes, Bayou Terrebonne, Humble Canal and Bayou Petit Caillou. Details regarding specific features and schedules for work-in-kind would continue to be coordinated between the USACE and the non-Federal sponsor throughout the design and construction phase of the project. Table S-3 compares the Federal and non-Federal cost-share of the authorized project and post-authorization project.

Table S-3. Changes in Cost Apportionment (Costs in $1000s)

(All costs in $1000s)	Authorized Project		Post Authorization 1% AEP Plan
	Authorized in WRDA 2007	Updated Price Levels	
Effective Price Level:	Oct 2006	Oct 2012	Oct 2012
Total Project Cost	$886,700	$1,113,000	$10,265,100
Federal Share (65%)	576,355	723,450	6,672,315
Non-Federal Share (35%)	310,345	389,550	3,592,785
Value of Proposed Work-in-Kind	139,655	177,934	2,947,262
LERRDs	51,970	60,357	645,523
Additional Cash Required	118,720	151,259	0

Note: Authorized project costs (including LERRDs) in 2006 and 2012 price levels developed using Form 17 (project cost estimates used to support budget submissions). Authorized project work-in-kind and cash breakdown in 2006 and 2012 price levels estimated based on percentage from 2002 feasibility report (work-in-kind was 45% of non-Federal share).

Public Involvement and Environmental Considerations

A Revised Programmatic Environmental Impact Statement (RPEIS) was prepared to update environmental and socioeconomic impacts of the project. Given the size and complexity of the Morganza to the Gulf project and the uncertainties associated with unidentified borrow sources and future indirect impacts, most of the RPEIS is at a broad, programmatic level. The RPEIS

includes a more in-depth analysis of features for which borrow sources have been identified and that could be constructed in the near future, including the HNC lock complex, the Bayou Grand Caillou floodgate, and levee reaches F and G-1. For these features, the RPEIS provides sufficient detail so that no further environmental clearances would be needed upon signing of a Record of Decision. The remaining programmatic features would require supplemental National Environmental Policy Act (NEPA) documents (i.e. EISs or Environmental Assessments) before they could be constructed.

The Draft PAC/RPEIS documents were open to public comment from January 4, 2013 to February 19, 2013. A public meeting was held in Houma, LA on January 31, 2013. The most common concern in the top 10 comment themes (representing over 50 percent of the comments) are the potential environmental impacts of the project and associated mitigation requirements. Federal and State agencies noted that in the future there is a potential for adverse indirect and cumulative impacts to wetlands, fisheries, water quality, and navigation assuming there would be increased frequency and duration of water control structure closures as sea level rise accelerates. In response to these concerns, the Final PAC/RPEIS includes a quantitative analysis of the range of potential indirect impacts, including a mitigation plan, for the constructible features, and a qualitative analysis of the potential impacts for the entire project.

The remaining 9 of the top 10 comment themes (from most common to least common with each theme representing less than 10 percent of the total comments) were related to communication, coordination, and consideration of socioeconomic/cultural impacts; plan realignment or reformulation; design standards, RSLR, & constructability; project economics; nonstructural measures; the relationship of Morganza to other plans or projects, such as the State Master Plan and Louisiana Coastal Area (LCA) projects; GIWW floodgate size change, modeling, and impacts; non-Federal contributions; and general support for the project.

Project Benefits, Costs, and BCR Risk and Uncertainty

Although the costs, benefits, and project benefit-to-cost ratio are presented as "most likely" values in this report, there are large uncertainties surrounding single values. Uncertainty and variability are intrinsic in water resources planning, modeling, and design. Risk and uncertainty arise from measurement errors and from the underlying variability of complex natural, social, and economic situations. Major sources of uncertainty associated with the Morganza to the Gulf project that could impact the benefit-cost ratio include the following:

- **Uncertainty surrounding engineering models and inputs.** The uncertainty surrounding three key engineering parameters (ground elevations, stage-probability curves, and performance of existing and proposed levees) was quantified in the HEC-FDA model. Uncertainty in these engineering inputs arises from imperfect data and coastal storm damage models that represent complex meteorological and hydrologic conditions across a highly variable and changing coastal landscape. In general, the standard deviation associated with estimated still water elevations is greater than a foot. In reviewing the HEC-FDA risk analysis, the USACE Risk Management Center concluded that the lack of data on existing levees resulted in underestimating the without-project damages. The analysis performed on the with-project conditions understated the performance of the Federal levees as well as overstated the residual damages due to the interior/exterior relationships used in the economic model. The net effect of understating

the without-project damages and overstating the with-project residual damages is that the benefit-cost ratio is understated.

- **Uncertainty surrounding economic inputs.** The uncertainty surrounding four key economic variables (structure values, contents-to-structure value ratios, first floor elevations, and depth-damage relationships) was quantified in the HEC-FDA model. Uncertainties arise from measurement errors associated with collection of this economic input data and extrapolation of a small set of samples to a large, highly variable study area. The samples used to develop the contents-to-structure value ratios (CSVRs) for the various residential and non-residential damage categories could not be randomly selected due to the difficulty in obtaining volunteers to participate in the surveys. Consequently, there is the risk that the CSVRs used in the economic analysis may not accurately reflect the true mean CSVRs of the population of residential and non-residential properties in the evaluation area. The uncertainty surrounding the first floor elevations assigned to structures is less than a third of a foot; however, adjustments were made to the structure inventory assuming that structures that are repetitively damaged (i.e. in the 10-yr damage floodplain) will be rebuilt over time to higher elevations such that they are outside of the 100-yr floodplain. If more structures than expected are removed from the floodplain, the benefits would be lower than estimated. If fewer structures than expected are removed from the floodplain, the benefits would be higher than estimated.

- **Changes in the discount rate.** The 2013 discount rate of 3.75 percent is at a record low. All other variables being equal, the breakpoint for project justification is around a discount rate of 5.25 percent, meaning that if the discount rate rises above 5.25 percent and all other variables remain unchanged, the project would no longer have a benefit-cost ratio greater than unity.

- **RSLR uncertainty**. The National Research Council predicts that eustatic sea level (not accounting for subsidence) could increase 1 to 4 feet by 2085. Assuming that historic, local subsidence rates continue into the future, the amount of RSLR (sea level rise plus subsidence) for the Morganza to the Gulf area could range from 1.7 to 4.8 ft. The RSLR scenario selected for calculating the benefit-cost ratio assumes an intermediate level of approximately 2.4 ft RSLR by 2085. If the combined effect of RSLR is lower than expected, project benefits and costs would both be lower than estimated. If actual RSLR is higher than 2.4 ft, project benefits and costs would be higher than estimated. Based on a sensitivity analysis, the net effect on the benefit-cost ratio is minimal (a few tenths of a point) and the project would still be justified under any RSLR scenario (holding all other variables the same).

- **Total project cost uncertainties.** For the Morganza to the Gulf project, the high risk cost items are the structural and geotechnical uncertainty, steel cost, fuel cost, unidentified borrow pit for hauled in material, and construction modifications. A detailed cost risk analysis has been conducted and resulting project feature contingencies range from 26 to 35 percent, which results in a total project contingency of around $2.3 billion. Although there is a risk that costs could be underestimated, there is also the risk that costs could be overestimated.

- **Mitigation cost uncertainties for indirect impacts.** Compensatory mitigation costs for the 1% AEP plan are based on direct construction impacts assuming that tidal ebb and flow would be maintained by the environmental control structures designed throughout the levee alignment such that there would be minimal indirect impacts. However, if future sea level rise predictions come to fruition, these environmental control structures would have to be closed more frequently resulting in indirect impacts. Based on this assumption, the Habitat Evaluation Team estimates the constructible features could impact an additional 721 to 1,442 acres (216 to 418 AAHUs), which would require additional compensatory mitigation at a cost of $60 million to $120 million. Mitigation costs resulting from additional indirect impacts from the programmatic features would be covered in future supplemental NEPA documents.

- **Site-adapting the HSDRRS standards.** Estimated project costs in the PAC report are the best available and compliant with current HSDRRS standards. Based on a risk assessment of the Morganza to the Gulf project conducted after the PAC analysis was complete, risk-based modifications to current design criteria have the potential to reduce the total project cost estimates reflected in the PAC report. The USACE is also conducting a risk assessment to ensure risk is addressed consistently across the country. Once this assessment is complete, the results may be applied to the Morganza to Gulf project area. Such modifications would be made to designs and costs during the next phase of implementation, Pre-construction Engineering and Design (PED). Any decrease in costs would increase the benefit-cost ratio assuming the benefit remain unchanged.

Findings

The District Commander has updated the Morganza to the Gulf of Mexico, Louisiana authorized project and finds that the updated 1% AEP plan is economically justified, environmentally acceptable and engineeringly sound. Post-Katrina engineering design criteria and standards for gulf coast communities were applied to reduce the potential of loss of life and property from coastal storms. These engineering practices were developed using the findings of the *Interagency Performance Evaluation Task Force* including key lessons learned from Hurricane Katrina and their implications for future hurricane preparedness and planning for south Louisiana. Project modifications were also found necessary to address developments after the project was authorized, including community resettlement patterns after Katrina, to incorporate improved water control elements and floodgates, and to update other outmoded aspects of the authorized project to more effectively provide the utility of function originally intended by Congress.

The District Commander has considered all the significant aspects of this study including the environmental, social, and economic effects, the engineering feasibility, and the comments received from other resource agencies, the non-Federal sponsors, and the public and has determined that the updated authorized project presented in this report remains in the overall public interest and a justified expenditure of Federal funds. In coordination with other Federal, State of Louisiana, and parish agencies, USACE planned and would design a project that serves the needs of the nation.

The updated total cost for the project is $10,265,100,000 (October 2012 dollars) inclusive of associated investigation, environmental, engineering and design, construction, supervision and administration, and contingency costs (and exclusive of OMRR&R costs). The fully funded total project cost (includes inflation) is approximately $12,872,846,000. The project would be funded 65 percent by the Federal Government and 35 percent by the non-Federal sponsors, and subject to the implementation requirements specified in section 8 of this report.

The OMRR&R costs of this project are estimated to be approximately $7,400,000 annually. Approximately $1,700,000 annually would be a Federal responsibility, including OMRR&R for the GIWW floodgates and the HNC lock complex and a portion of cost-shared mitigation OMRR&R. The remaining annual OMRR&R costs of approximately $5,700,000 would be the responsibility of the non-Federal sponsor, including OMRR&R for all project features (except the GIWW floodgates and HNC lock complex), the sponsor funded additional work item, a portion of cost-shared mitigation OMRR&R, and the remainder of mitigation OMRR&R once USACE determines that the mitigation has achieved initial success criteria.

The benefit-to-cost ratio for the post-authorization project is 1.43 based on October 2012 price levels and a 3.75 percent interest rate.

The recommendations contained herein reflect the information available at the time and current Department of the Army policies governing the formulation of individual projects. They do not reflect programming and budgeting priorities inherent in the formulation of National Civil Works construction program nor the perspective of higher review levels within the Executive Branch. Consequently, the recommendations may be modified before they are transmitted to Congress as proposals for implementation funding. However, prior to the transmission to Congress, the state, federal agencies, and other parties will be advised of any modifications and will be afforded the opportunity to comment.

FINAL POST AUTHORIZATION CHANGE REPORT
MORGANZA TO THE GULF OF MEXICO, LOUISIANA

May 2013

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1 Introduction and Background

The following sections describe the project construction authorization, purpose and scope of the Post Authorization Change (PAC) report, project history, design changes, cost sharing and funding, and related projects.

1.1 Project Authorization

The Water Resources Development Act of 2007 (WRDA 2007), Public Law 110-114, authorized construction of the Morganza to the Gulf of Mexico, Louisiana hurricane and storm damage reduction project ("Morganza to the Gulf") at a total cost of $886.7 million. Section 1001 of WRDA 2007 states:

> *"(24) MORGANZA TO THE GULF OF MEXICO, LOUISIANA.—*
> *(A) IN GENERAL.—The project for hurricane and storm damage reduction, Morganza to the Gulf of Mexico, Louisiana: Reports of the Chief of Engineers dated August 23, 2002, and July 22, 2003, at a total cost of $886,700,000, with an estimated Federal cost of $576,355,000 and an estimated non-Federal cost of $310,345,000.*
> *(B) OPERATION AND MAINTENANCE.—The operation, maintenance, repair, rehabilitation, and replacement of the Houma Navigation Canal lock complex and the Gulf Intracoastal Waterway floodgate features of the project described in subparagraph (A) that provide for inland waterway transportation shall be a Federal responsibility in accordance with section 102 of the Water Resources Development Act of 1986 (33 U.S.C. 2212)."*

In accordance with the 2002 and 2003 reports of the Chief of Engineers, the Morganza project is authorized as a feature of the Mississippi River and Tributaries (MR&T). To date, Congress has not appropriated any construction funds. Therefore, the project remains in Preconstruction Engineering and Design (PED) phase.

1.2 Purpose and Scope of the PAC Report

Plans in the 2002 Morganza to the Gulf Feasibility Study were developed well before Hurricane Katrina's devastating impact on the New Orleans hurricane levees in August 2005. Changes in hurricane levee design standards and other changes since project authorization caused the Morganza to the Gulf project to exceed the 20 percent cost increase limit specified in WRDA 1986, Section 902.

There are two types of post-authorization reevaluation studies. "General" reevaluation studies are to affirm, reformulate or modify a plan, or portions of a plan, under current planning criteria. "Limited" reevaluation studies should only require modest resources and documentation. If any part of the reevaluation will be complex, or will require substantial resources, or if the authorized plan will change in any way, a general reevaluation is required. General reevaluation studies frequently are similar to feasibility studies in scope and detail. Given the scale of the Morganza to the Gulf project and the significant design changes since authorization, the PAC report resembles a general reevaluation study, but does not include a complete reformulation of the project.

The PAC primarily focuses on analysis of two levels of risk reduction (pre- and post-Hurricane Katrina "100-year" designs) along the authorized alignment. The PAC report includes discussions on post-Katrina design criteria, project designs and costs, and economic analysis

necessary for plan selection. For a summary of major changes since project authorization, see section 1.5. For example, the term 100-year has been replaced with 1 Percent Annual Exceedance Probability (1% AEP).

The Morganza to the Gulf project authorized for construction in the 2007 WRDA is based on the 2002 Feasibility Report and subsequent 2002 and 2003 Chief of Engineers Reports. The PED phase started two months after publication of the March 2002 Feasibility Report and has continued through the PAC analysis. The PAC report describes all changes to the Morganza project since the 2002 Feasibility Report. These changes have occurred at different points in time over the last 10 years and are based on evolving data sets and methodologies. The following summary highlights sections throughout the PAC report describing project changes and analyses occurring at different periods of time:

- Section 1.3, **Authorized Project Description**, describes the 72-mile authorized levee project as recommended in the 2002 Feasibility Report and the 2002 and 2003 Chief's Reports as authorized by WRDA 2007.

- Section 4, **Initial Reevaluation of the Authorized Plan**, includes a preliminary evaluation of three alternative levee alignment strategies in 2008 (section 4.3), which was a reconnaissance-level analysis for the purpose of determining whether or not there would still be a Federal interest in the project with post-Katrina criteria incorporated and whether a feasibility-level PAC report should be initiated.

- Section 5, **Post Authorization Changes to the Levee Alignment**, describes modifications and extensions to the authorized levee alignment during PED and the PAC analysis. The resulting 98-mile post-authorization levee alignment (see the project map in Attachment 1) is the basis for comparison of two levels of risk reduction in Section 6.

- Section 6, **Evaluation and Comparison of Alternatives**, describes two alternatives that are the primary focus of the feasibility-level PAC analysis. The alternatives follow the same 98-mile levee alignment but at different levels of risk reduction (1% and 3% AEP).

- Section 7, **Description of the 1% AEP Post-Authorization Plan**, describes in more detail the alternative that has the highest net benefits of the two alternatives.

As the result of post-authorization changes, all project-related benefits and environmental impacts must be updated. A Draft Revised Programmatic Environmental Impact Statement (RPEIS) was prepared concurrently with the PAC report to document the environmental and socioeconomic impacts. Given the size and complexity of the Morganza to the Gulf project, most of the Draft RPEIS is at a broad, programmatic level; however, the Draft RPEIS includes a more in-depth analysis for a few features that have borrow sources identified and are closest to being ready for construction. For those features, the Draft RPEIS provides sufficient detail so that no further environmental clearances would be needed upon signing of the Record of Decision. All other features would require supplemental National Environmental Policy Act (NEPA) documents (i.e. environmental impact statements or environmental assessments) before they can be constructed.

The report will ultimately be reviewed by the Assistant Secretary of the Army (Civil Works) and coordinated with the Office of Management and Budget as appropriate for submission to the Congress.

1.3   Authorized Project Description

The primary purpose of the authorized Morganza to the Gulf of Mexico MR&T project, as described in the WRDA 2007 authorization, is *hurricane and storm damage reduction*.

1.3.1   Location

The project/study area is about 60 miles southwest of New Orleans, LA (figure 1-1) and includes most of Terrebonne Parish and a portion of Lafourche Parish. The study area extends south to the saline marshes bordering the Gulf of Mexico and encompasses approximately 1,900 square miles.

The Gulf Intracoastal Waterway (GIWW) and the Houma Navigation Channel (HNC) are major waterways in the area. The GIWW passes through Houma in an east-west direction. The HNC extends due south from Houma to the Gulf of Mexico. Bayou Lafourche runs along the northeastern boundary of the project/study area.

Figure 1-1. General Project Location

The authorized Morganza to the Gulf of Mexico levee alignment primarily follows existing hydrologic barriers, such as natural ridges, roadbeds, and existing local levees. Figure 1-2 shows the authorized Morganza to the Gulf levee alignment. The eastern extent of the authorized levee alignment ties into Larose to Golden Meadow levee south of the GIWW. The western extent of the authorized alignment ties into the Bayou Black ridge north of the GIWW.

1.3.2   Features

The August 2002 Chief's Report recommended a plan to reduce hurricane and storm damages by providing a 1% AEP (referred to as 100-year in the Chief's Report) level of risk reduction including the following features:

> *"...raising or constructing 72 miles of levees to elevations varying from +15.0 feet National Geodetic Vertical Datum (NGVD) to +9.0 feet NGVD. The plan includes constructing nine 56-foot-wide sector gate structures in various waterways, one 125-foot floodgate in the Gulf Intracoastal Waterway (GIWW) near Bayou Lafourche, two 125-foot floodgates in the GIWW near Houma, a 200-foot-wide and 1,200-foot-long lock structure in the Houma Navigation Canal (HNC), and new discharge pipes for six existing pump stations. The plan also includes twelve sets of 6-foot by 6-foot concrete box culverts through the levee to allow tidal ebb and flow. Mitigation features in the plan include the creation of 1,352 acres of marsh habitat and widening 10,600 feet of Minors Canal by 40 feet. Consistent with reducing hurricane and storm damages in an environmentally sustainable manner, the project will be designed and operated to achieve coastal wetland conservation through the improved distribution of freshwater inflows to wetlands wherever feasible. The specific designs and operating plans will be formulated in consultation with the interagency habitat evaluate team."*

The purpose of the earthen levee system is to stop or slow down surge inundation. Floodgates provide storm damage reduction during tropical storms and allow currently navigable waterways

to remain open to navigation during non-storm conditions. The purpose of the HNC lock is to control saltwater intrusion at the Houma water treatment plant while allowing for navigation. Environmental control structures (box culverts) through the levee allow tidal ebb and flow.
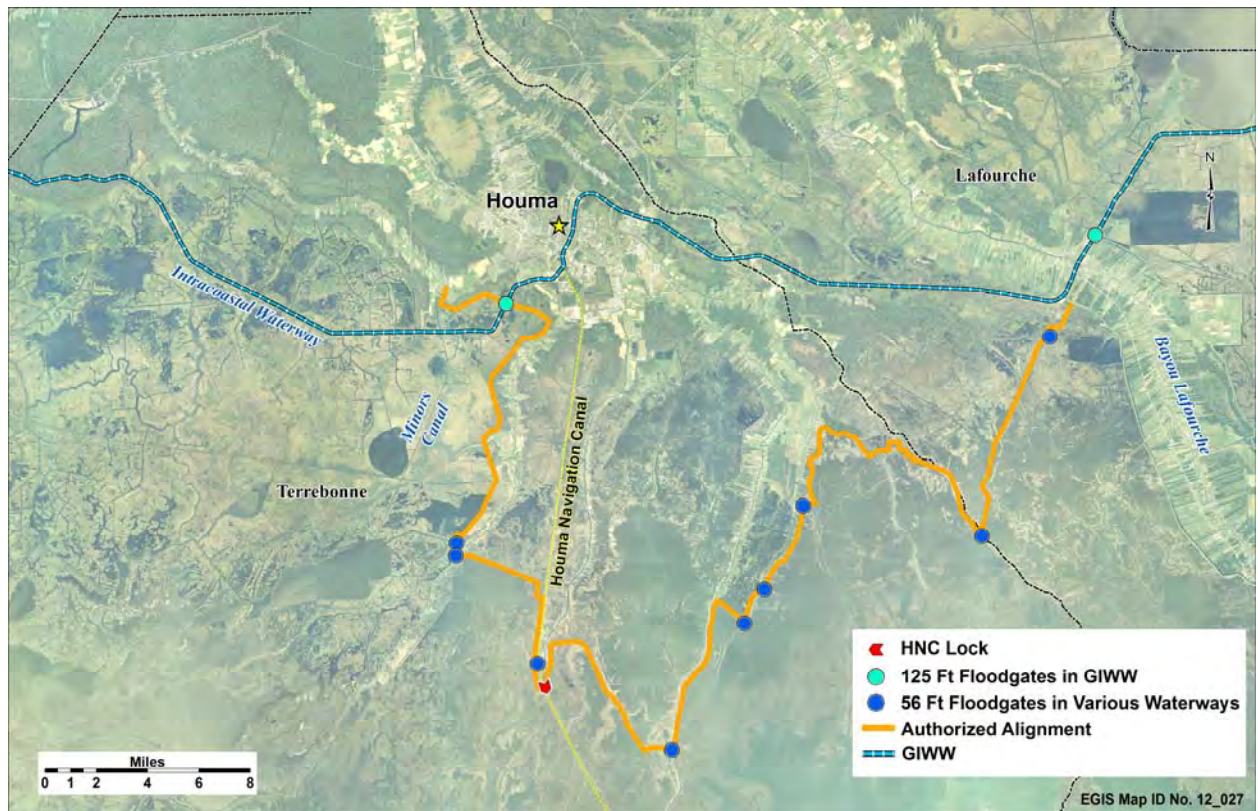
Figure 1-2. Authorized Project Map

1.4   Project History

Table 1-1 provides a timeline of authorizations, studies, and tropical storm events related to the Morganza to the Gulf and HNC lock projects.

Table 1-1. Timeline of Morganza Authorizations and Studies

1985	Hurricane Juan caused extensive flooding in Terrebonne and Lafourche parishes.
1992	Reconnaissance study authorized by resolution adopted April 1992 by the Committee of Public Works and Transportation of the U.S. House of Representatives. In August, Hurricane Andrew caused extensive flooding in Terrebonne and Lafourche parishes.
1994	USACE completed the Morganza to the Gulf reconnaissance report (USACE, 1994).
1995	In the Energy and Water Development Appropriation Act of 1995 (PL 103-316), Congress directed the USACE to consider the interrelationship of studies and projects that impact the coastal area of Louisiana, including the Morganza feasibility study, the Lower Atchafalaya Basin reevaluation study, and several projects being pursued under the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) program, and directed the USACE to consider improvements at and/or within the HNC. The Feasibility Cost Share Agreement was executed in June 1995.
1996	Section 425 of WRDA 96 (PL 104-303) required the USACE to develop a study of the HNC lock as an independent feature of the Morganza to the Gulf project.
1997	USACE completed the HNC lock study, which recommended a 200-ft wide lock in the HNC south of Bayou Grand Caillou and concluded that a lock structure would provide direct and indirect benefits to the environmental (marsh) habitat in the study area (USACE, 1997). The report recommended that the HNC lock continue to be investigated as part of comprehensive Morganza to the Gulf hurricane and storm damage reduction plans and that the detailed design phase of the lock be expedited and proceed concurrently with the feasibility study.
1998	Congress authorized the USACE to initiate detailed design of the multipurpose HNC lock.
2000	The Morganza to the Gulf of Mexico project was conditionally authorized in WRDA 2000 at a cost of $550 million subject to having a favorable Chief of Engineer's report completed by December 2000; the terms of this conditional authorization were not met. The PED phase on the HNC lock complex was initiated in advance of the PED phase for the Morganza to the Gulf of Mexico hurricane and storm damage reduction project. The PED Agreement for the HNC lock was signed in January 2000.
2002	The Morganza to the Gulf feasibility study and PEIS were completed in March 2002 (USACE, 3/2002). The PED Agreement for the overall project was signed in May 2002. In August 2002, the USACE issued a Chief of Engineers report (USACE, 9/2002). In September and October, Tropical Storm Isidore and Hurricane Lili impacted the study area.
2003	In July 2003, the USACE issued a supplemental Chief of Engineers report (USACE, 2003), which made changes to the non-Federal sponsor's in-kind services.
2004	Section 158 of the Energy and Water Development Appropriations Act, 2004 (Public Law 108-137) authorized construction on Reach J-1, which had been previously identified as work-in-kind.
2005	The PED Amendment 1 executed in March 2005 combined the two PED efforts into one and allowed the non-Federal sponsor to advance funds on the combined PED effort. In August and September, Hurricanes Katrina and Rita impacted the study area.
2007	WRDA 2007 authorized the Morganza to the Gulf of Mexico, Louisiana project for hurricane and storm damage reduction at a total cost of $886.7 million.
2008	A recon-level analysis and programmatic cost estimate (ARCADIS, 2008) was completed to determine whether or not there would still be a Federal interest in the project with post-Katrina interim criteria (USACE, 2007) incorporated and whether a feasibility-level PAC report should be initiated. Based on an analysis of four alternatives, the general alignment strategy for the PAC report was determined, but not the final level of risk reduction. Phase I Design for the HNC lock and floodgate was finalized in a 50 percent Design Documentation Report (URS, 2008). In September, Hurricanes Gustav and Ike impacted the study area.
2011	The PED Amendment 2 executed in January 2011 increased the funding ceiling and changed the name of the non-Federal sponsor from Louisiana Department of Transportation and Development (DOTD) to the Louisiana Coastal Protection and Restoration Authority.
2012	Legislation changed the former Office of Coastal Protection and Restoration (OCPR) to the Coastal Protection and Restoration Authority (CPRA) and changed the former Coastal Protection and Restoration Authority (CPRA) to the Coastal Protection and Restoration Authority Board (CPRAB).

1.5   Policy, Procedural, and Design Criteria Changes since Authorization

Several policy, procedural, and design criteria changes have been made since the 2002 Morganza to the Gulf Feasibility Report was completed. Lessons learned from Hurricane Katrina and other recent storms have been incorporated into new design guidelines referred to as the Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines (USACE, 2/2011). These guidelines were developed by members of the U.S. Army Corps of Engineers (USACE), academia, and industry and have been peer reviewed. In accordance with Assistant Secretary of the Army (Civil Works) direction, USACE is applying the HSDRRS guidelines to all hurricane levee system work in the New Orleans District, including the Morganza to the Gulf PAC project. The HSDRRS guidelines provide a comprehensive collection of best practices and were developed to provide redundancy, resiliency, and robustness of the interfaces between structures, materials, and members of the hurricane risk reduction system for the desired level of risk reduction. The HSDRRS guidelines are available at www.mvn.usace.army.mil/ENG/PageA.asp.

The HSDRRS guidelines include some criteria that are more stringent than required for other USACE structures. Changes leading to larger levees and structures, and higher costs, include the following:

- **Increase in Hydraulic Design Elevations –** Storm surge modeling in the 2002 report was based on only 17 tropical storms and did not consider relative sea level rise in the model. New storm surge modeling predicts water levels based on 115 theoretical storms and incorporates the effects of relative sea level rise within the model. In addition, the 1 percent annual chance exceedance water levels in the 2002 report were based on the 50 percent confidence values, which have a 50 percent chance of being under-predicted. The new design guidelines require levees to be designed based on the 90 percent confidence values, which have only a 10 percent chance of being under-predicted. All of these factors result in prediction of higher surge and waves, and wave run up used to set levee elevations.

- **Change from I-Walls to T-Walls –** In the 2002 report, floodwalls could be based on I-wall designs. Under the new guidelines, I-walls are not permitted in most cases and have been replaced with more robust and more expensive T-walls.

- **Increase in Geotechnical Stability Factor of Safety –** The analysis method for global stability changed, leading to a higher factor of safety, resulting in taller and wider levees and cost increases.

- **Addition of Structural Superiority –** All new structures that are difficult to construct because of disruptions to navigation or traffic, large utility crossings, or requiring cofferdams must be designed with a minimum of 2 ft of additional wall height resulting in cost increases.

In addition, new regulations on risk and sea level change have been issued since 2002 as discussed in the following sections.

1.5.1   Changes in Risk Reduction Levels for Authorized Design Elevations

The pre-Katrina levee elevations for the 2002 Morganza to the Gulf project are based on outdated storm and levee design information, resulting in a project that is unable to provide the 1% AEP level of risk reduction. The first step in assessing the authorized project's current level of risk reduction was to convert the 2002 feasibility report stillwater elevations from the National Geodetic Vertical Datum (NGVD) to the current North American Vertical Datum (NAVD88

epoch 2004.65). The next step was to perform a statistical analysis on each levee reach, which determined that a 3% AEP level of risk reduction best describes the 2002 Chief's Report Plan. For the PAC report, the 2002 Chief's Report Plan was subsequently redesigned based on the current projected 3 percent annual chance exceedance surge elevations. The PAC addresses the civil and structural improvements required to achieve the pre-and post-Katrina designs (3% and 1% AEP) as per the HSDRRS guidelines (USACE, 2/2011).

1.5.2   Risk and Uncertainty Guidance

Risk and uncertainty are intrinsic in water resources planning and design. Engineering Regulation (ER) 1105-2-101 provides guidance on the risk evaluation framework to be used in USACE flood damage reduction studies. In a coastal environment, flood risk can be caused by a combination of hurricane surge, waves, wave overtopping of structures, riverine flooding due to rainfall and/or snowmelt, or other sources. Unlike areas located along major rivers such as the Mississippi River or Atchafalaya River, in the

> Risk is composed of two components: (1) the likelihood of being flooded and (2) the consequences.

Morganza to the Gulf project area, the dominant source of flood risk is from hurricane storm surge and waves. Both storm surge and waves are taken into account in proposed levee designs, however, the risk analysis has been simplified as "the probability an area will be flooded by storm surge, resulting in undesirable consequences."

ER 1105-2-101 requires project performance to be described in terms of annual chance or exceedance probability and long-term risk, rather than level-of-protection. A '100-year level-of-protection levee' is often misunderstood to mean a levee that defends against a storm surge that is only expected once every 100 years. In terms of annual chance or exceedance probability, a 100-year levee is designed to withstand a storm surge that has a 1 in 100, or 1 percent, chance of occurring in any given year, not a storm surge that occurs only once every 100 years. For this reason, the terminology '100-year' has been changed to the '1% AEP.'

For investment decisions, risk is typically evaluated in annual probabilities, but when considering the value of levee investments and the assets behind them, one year is short term. Although the 1% AEP levee design has a low probability of being exceeded in any given year, the probability increases the longer the timeframe considered. For the average person's mortgage (30 years), the probability of the 1% AEP flood occurring is 26 percent. For the average person's life, the probability is greater than 50 percent.

Table 1-2. Comparison of AEP and Long-Term Risk

Example Alternative Design Level	Annual Chance of Exceedance	Annual Probability of Exceedance	Long-Term Risk (Probability of Exceedance Over Indicated Time Period)				
			10 Years	30 Years	50 Years	70 Years	100 Years
No Action	1 in 10	0.10 or 10%	65%	96%	99%	100%	100%
3% AEP Design	1 in 35	0.03 or 3%	25%	58%	77%	87%	94%
1% AEP Design	1 in 100	0.01 or 1%	10%	26%	39%	51%	63%
0.2% AEP Design	1 in 500	0.002 or 0.2%	2%	6%	10%	13%	18%

Table 1-2 provides an example of long-term risk for several design levels. The 10% AEP is an example of the no action condition in some locations; the actual AEP varies by location within the study area. Similarly, the 3% AEP and 1% AEP system designs do not result in a consistent AEP in every location. Actual AEP depends on factors unique to every location including elevation, hydrology, presence of local levees, etc.

1.5.3   Sea Level Change Guidance

Recent climate research by the Intergovernmental Panel on Climate Change predicts continued or accelerated global warming for the 21st Century and possibly beyond, which will cause a continued or accelerated rise in global mean sea level. Sea level change can cause a number of impacts in coastal and estuarine zones, including changes in shoreline erosion, inundation or exposure of low-lying coastal areas, changes in storm and flood damages, shifts in extent and distribution of wetlands and other coastal habitats, changes to groundwater levels, and alterations to salinity intrusion into estuaries and groundwater systems. Impacts to coastal and estuarine zones caused by sea level change must be considered in all phases of Civil Works programs. Engineering Circular (EC) 1165-2-212, *Sea-Level Change Considerations for Civil Works Programs,* which replaced EC 1165-2-211, provides guidance for incorporating the direct and indirect physical effects of projected future sea level change in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects. Feasibility level designs, cost estimates, and benefit-cost ratios for the PAC report alternatives are based on an Intermediate Relative Sea Level Rise (RSLR) scenario of 2.4 ft. The effects of the Low (Historic) or High RSLR scenario were also evaluated, consistent with EC 1165-2-212.

1.6   Non-Federal Sponsor

The Louisiana Coastal Protection and Restoration Authority Board (CPRAB) and the Terrebonne Levee and Conservation District (TLCD) intend to be non-Federal co-sponsors for the Morganza to the Gulf project (hereafter referred to as the non-Federal sponsor). In a letter dated 21 December 2012, the non-Federal sponsor expressed commitment and understanding of non-Federal cost share responsibilities for construction and operation and maintenance, repair, replacement and rehabilitation (OMRR&R). Section 1001(24) of WRDA 2007 specifies Federal responsibility for OMRR&R of the HNC lock complex and the GIWW floodgate features that provide for inland waterway transportation in accordance with Section 102 of WRDA 1986, as amended. The non-Federal sponsor is responsible for OMRR&R of all other project features. Additional responsibilities of the non-Federal sponsor are listed in section 8.3 of this report.

1.7   Funding and Cost Share

Federal and non-Federal cost share proportions change as a USACE Civil Works project progresses from the feasibility phase to PED to construction. Expenditures for the completed Morganza to the Gulf feasibility study were $9,320,000, which was cost shared 50 percent Federal and 50 percent non-Federal.

The Energy and Water Appropriations Act of 1998 (PL105-62) included funds to initiate design on the HNC lock feature of the Morganza to the Gulf project, which initiated the Preconstruction Engineering and Design (PED) phase for the HNC lock feature in advance of completing the Feasibility Report (2002) and signing of Chief's report (also 2002). The USACE and non-Federal sponsor signed a Design Agreement for the HNC lock in January 2000, and the non-

Federal sponsor first contributed matching funds for PED in 2000. The first non-Federal contributions to the overall Morganza to the Gulf project were in September 2002.

Approximately $61,650,000 has been allocated for the Morganza to the Gulf PED phase, which includes the PAC report. Most of the PED funds have been spent on engineering design and geotechnical investigations, rather than on the PAC feasibility-level analysis. Per USACE guidance, study costs for the PAC report are being cost shared 75 percent Federal and 25 percent non-Federal. The PAC study is cost shared under a Design Agreement originally executed on 22 May 2002 and amended on 24 March 2005and 11 January 2011. While the Design Agreement provides for 75/25 cost share during design, WRDA 1986 (PL99-662), Section 105(c), stipulates that the non-Federal share of the cost of design is the same percentage as the non-Federal share for construction, which in this case is 35 percent. The design cost is shared per the percentage of construction cost with 25 percent being collected from the non-Federal sponsor during the Design Agreement and the remaining 10 percent collected in the first year after the Project Partnership Agreement (PPA) is executed.

Pending re-authorization, the construction cost share would be 65 percent Federal and 35 percent non-Federal. No Federal funds have been appropriated for construction of the Morganza to the Gulf project.

1.8   Non-Federal Sponsor Work Independent of the Federal Authority

The 2003 Chief's Report and 2004 Energy and Water Development Appropriations Act specified two floodgates and 21.5 of the original 72 miles of levee in the authorized project for in-kind credit. The July 2003 supplemental Chief's Report listed features that *may be designed, constructed, or managed during construction by the sponsor with in-kind services* as follows:

- *A 56-foot-wide floodgate on Bayou Pointe au Chien[1];*
- *A 56-foot-wide floodgate on Bush Canal;*
- *A 14-foot-high and 12-mile-long levee from the Bayou Pointe au Chien floodgate to the Humble Canal floodgate, and the structures therein;*
- *A 14-foot-high and 6.5-mile-long levee from the Bayou Petite Caillou floodgate to the Bush Canal floodgate, and the structures therein; and,*
- *A 14-foot-high and 3-mile-long levee from the Bush Canal floodgate to the Bayou Terrebonne floodgate and the structures therein.*

Reach J-1 was separately authorized for construction in advance of the Morganza to the Gulf project. Section 158 of the 2004 Energy and Water Development Appropriations Act (Public Law 108-137) authorized construction of Reach J, Segment 1 (J-1) at a total cost of $4,000,000.

> *"The Secretary may carry out the Reach J, Segment 1, element of the project for hurricane and storm damage reduction, Morganza to the Gulf of Mexico, Louisiana, in accordance with the report of the Chief of Engineers, dated August 23, 2002, and supplemental report dated July 22, 2003 at a total cost of $4,000,000."*

In order to receive credit for in-kind work, the non-Federal sponsor must sign a PPA with the Department of the Army prior to the work being undertaken by the project sponsor. A PPA has

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1 Pointe au Chien and Pointe aux Chenes are used interchangeably throughout the report. The community of Pointe aux Chenes was Pointe au Chien until community members had the legislature make a name change in the late nineties. Various features mentioned throughout this report such as the bayou, the proposed floodgate, the Wildlife Management Area (WMA) may be spelled differently but all refer to the same area.

not been signed by the sponsor and the Army. The non-Federal sponsor has started work on reaches that were initially proposed to be a part of the Morganza to the Gulf project, at their own expense, acknowledging that there is no signed PPA in place. The sponsor has substantially completed approximately 9 miles of what would amount to first lift levees and a few floodgates located along the proposed Morganza to the Gulf project alignment. Discussion of those features is included in section 2 of this report.

In the absence of an executed PPA, the locally constructed levees do not form an integral part of the Morganza to the Gulf project, and the work performed by the non-Federal sponsor is not eligible for consideration and approval of work-in-kind credit. If the Morganza to the Gulf project is reauthorized, the non-Federal Sponsor will be entitled to LERRRDs credit for the real estate acquired for those local levees only to the extent that the non-Federal sponsor is required to provide authorization for entry to LERRDs necessary for any future Morganza to the Gulf project work that is conducted on the locally constructed levees.

1.9   Related Projects

Some of the most heavily used navigation waterways in Louisiana are located within the project area, including the Federally-maintained HNC and GIWW. Other major water resource projects in the vicinity include the Larose to Golden Meadow hurricane and storm damage reduction project and several ecosystem restoration projects. Table 1-3 summarizes projects most relevant to the Morganza to the Gulf project. Additional details are provided in the RPEIS.

Table 1-3. Projects, Programs and Reports Related to Morganza to the Gulf

	Name	Relationship to Morganza
Navigation Projects	Gulf Intracoastal Waterway (GIWW)	The GIWW runs contiguously through the Morganza to the Gulf project area from Bayou Lafourche at Larose through Houma and on to the Atchafalaya River. The Morganza to the Gulf hurricane levee system crosses the GIWW in two locations requiring floodgates. The GIWW has an authorized depth of 12 ft.
	Houma Navigation Canal (HNC) Deepening Study	The 40-mile HNC connects Houma and the GIWW directly to the Gulf of Mexico. Oil and gas industries in Houma rely heavily upon the channel. In accordance with USACE planning regulations, Morganza to the Gulf plans must assume that the current authorized depth of the canal, -15 ft, will remain as currently authorized in the future; however, there is the possibility that the HNC will be deepened in the future given the preliminary results of an ongoing HNC Deepening Study.
Storm or Flood Damage Reduction Projects	Larose to Golden Meadow Hurricane and Storm Damage Reduction Project	The eastern endpoint of the Morganza to the Gulf authorized levee ties into the existing 48-mile Larose to Golden Meadow ring levee system. The Larose project was originally authorized to provide risk reduction from a Standard Project Hurricane, but is currently undergoing a PAC analysis to ensure that completion of project features are in compliance with post-Katrina guidelines.
	Non-Federal Levees in Terrebonne Parish	In Terrebonne Parish, there are approximately 92 miles of non-Federal levees, along with several pump stations and floodgates, which are operated and maintained by either the Terrebonne Parish Consolidated Government or the Terrebonne Levee and Conservation District (TLCD). The TLCD has recently started building components of the authorized Morganza to the Gulf project, including 9 miles of first-lift levees and interim barge gate structures on several critical bayous.

	Name	Relationship to Morganza
Ecosystem Restoration Programs	Louisiana Coastal Area (LCA) Program	Several LCA projects authorized by WRDA 2007 are located within the Morganza study area, including but not limited to: (1) Convey Atchafalaya River Water to Northern Terrebonne Marshes and Multipurpose Operation of Houma Navigation Lock (2) Modification of Davis Pond Diversion and (3) Land Bridge between Caillou Lake and Gulf of Mexico. By letters dated 20 August 2012 and 16 October 2012, CPRAB has notified USACE that it desires to suspend study and design on these projects. The decision of CPRAB to suspend these projects results in some degree of uncertainty regarding implementation of these projects as part of the authorized Federal LCA. For more details, see section 10.1.7, LCA Program Uncertainties.
	Coastal Impact Assistance Program (CIAP)	The CIAP Falgout Canal Freshwater Enhancement Project is located along the proposed footprint of Morganza Levee Reach E. The goal of the project is to improve efficiency of freshwater flow and to restore salinity to levels that are favorable for fresh and intermediate marsh within the basin. Morganza Reach E includes culverts that would provide similar benefits.
	National Estuary Program	The Barataria-Terrebonne Basin was selected for the National Estuary Program in 1990. The program is cost-shared by the U.S. Environmental Protection Agency (USEPA) and Louisiana's Wetlands Conservation and Restoration Trust Fund.
Multi-Purpose Coastal Plans	Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report	The LACPR technical report includes a coast-wide analysis of a multiple lines of defense approach to "Category 5" risk reduction. The report includes structural measures, such as floodgates, floodwalls, and levees, nonstructural measures, such as elevating homes, and coastal restoration measures, such as diversions and marsh restoration. LACPR technical data was used in the preliminary post-authorization analysis of Morganza alternative alignments.
	Louisiana's 2012 Coastal Master Plan	In the first implementation period (2012-2031) of its 50-year coastal master plan (2012-2061), the State of Louisiana recommends constructing the Morganza to the Gulf hurricane storm surge risk reduction project at the 1% AEP design elevations.

2 Risk-Based Inundation Reduction Benefits Analysis

Most benefits from a hurricane and storm damage reduction project result from the reduction of actual or potential damages due to inundation. Physical inundation reduction damages include damages to residential and commercial structures, losses to the contents in those structures, and damages to privately owned automobiles. Inundation reduction benefits on both existing and future development were considered for project justification.

2.1 Analysis Years

The period of analysis begins with day 1 of the base year and extends generally 50 years for USACE projects. The National Economic Development Procedures Manual, Institute of Water Resources Report 93-R-12 excludes the construction or implementation period from the period of analysis. Early in the PAC study, the 50-year period of analysis was defined as 2035 to 2085. The base year was set based on the assumption that 2035 would be the soonest that the Morganza to the Gulf project constructed to post-Katrina design standards could achieve the 1% AEP level of risk reduction.

Project construction is expected to take place over many years and would result in a closed system providing partial risk reduction prior to achieving the full base year level of risk reduction. It was determined that both alternatives could achieve a closed system by 2024, which provides benefits during construction. Therefore, Hydrologic Engineering Center Flood Damage Analysis (HEC-FDA) models were developed to represent existing conditions (2010), pre-base year conditions (2024), base year conditions (2035), and future year conditions (2085). Outputs from each model are then interpolated to develop annual damage and benefit results.

2.2 Reaches

The PAC report refers to two types of "reaches" as follows:

- **Levee reaches** are segments of the proposed Federal hurricane levee. For the risk analysis, the Morganza levee was divided into major reaches designated by a letter (e.g. J). For detailed design, some of the levee reaches are further sub-divided (e.g. J-1, J-2, J-3).

- **Economic reaches** are sub-areas within the overall study area. Each economic reach has an assigned stage-probability relationship, which is used to calculate damages and benefits. For purposes of planning and analysis, the Morganza study area was originally divided into 276 economic sub-areas or reaches (see Economic Reaches map in Attachment 2). Of the 276 reaches, 264 were determined to be at risk from storm surge. Of the 264 reaches at risk, 234 reaches contain residential and/or non-residential structures; the remainders are open water or wetland areas.

2.3 Datum

The only datum used in the PAC analysis is NAVD88 2004.65. If figures or tables reference older data that was based on a different datum, it is clearly stated.

2.4 Risk and Uncertainty Variables

The HEC-FDA software program was used to perform the risk-based benefits analysis. The HEC-FDA program incorporates risk using statistical Monte Carlo simulations. As required by

ER 1105-2-101, risk and uncertainty is explicitly incorporated in the calculation of net benefits for the following variables:

- **Stage Discharge Function Exceedance Probability with Uncertainty.** The probability functions are assigned an equivalent record length in HEC-FDA. The model uses this parameter to derive confidence limit curves/bands.

- **Geotechnical Failure Analysis.** This feature allows the user to assign uncertainty to the structural integrity of the levee based on geotechnical evaluations of existing and proposed levees. Levees can experience different modes of failure. Slope failure is normally the first in a series of events that happen to cause a breach. Based on geotechnical analyses, the probability of failure due to stability was insignificant compared to the probability of failure due to overtopping and erodability. Therefore overtopping rates and erodability were the predominate factors used to define levee performance. Levee performance is captured in the HEC-FDA model by designating the "top of levee" only or with a supporting fragility curve. The "top of levee" is the 100 percent levee exceedance or failure point.

- **Wave Overtopping Analysis.** This feature allows the user to include uncertainty to account for the impacts of wave overtopping when analyzing levees. An analysis was completed outside of the software and incorporated by entering the computed failure height as the top of levee elevation for each levee in HEC-FDA.

- **Economic Variables.** Quantified uncertainty surrounding the following four key economic variables: structure values, contents-to-structure value ratios, first floor elevations, and depth-damage relationships. The HEC-FDA model uses the uncertainty surrounding these variables to estimate the uncertainty surrounding the stage-damage relationships developed for each study area reach.

The geotechnical failure and wave overtopping analyses were based on assessments of prior levee loadings due to hurricane surge in similar coastal areas, and discussions with national geotechnical engineering experts at the USACE Risk Management Center, as well as other geotechnical engineers in the New Orleans District who have had field experience in developing fragility curves during the rebuilding of levees following Hurricane Katrina. The erosion analysis was based on the extensive levee overtopping laboratory tests performed for the New Orleans District following Hurricane Katrina. Therefore, the overtopping rate that caused erosion on levees with similar composition was well established.

2.5   Simplifying Assumptions

Net benefits are calculated as "without-project damages" minus "with-project damages." Figure 2-1 depicts the actual without-project conditions (existing local levees and no Federal action) and with-project conditions (existing local levees and proposed Federal levees). Additional details on the without-project condition are provided in section 3. Figure 2-2 depicts the simplified approach used to calculate damages in the certified HEC-FDA model.

Use of the certified HEC-FDA model requires a simplified approach for two primary reasons: (1) Model functionality does not allow more than one levee to be assigned to a single economic damage reach, so both sets of levees could not be directly modeled in HEC-FDA. (2) The overtopping equation in the model applies to river levees and is different than the equation used to calculate overtopping for hurricane and storm damage reduction levees. Instead, overtopping can be calculated outside of the program.

Without-Project Condition:

OUTSIDE LOCAL DRAINAGE LEVEES (FLOODSIDE): Damages from direct impact of storm surge, waves, wind, and rainfall.

Existing Local Levees

BEHIND LOCAL LEVEES: Damages from wind and rainfall, local levee overtopping and/or breaches.

With-Project Condition:

OUTSIDE FEDERAL LEVEE (FLOODSIDE): Damages from direct impact of storm surge, waves, wind, and rainfall.

Proposed Federal Levees

BETWEEN FEDERAL AND LOCAL LEVEES: Damages from wind and rainfall, Federal levee overtopping and/or breaches.

Existing Local Levees

BEHIND LOCAL LEVEES: Damages from wind and rainfall, local levee overtopping and/or breaches.

Figure 2-1. Conceptual Depiction of Without- and With-Project Conditions

Modeled Without-Project Condition:

OUTSIDE LOCAL DRAINAGE LEVEES (FLOODSIDE): Damages from direct impact of storm surge are calculated. Damages from waves, wind, and rainfall are not calculated.

Existing Local Levees

BEHIND LOCAL LEVEES: Fragility curves determine when to calculate damages due to surge inundation (e.g. breach). Damages from wave overtopping, wind, and rainfall are not calculated.

Modeled With-Project Condition:

OUTSIDE FEDERAL LEVEE (FLOODSIDE): Damages from direct impact of storm surge are calculated. Damages from waves, wind, and rainfall are not calculated.

Proposed Federal Levees

BEHIND FEDERAL LEVEES: Residual damages due to surge inundation are calculated when the Federal levee design is exceeded (based on simplified fragility curves). Damages from wave overtopping, wind, and rainfall are not calculated. Local levees are not modeled.

Figure 2-2. Conceptual Depiction of Simplified Conditions in the Certified HEC-FDA Model

Given the simplifying assumptions needed to calculate damages in HEC-FDA, the modeled benefits are expected to be less than the actual economic benefits of the Federal levee for the following reasons:

- Not including the local levees in the with-project condition could overstate residual damages for some types of storm events. In the event of a hurricane event large enough to exceed the Federal design, the performance of the local levees would depend on the location of the breach and how much storage was between the Federal levee and the local levee. As modeled, the volume of water overtopping the Federal levees before failure would not be large enough to raise the interior stages high enough to cause damages. In most cases, the overtopping water would not even reach the local levees. If a hurricane event is large enough to cause a breach in the project levees, then it is assumed the local levees would breach as well; therefore, the impact of not including the local levees in the analysis is minor.

- Ignoring wave damages in both the without- and with-project conditions underestimates damages mostly for the without-project condition, which underestimates benefits of the Federal levee. Surge and/or wave overtopping damages in the with-project condition (where the levee design is not exceeded) are expected to be minimal because of the large storage areas behind most of the Federal levees.

- Erosion of the Federal levee is assumed to occur before the surge still water elevation reaches the top of the levee because of waves on top of the still water elevation. Once the HEC-FDA model determines that an exterior stage (floodside) would cause the Federal design to be exceeded, an exterior/interior relationship defines the interior (protected side) stage. If this relationship is not specified, the HEC-FDA model will use the same stages on the interior as on the exterior. Since the exterior stage would be lower once the levee breaches, a simplifying assumption was that the interior stage will equal the without-project stage once the levee breech occurs. In reality, due to insufficient volume of water and duration associated with a typical hurricane storm surge (short duration), filling the protected area to the without-project elevation is extremely unlikely. The consequences of a hurricane event would be based on a flood elevation that is much lower than the estimated surge elevation. This effect has been validated by observations during Hurricane Katrina in the New Orleans Metro area, New Orleans East, and St. Bernard Parish where the flood elevations in the protected areas were notably lower than the storm surge elevations.

Rainfall is not part of the HEC-FDA model because the proposed Federal levee is not expected to impact rainfall damages in the populated areas during or after construction (i.e. rainfall would be the same both with and without the proposed Federal levee). The populated areas are located within forced drainage levee systems, i.e. a system of levees and pumps that provide controlled drainage for an area, allowing the area to remain dry under normal tidal conditions, even if the area is below sea level. Rainfall would continue to be pumped outside of the local forced drainage levee systems and into an area that would be surrounded by the proposed hurricane levee, leaving storage between the Federal and local levee systems.

Impacts of the proposed Federal levee on the interior area were initially modeled using UNET (Unsteady flow through a NETwork of open channels). The purpose of the UNET model was to determine the relative change in water surface elevation between without and with project

conditions. The only appreciable difference in water surface occurs when the levees are overtopped at the less frequent return intervals.

In the areas where natural drainage has been cut off by the proposed levees, environmental structures have been evaluated and sized using an ADH model for general flow patterns; designs were then refined using a HEC-RAS model. These models were used to assess rainfall and interior drainage on the protected side of the proposed hurricane levee system and the structures were sized to prevent induced ponding of any additional water during normal open operating conditions. The environmental structures are to be closed only during a tropical event that affects the local area. The proposed levee system will cause the water to remain in the system for a longer duration than without the proposed levee. The storage time of 7 days has been coordinated with the Habitat Environmental Team and the water in the system is fresh rather than saline. The environmental control structures cannot and are not intended to be operated to reduce rainfall-related damages.

In the areas not cut off by the proposed Federal levee, natural drainage is not affected and so no further studies or drainage improvements were considered. Given the large storage areas behind the Federal levee (e.g. Lake Boudreaux), and environmental control structures throughout the levee alignment, additional pumping capacity for rainfall is not needed.

2.6 Stage-Probability Inputs

For each of the four HEC-FDA models (2010, 2024, 2035, and 2085), stage-probability inputs were based on the following eight points: 0.999, 0.2, 0.1, 0.05, 0.02, 0.01, 0.005, and 0.002. Except for the 2010 existing conditions model, which does not have a with-project condition, each HEC-FDA model contains two sets of stage- probability data in each of the 264 economic reaches—one for the with-project condition and one for the without-project condition.

Stage-probability values were developed using gage data for the more probable events (lower stages) and surge model results for the less probable events (higher stages). Since RSLR was one of the surge model inputs, the surge model results already reflected RSLR effect on future stages; however, RSLR had to be added to the gage data for future years. The 0.999 (1-year), 0.2 (5-year), and 0.1 (10-year) stages were determined from gage data. The 0.02 (50-year), 0.01 (100-year), 0.005 (200-year), and 0.002 (500-year) stages are based on interpretation of surge model results. The 0.05 (25-year) stages were interpolated between the 0.1 (10-year) and 0.02 (50-year) points. If the surge model did not show surge propagated far enough in the basin to produce a surge elevation in an economic reach, an elevation lower than the invert elevation was reported. The minimum elevation is referenced to Morganza to the Gulf project LiDAR (NAVD88).

2.7 Existing Non-Federal Levees in the Without-Project Condition

Fragility curves describe the performance of existing non-Federal levees in the HEC-FDA model. The fragility curve gives the probability of the levee segment reaching its respective failure limit, conditioned on surge water levels. There are two types of non-Federal levees within the Morganza to the Gulf project area. The first type refers to hundreds of miles of forced drainage levees and/or levee-like features, including local levees, roads, natural ridges, drainage ditch spoil banks, and other non-engineered levees that lie interior to the authorized alignment. The second type refers to 9 miles of non-Federal levees that were designed for hurricane surge risk reduction and were constructed along the Morganza to the Gulf project alignment.

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2.7.1   Forced Drainage Levees and Other Levee-Like Features

Existing forced drainage levees and other levee-like features protect against normal tidal fluctuations, but not storm surge flooding from hurricanes. Either the Terrebonne Parish Consolidated Government or the TLCD currently maintains and operates a total of 92 miles of forced drainage levees along with several pump stations and flood control structures. Private landowners have also constructed levee systems to protect their land from frequent flooding; however, private levees are not necessarily subject to regular maintenance. This set of levee features is reflected in the existing condition, and fragility curves were developed for each of the levees and levee-like features that enclose 78 of the 276 storage areas in the Morganza to the Gulf study area.

2.7.2   Non-Federal Levees Constructed as a Base for the Morganza to the Gulf Project

Approximately 9 miles of non-Federal levees are located along the authorized Morganza to the Gulf alignment in reaches J-1, H-2, and H-3 (refer to the project map in Attachment 1 for the location of these reaches). These levees were constructed by the non-Federal sponsor to meet HSDRRS criteria (USACE, 2/2011) in anticipation of Federal project re-authorization but are not part of the Federal project. Of these 9 miles of levee, only Reach J-1 (elevation +8.5 ft NAVD88), authorized separately for construction under Energy and Water Development Appropriations Act of 2004, was in place in 2008 when the project storm surge modeling was initiated and is included as part of the existing condition. There is a local levee fragility curve associated with levee Reach J-1. Construction of the remaining 6 miles, Reaches H-2 and H-3 (elevation +10 ft NAVD88), did not begin until 2009 and therefore could not be reflected in the without-project condition. Since H-2 and H-3 do not create a closed system (remains open to south, west, and north), their construction would not significantly affect the without-project condition, and there is no fragility curve associated with these two levee reaches.

Levees constructed by the non-Federal sponsor along the Morganza to the Gulf alignment are generally at an elevation less than +12 ft NAVD88 in areas where the Federal levee could eventually be over +26 ft NAVD88. The local levees appear to be constructed to HSDRRS standards (USACE, 2/2011) and would likely promote soil consolidation as a foundation to the Federal project. Floodgates put in place by the non-Federal sponsor (steel barge gates) do not meet HSDRRS standards and would eventually be removed and replaced with sector gates that are part of the Federal project.

2.7.3   Development of Fragility Curves for Existing non-Federal Levees

Development of fragility curves for the local levees considered two conditions: (1) global stability without overtopping and (2) overtopping with subsequent erosion. The local levees vary in elevations, compositions, top width, and side slopes and are scattered throughout the study area. Local levee elevations range from approximately +3 to +10 ft and have an average elevation of around +6 ft NAVD88. Levee slopes range from 1:10 to 1:3. There is little or no levee design documentation available, so geometric and engineering material properties of the local levees were determined from aerial photographs, Geographic Information System (GIS) overlays, available geotechnical data, extensive interviews with levee district personnel, and best engineering judgment. Stability analysis was performed on the local levees based on this limited information. The results of the stability analysis indicate that the local levee system's probability of failure due to stability or under seepage was relatively low for still water elevations reaching to the top of the levee. Therefore, the fragility curves were primarily based on erosion from wave overtopping, which is expected to be the dominant failure mode.

To construct the fragility curve for each local levee system, calculations were made to correlate surge elevations with overtopping rates, and failure probabilities were then assigned to those surge elevations. Based on the past loading and performance of these local levees for Hurricanes Gustav (August 2008) and Ike (September 2008), the probabilities of levee failure assigned to 0.1 and 1.0 cfs/ft were established based on the best available engineering judgment. Since the levee would begin to erode at an overtopping rate of 0.1 cfs/ft, the surge still water elevation corresponding to this overtopping rate was assigned a failure probability of 45 percent. At an overtopping rate of 1 cfs/ft, the levee could experience catastrophic failure and was therefore assigned a failure probability of 95 percent. The probabilities of failure assigned to the various overtopping rates are very subjective. Most levee performance criteria contain information on flow rate, embankment geometry, material, and vegetation cover and the engineer tries to relate it to probability. Each project is completely different and dependent on the external conditions and forces acting on the levee as well as the levee integrity.

As an example of how the local levee fragility curves relate to actual performance, one reach of local levee has an average levee height of approximately 7 ft. An overtopping computation resulted in a 1.0 to 1.2 cfs/ft overtopping rate at elevation 6.5 ft. The probability of failure in the fragility curve at that elevation is 95 percent, stating that failure is likely or very likely. This levee actually failed during Hurricane Ike with a storm surge height of 6.2 ft. In contrast, this levee reach experienced a storm surge height of 4.9 ft during Hurricane Gustav and did not fail. Based on the local levee fragility curve developed for this reach, at elevation 5.3 ft, the levee has a less than 10 percent chance of failure.

Figures 2-3 and 2-4 are examples of breaches at two locations. There were no levee breaches during Hurricane Gustav. Hurricane Ike produced some severe scour areas, two levee breaches, and a fronting protection failure. Of those damages, only the control structure breaches along the Louisiana Department of Wildlife and Fisheries marsh management levee (Morganza Reach J-2) and a heavy scour area on the west side of Bayou Dularge (Morganza Reach B) fall along the Morganza authorized alignment. All other breaches and scour areas were along local levees that are not planned for incorporation into the Morganza alignment. Morganza Reach J-1, constructed by the non-Federal sponsor, was overtopped but remained fully intact throughout this storm event. There were no initial damages to report on Reach J-1.

2.8   Proposed Federal Levees in the With-Project Condition

For each major Federal levee reach, a design exceedance elevation is entered in HEC-FDA along with an exterior/interior stage relationship. The design exceedance point is below the actual top of levee based on wave heights, overtopping, and erodability calculations. The design exceedance elevation is set at the still water elevation that equates to 2 cfs/ft wave overtopping. The 2 cfs/ft point was selected based on data from Colorado State University lab testing on grass-covered levees that showed erosion starting at 2 cfs/ft. This approach is conservative in that benefits and project performance are underestimated; the proposed Federal levee is designed to withstand higher surge elevations and some erosion before breaching would occur. Using the actual top of levee, however, would have overestimated benefits and project performance to some extent because there would be more overtopping using the actual top of levee elevations.

Figure 2-3. Breach at West Corner of "Stair-Step" Portion of Montegut Non-Federal Levee

Figure 2-4. Breach at Lashbrook Pump Station on Ward 7 Levee along Boudreaux Canal

Scour has undermined timber fronting protection, allowing water to seep under pump station.

3 Without-Project Conditions (No Action)

Existing conditions and modeling methods have changed in the approximately 15 years since the original feasibility analysis was initiated. Some of the major changes include storm surge and wave height predictions, sea level predictions, and updates to the residential and nonresidential structure inventory. Further details on without-project conditions can be found in the RPEIS and Economic and Engineering Appendices.

3.1 Topography and Land Use

The study area lies at the southern end of the Terrebonne Basin and contains a complex of habitat types, including natural levees, lakes, swamps, marshes, and bayous formed from sediments of abandoned Mississippi River deltas. The majority of the study area is undeveloped wetlands and open water. Elevations in the study area range from less than 1 ft near the Gulf of Mexico to approximately 10 ft near Houma, the largest city in the area. The elevation along the bayou ridges is 4 to 5 ft.

The study area is situated within the Barataria-Terrebonne estuary. The marsh habitat in the estuary transitions from fresh marsh in the more northerly portions to intermediate and brackish marshes, and to saline marsh near the coast. The remaining wetlands consist mainly of woody wetlands (primarily bald cypress/tupelo swamps and bottomland hardwood forest). Wetland areas within the hurricane and storm damage reduction system are deteriorating due to subsidence, saltwater intrusion, and storm surge flooding.

Only about one tenth of the study area has been developed. Communities located within the study area include the city of Houma, the towns of Chauvin, Dulac, and Montegut in southern Terrebonne Parish, the towns of Donner and Gibson in western Terrebonne Parish, and the towns of Gray and Schriever in northern Terrebonne Parish. Also included are the towns of Raceland, Lockport, and Pointe aux Chenes in Lafourche Parish and the portion of the city of Thibodaux south of Bayou Lafourche. Agricultural land is primarily used for pasture/hay and sugar cane.

Nearly all residential development occurs along one of the major bayou ridges, which generally do not flood except during extended and/or strong tropical storm events. Future development is expected to remain within forced drainage systems and along the ridges due to the increased costs associated with constructing homes outside of those areas. Very few residential structures are located in marsh; rather, hunting and fishing camps built on pilings are typically found in those areas.

3.2 Coastal Vegetation and Wetland Loss

Wetland loss in coastal Louisiana continues to be a major problem affecting both the ecosystem and storm surge risk reduction. The study area encompasses over half a million acres of vegetated coastal wetlands. The Louisiana Natural Heritage Program, which describes rare, unique, and imperiled plant species and vegetative communities occurring in Louisiana, lists 45 plant species or natural communities as occurring in Terrebonne and Lafourche Parishes. Storm surge exerts widespread stress upon vegetation through the introduction of higher salinity concentrations than are normally present within the study area and by direct erosion of marsh plants and soils. As area marshes convert to open water, vital fish and wildlife habitat, economic benefits, and hurricane and storm damage reduction benefits are lost. While wetlands cannot prevent the devastating effects of major hurricanes such as the recent Hurricanes Katrina, Rita,

and Gustav, wetlands can potentially reduce the storm surges associated with smaller tropical storm events.

As shown in figure 3-1, the land trend between 1985 and 2008 was a loss of approximately 2,600 acres per year, which equates to almost 60,000 acres lost over that period. Projecting that loss rate over the next 75 years would be almost 200,000 additional acres lost, which could be even greater with higher rates of sea level rise and/or subsidence.
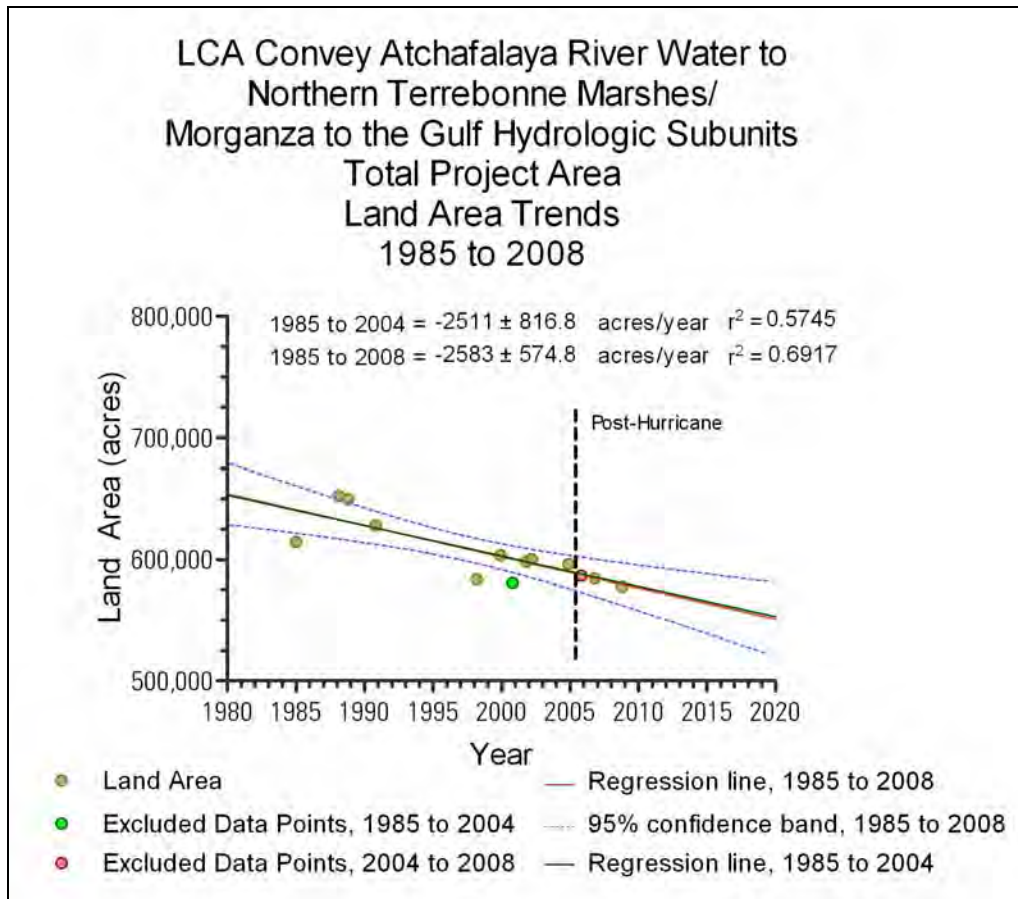
Figure 3-1. Land Area Trends from 1985 to 2008 Projected through 2020

Significant resources may be affected by coastal vegetation and wetland loss including, but not limited to, prime and unique farmland; aquatic resources; fisheries; essential fish habitat; wildlife; threatened and endangered species; water quality; recreation; aesthetics; and cultural resources. For a complete discussion on significant resources, refer to the RPEIS.

3.3   Hurricane Surge and Waves

Since the 2002 feasibility report was completed, Hurricanes Katrina and Rita (2005) and Hurricanes Gustav and Ike (2008) have impacted the study area. Flooding as a result of Hurricane Ike significantly impacted the town of Chauvin, which is located southeast of Houma. The town remained under water for nearly a week after the hurricane, as shown in figure 3-2.

Figure 3-2. Flooding Impacts of Hurricane Ike in Chauvin, Terrebonne Parish

Source: Louisiana Recovery Authority (http://www.lra.louisiana.gov)

Without-project surge and wave modeling was completed in 2008 using 2007 conditions to represent existing/no action conditions. A suite of theoretical hurricanes were modeled within an integrated modeling system to generate storm surge and wave simulations. The ADCIRC Coastal Circulation and Storm Surge Model is coupled with offshore and nearshore wave models (WAM and STWAVE). The ADCIRC surge model used for the Morganza PAC analysis is based on the model that was validated for Interagency Performance Evaluation Task Force (IPET) with Hurricane Katrina data and subsequently validated with data from Hurricane Rita for this and other USACE coastal surge studies. Prior to use of the surge model, modifications to the bathymetry, topography, and levee heights specific to the Morganza study area were made.

The without-project condition was modeled with all existing non-Federal levees and other "levee-like" features in place except for the 6 miles of first lift levees along reaches H-2 and H-3, which were not constructed until after modeling was complete. The without-project conditions do not need to be re-modeled with H-2 and H-3 in place because they do not result in a closed system and would not significantly reduce damages. Storm surge could flank the levee system and/or proceed up uncontrolled bayous and canals, thereby flooding the project area. The constructed first-lift levee reaches are expected to settle within the first couple of years, reducing their ability to protect against a tropical storm surge. If overtopping occurs during a storm event, scour would most likely erode the unprotected side, causing the levees to fail.

Future relative sea level rise was incorporated into surge and wave modeling. Surge propagation over wide, shallow wetland areas is highly sensitive to sea level rise. Increased water depths resulting from sea level rise and wetland degradation can increase surge propagation speed and allow greater inundation. If the mean water level increases in the future, low-lying areas could experience higher surges on average. Waves also generally increased significantly for all sea level rise cases. Wave height increases are significant, but less dramatic than the surge increases.

Three Relative Sea Level Rise (RSLR) scenarios, based on Low (historic), Intermediate, and High sea level rise rates, are part of the risk assessment for the Morganza to the Gulf project. The local historic RSLR trend was determined based on the Leeville, Louisiana gage. Subsidence is included in the gage readings. Applying a linear trend to the 43 years of available data gives a yearly relative sea level increase of approximately 0.3 in/yr. EC 1165-2-212, *Sea-level Change Considerations for Civil Works Programs*, provides the equations for calculation of the RSLR values for the three scenarios. Table 3-1 shows the calculated values for 2010 to 2035 (existing conditions to base year), and for 2010 to 2085 (existing conditions to future year).

Table 3-1. Total Relative Sea Level Rise by 2035 and 2085 by Scenario

Scenario	Total RSLR (ft)	
	Over 25 years: 2010 to 2035	Over 75 years: 2010 to 2085
Low (Historic Rate)	0.6	1.7
Intermediate (Rate 1)	0.7	2.4
High (Rate 2)	1.2	4.8

Since much of the surge and wave modeling for the Morganza to the Gulf PAC was completed prior to issuance of EC 1165-2-212, the modeled RSLR values vary slightly from the EC-calculated values. Simulations were run for approximately 1, 3, and 5 ft of RSLR, and the results were then interpolated for the three RSLR scenarios. Modeling results demonstrated that surge does not increase linearly with sea level rise. Present day surges of 7 to 10 ft could increase by as much as 3 to 7 ft more than the sea level rise increase in the future.

After running storm surge models, results were statistically analyzed to estimate stage-probabilities for the intermediate RSLR scenario (table 3-2).

Table 3-2. Representative Without-Project Surge Still Water Elevations (No Waves)

Frequency	Existing Condition (2010)	Base Condition (2035) Intermediate RSLR	Future Condition (2085) Intermediate RSLR
0.04 (25-yr)	6 to 8 ft	7 to 9 ft	9 to 10 ft
0.02 (50-yr)	9 to 10 ft	10 to 11 ft	11 to 13 ft
0.01 (100-yr)	11 to 12 ft	12 to 13 ft	13 to 15 ft
0.005 (200-yr)	13 to 14 ft	14 to 15 ft	15 to 17 ft
0.002 (500-yr)	15 to 16 ft	16 to 17 ft	16 to 19 ft

Note: Values are rounded to the nearest foot. More detailed information can be found in the Engineering Appendix. Values are for the Intermediate RSLR scenario. See section 6.9 for a discussion on the Low and High RSLR scenarios.

See Attachment 3 for a set of maps corresponding to the 50-, 100-, and 500-yr surge inundation depths for 2010, 2035, and 2085 without-project conditions.

3.4   Local Hydrodynamics and Salinity

The major rivers and their tributaries affecting the study area are the Mississippi River to the east and the Atchafalaya River to the west. Rainfall typically flows away from the major streams and collects between the ridges to form bayous, such as Bayou Lafourche, that flow south toward the Gulf of Mexico. Anthropogenic changes within the study area, such as construction of canals, pipelines, roads, railroads, navigation channels, and levees, have altered the natural flow patterns. Major waterways crossing the study area include the east-west GIWW and the north-south HNC. The HNC has been implicated in higher salinity in the Houma area. The study area also has numerous forced drainage systems that remove excess stormwater by a combination of earthen levees, drainage canals, and pumps.

To determine hydrodynamic and salinity effects, a TABS-MDS numerical model has been created for the Morganza area. The primary freshwater inflows to the system are the Atchafalaya River and the Wax Lake Outlet. Additional smaller inflows (Bayou Boeuf, GIWW at Larose, Bayou Lafourche, and three drainage channels) were also included in the model geometry. Without the project in place, storm surges push seawater from the Gulf of Mexico and increases salinity in the study area. The salinity gradient in the basin varies from 0 ppt in the upper basin to 27 ppt at certain times of the year in the Gulf of Mexico as shown in figure 3-3. Factors affecting salinity in the project area include the Atchafalaya River discharge via the GIWW, Gulf of Mexico salinity and circulation, wind, rainfall/evaporation, tidal exchange, vertical mixing in deeper channels, and bathymetry.
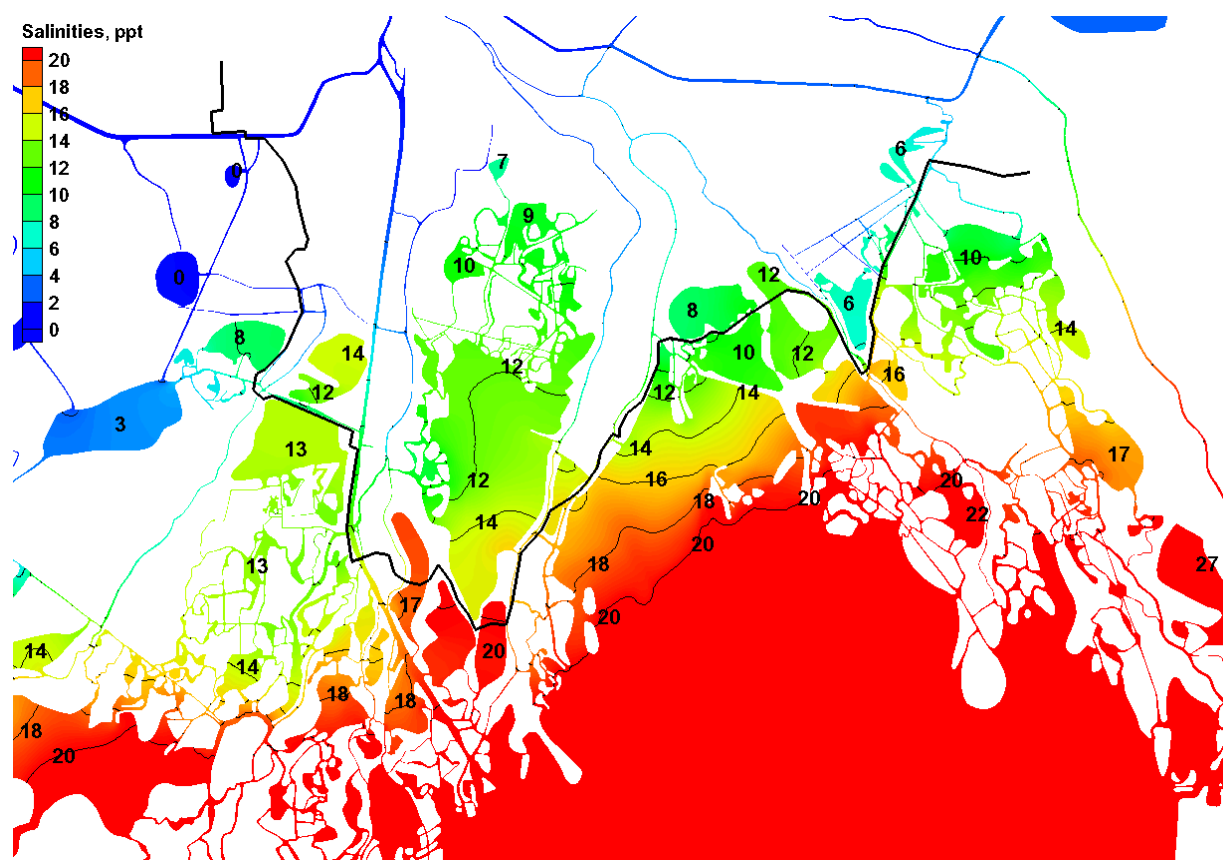
Figure 3-3. Base Average Salinity for October to December 2004 Without-Project Conditions

3.5 Population and Storm Surge Damages

The structure inventory for the original Morganza feasibility study (2002) was collected in 1997-98. To update the inventory for the PAC report, field surveys were collected in 2009. The location of inventoried residential and non-residential structures is shown in figure 3-4. As shown in table 3-3, the number of structures in the study area has more than doubled since the original inventory was collected in 1997/1998 because of population growth and an expansion of the flood risk area.

Since Morganza PAC surge modeling results were not yet available when the inventory was collected, potential damage areas were identified using the largest surge extent developed as part of the post-Katrina LACPR evaluation. In addition to areas outside of the storm surge extent, the area north of Bayou Lafourche between Lockport and Larose was not inventoried because it was originally part of the Donaldsonville to the Gulf study area and was not incorporated into the Morganza to the Gulf study area until after the inventory was collected.

Based on the information collected during the field surveys, a depreciated replacement cost for residential and non-residential structures was calculated. The value of the land was not included in the analysis. A small number of industrial structures were also inventoried using interview forms approved by the Office of Management and Budget.
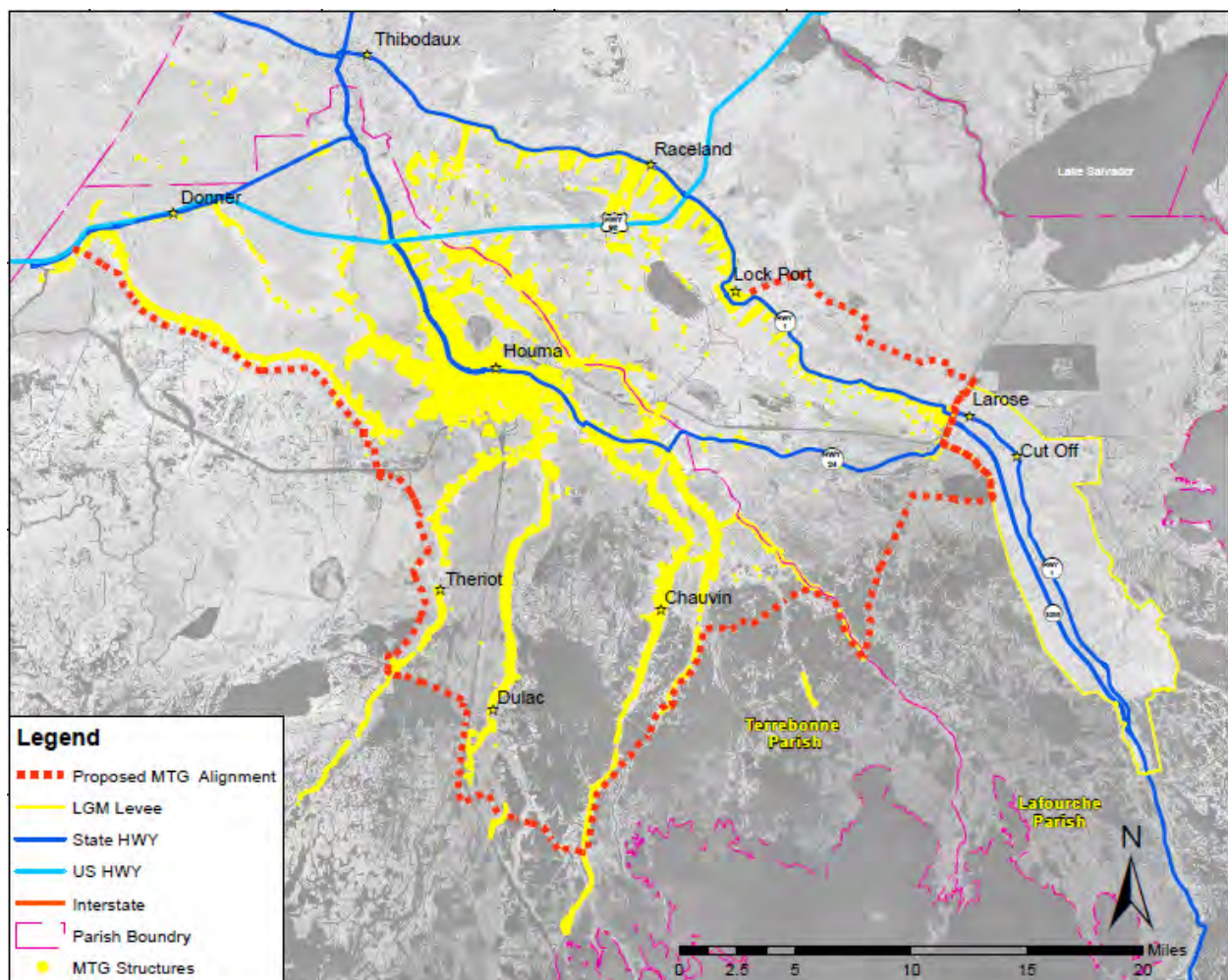
Figure 3-4. Location of Structures in the Morganza to the Gulf Study Area

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Table 3-3. Number of Structures Inventoried in 1998 and 2009.

Structure Type	Number Inventoried	
	Updated PAC Survey (2009)	**Original Survey (1997/98)**
One-Story Slab	21,694	10,146
One-Story Pier	12,723	9,891
Two-Story Slab	1,656	2,399
Two-Story Pier	615	401
Mobile Home	9,859	1,753
Multi-occupancy	See note*	178
Residential Subtotal	**46,547**	**24,768**
Nonresidential Subtotal**	**6,263**	**880**
Total Structures	**52,810**	**25,648**

*In the 2002 feasibility report, multiple-occupancy/multi-family structures were grouped under residential. Multiple-occupancy/multi-family structures are currently grouped under non-residential and there were 309 inventoried in 2009.
**In the 2002 feasibility report, the four nonresidential categories were (1) Agriculture, Mining, Construction, Manufacturing, and Transportation/Communications/Utility sectors (2) Wholesale and Retail sectors (3) Finance/Insurance/Real Estate and Services sectors and (4) Public sector. In 2009, nonresidential structures were subdivided into the following eight categories: Eatery, Professional, Public, Repair, Retail, Warehouse, Grocery, or Multi-Occupancy.

The increase in the number of households that occurred in the two parishes between 1997 and 2009 was commensurate with the population growth experienced by the entire Gulf Coast region during the same period. In 2010, there were approximately 28,800 people residing in the inventoried structures in Lafourche Parish and approximately 104,900 people in Terrebonne Parish for a total of 133,700 residents at risk of storm surge inundation. The population of Lafourche Parish is projected to total approximately 29,300 in 2035 and about 31,200 in 2085. In Terrebonne Parish, the population in this area is expected to total 113,200 in 2035 and 133,800 in 2085.

Historically, hurricanes have not resulted in outmigration from the area (figure 3-5). Despite having experienced numerous flood events, residents have shown an unwillingness to leave the study area, in large part due to economic and cultural ties to the land. The population decline in the mid- to late-1980s was employment-driven (i.e. lack of employment due an oil bust), rather than as the result of Hurricane Juan.

Data from the 2000 Census show that approximately 65 percent of residents in the Lafourche and Terrebonne Parishes lived in the same housing unit as they had in 1995. This percentage ranged from a high of 81 percent in Dulac (southern portion of the study area) to a low of 54 percent in Thibodaux (northern portion of the study area). In comparison, the national percentage of the population residing in the same house in 2000 as in 1995 was 54 percent.
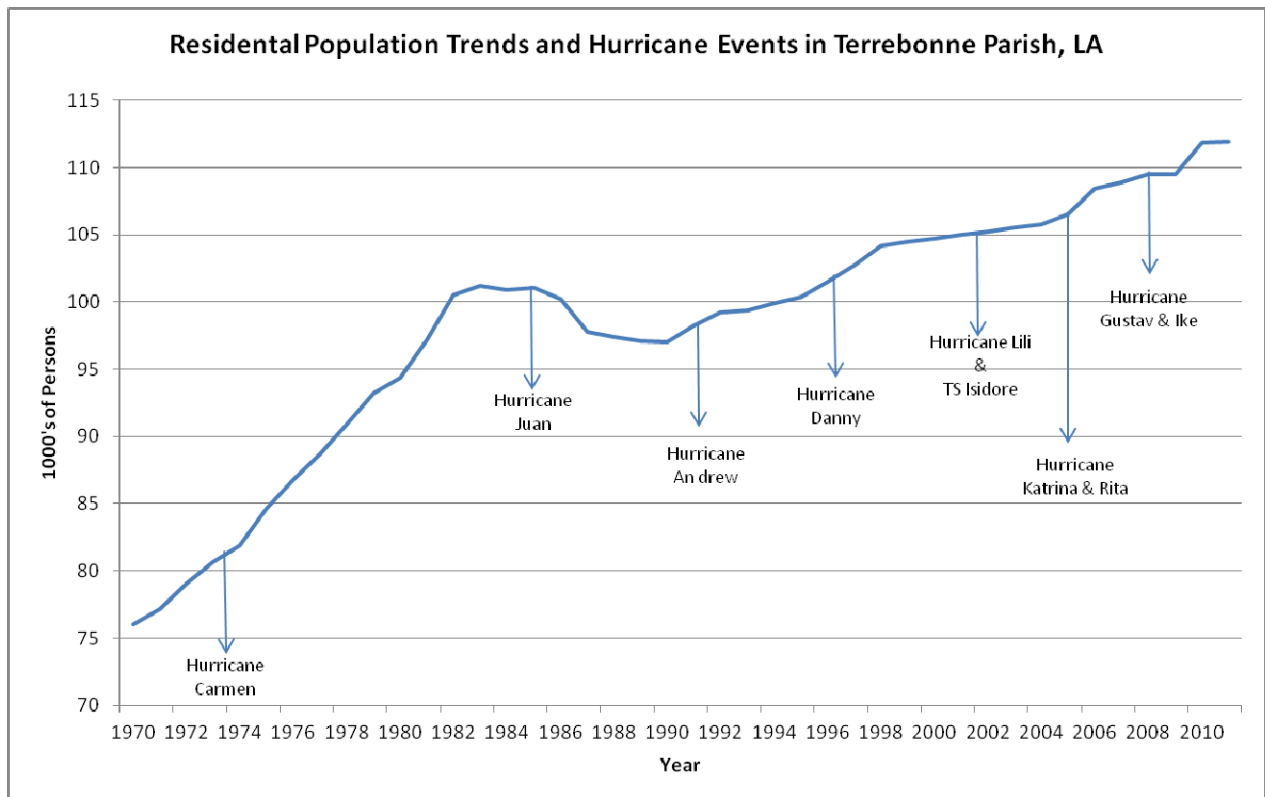
Figure 3-5. Terrebonne Parish Population Trends with Hurricane Events (Source: US Census Data)

According to local officials, residents in low-lying communities began relocating to areas in the northern parts of the study area after Hurricanes Katrina and Rita in 2005. Reasons for this intra-parish shift was a combination of weariness on the part of residents of having to deal with repeat flooding and the more stringent requirements to obtain permits for rebuilding after homes were damaged. In order to rebuild, residents had to incur the cost of building to higher elevations. The ability to secure insurance at a reasonable price was also cited as a reason for the exodus.

The rate of retreat from the southern communities slowed around 2008 after Hurricane Ike impacted the area due to federal assistance, as well as the construction of local levees, which reduced damages to the area. In addition, the two parishes have also implemented elevation programs designed to raise the structures in flood-prone areas. Local officials also stated that residents prefer to remain due to the culture of the residents and the economy of the area. The economy of Terrebonne Parish is closely tied to its abundant natural resources, and many of the residents in the small communities outside of Houma are shrimpers, oystermen, crabbers, fishermen, and trappers. In Lafourche Parish, the economy is strongly tied to the production and distribution of natural gas and oil, commercial fishing, and sugar cane.

The HEC-FDA model used to calculate damages implicitly assumes that all damaged assets are restored to their prior market value completely and within a year after a flood event. Historical data show that major flooding events have not resulted in significant outmigration from the study area, and the post-flood response of property owners is consistent with the HEC-FDA assumption of complete and immediate repair of damaged property. However, the manner in which property owners have responded in the past may or may not be representative of how they will respond in the future to more repetitive and damaging flood events. Even if property owners

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have been willing to repair flood-damaged property at existing elevations in the past, under future without-project conditions, increased flood risks due to expected relative sea level rise and an increase in the frequency of flooding may result in different post-flood responses over the period of analysis. Property owners could opt to have their structures raised in place, floodproof and/or retrofit their structures, relocate within the floodplain, or permanently evacuate from the study area. The course of action selected by an individual property owner following repetitive flood losses depends upon many factors, including the degree of aversion to future anticipated flood risk by that property owner and Federal, State, and local floodplain regulations.

The initial without-project expected annual damages computed by the HEC-FDA model did not consider the behavior of property owners whose structures incur repetitive flood losses. A breakdown of expected annual damages revealed that there were a significant number of structures with damage exposure from relatively frequent events. Approximately 7,500 residential and non-residential structures would incur flood damages from a 0.1 ACE (10-year) storm event in the year 2035, and approximately 2,000 residential structures would incur damages greater than or equal to 50 percent of the structural value. Given the number of structures at risk from frequent flooding, the magnitude of these damages, and the increased frequency which residential and non-residential structures would be exposed to flooding, adjustments to the implicit assumptions of the HEC-FDA model were deemed necessary. Thus, adjustments were made to the 2024, 2035, and 2085 structure inventories as described in the Economic Appendix.

The preliminary unadjusted expected annual without-project damages ranged from $515 million in 2010 to $1.46 billion in 2085 primarily as the result of increasing stage-probabilities due to relative sea level rise. The final expected annual without-project damages for the adjusted inventories range from $486 million to $1.16 billion in 2085.

4 Initial Reevaluation of the Authorized Plan

In the evolution of the Morganza to the Gulf project, various alternative plans have been developed and evaluated with the goal of maximizing hurricane and storm damage reduction benefits while minimizing costs and adverse impacts to the environment, local interests, navigation, and industry. Engineering, environmental, economic, sociological, and institutional factors have been key considerations in the formulation and evaluation of alternatives. Measures and alternatives that were either not feasible, unacceptable, or did not meet the planning objectives were eliminated from consideration as final alternatives.

This section describes the limited re-evaluation of post-authorization alternatives. Prior to initiation of the PAC analysis in 2008, a recon-level analysis of four alternatives was conducted to determine whether the Morganza to the Gulf project could still provide positive net benefits. The 2008 analysis evaluated the authorized alignment at two levels or risk reduction, and two other levee alignments to see if they would produce higher net benefits than the authorized alignment. The authorized alignment was determined to have the potential to produce the highest net benefits.

4.1   Problems, Opportunities, Goals, Objectives, and Constraints

No changes in problems, opportunities, goals, objectives, and constraints have occurred since the 2002 feasibility report analysis. The primary problem continues to be the flood risk associated with storm surge and waves, which is increasing due to wetland loss, sea level rise, and subsidence. Tropical storms cause widespread flooding of residential and commercial properties resulting in extensive damage and economic losses. Despite local efforts to maintain a system of forced drainage levees, pump stations, and flood control structures, an adequate storm surge risk reduction system is not currently in place. Although the USACE cannot affect the root causes of storm surge and wetland loss, such as hurricanes, sea level rise, and subsidence, it has the opportunity to address some of the problems related to storm surge flooding. The overarching goal is to reduce the risk to people and property in the vicinity of Houma, Louisiana. All project benefits are related to hurricane and storm damage risk reduction. No flood damage reduction, navigation, or ecosystem restoration benefits are quantified for this project.

4.2   Nonstructural Alternatives

Nonstructural measures complement structural measures and are an essential part of a comprehensive hurricane and storm damage reduction program (NAS 2009). Nonstructural measures reduce risk without significantly altering the nature or extent of flooding. Examples include flood proofing, relocation of structures, elevation of structures, flood warning and preparedness systems, evacuation plans, public education, zoning, and flood insurance. The WRDA of 1974 (PL 93-251) requires that Federal agencies consider nonstructural measures to reduce or prevent flood damage. Nonstructural measures are the responsibility of not just the Federal government, but also state and local governments and private citizens.

Nonstructural alternatives for direct comparison to the authorized Morganza to the Gulf levee project were not formulated as part of this PAC report. The 2002 Morganza to the Gulf feasibility study showed that hurricane and storm damage problems cannot be solved entirely through nonstructural means for the study area. Large-scale relocation is problematic both socially and economically because homes and businesses would have to be moved considerable distances north to the Houma or Thibodaux areas to remove them from the threat of coastal flooding from the 1% AEP (100-year) storm surge event. The 2002 feasibility report did explore

the possibility that some structures along the bayous might lend themselves to nonstructural solutions. For 6,000 residential structures along the bayou ridges south of Houma (including approximately 5,000 structures that are located within the authorized alignment), alternatives included either raising them above or relocating them out of the 1% AEP floodplain. Neither nonstructural option for these areas was economically justified so they were eliminated from further consideration.

Five communities (approximately 1,000 structures) residing outside the authorized Morganza to the Gulf alignment are described below (see figure 4-1):

1. Isle de Jean Charles – An isolated community of State-recognized Biloxi-Chitimacha tribe members that has lost a significant percentage of its population in the past 10 years. There are currently about 25 families using the "Island" as their primary residence. The majority of the remaining structures are weekend camps. Most of the residential structures are already elevated. The 2002 feasibility report determined that a relocation plan was economically justified; however, it was not recommended because the proposed plan was not supported by the Isle de Jean Charles community. Instead, the TLCD constructed an earthen levee to approximately elevation 6 ft. In addition, the only road to the island was raised to provide a better evacuation route.

2. Lower Chauvin/Cocodrie – The most southern part of LA Hwy 56 (4.5 miles) is outside the Morganza to the Gulf alignment. This area include a University Marine Research Facility (LUNCOM), several fishing marinas, commercial fishing docks and facilities, oil and gas facilities (docks) and many fishing camps. There are very few local residents who still consider this area their primary residence.

3. The "Four Point" area in lower Bayou Grand Caillou/Dulac – The only part of Grand Caillou that is located outside of the Morganza to the Gulf alignment consists of fishing camps and a small marina at the end of Four Point Road.

4. Lower Bayou Dularge – This portion of Bayou Dularge is the most populated area not included in the Morganza to the Gulf alignment. This area has many local commercial fishermen along with docks and one marina. The TLCD intends to protect the entire Dularge community as much as possible and currently maintains about 15 miles of local levees in lower Dularge with a 56-ft wide barge floodgate at the end of Bayou Dularge.

5. Gibson - Gibson is an unincorporated community in Terrebonne Parish at the termination of the western extent of the Morganza PAC alignment. Gibson was not identified as a community at risk in the original feasibility study because modeling at that time did not show storm surge reaching that area.

For the PAC report, a preliminary nonstructural buyout plan has been developed for high risk areas outside the proposed levee system as shown in figure 4-1 and described in section 6.5.1 of this document. Socioeconomic considerations associated with the potential buyouts are described in RPEIS section 5.2.13, and Environmental Justice is discussed in RPEIS section 6.14.8 and RPEIS Appendix J.
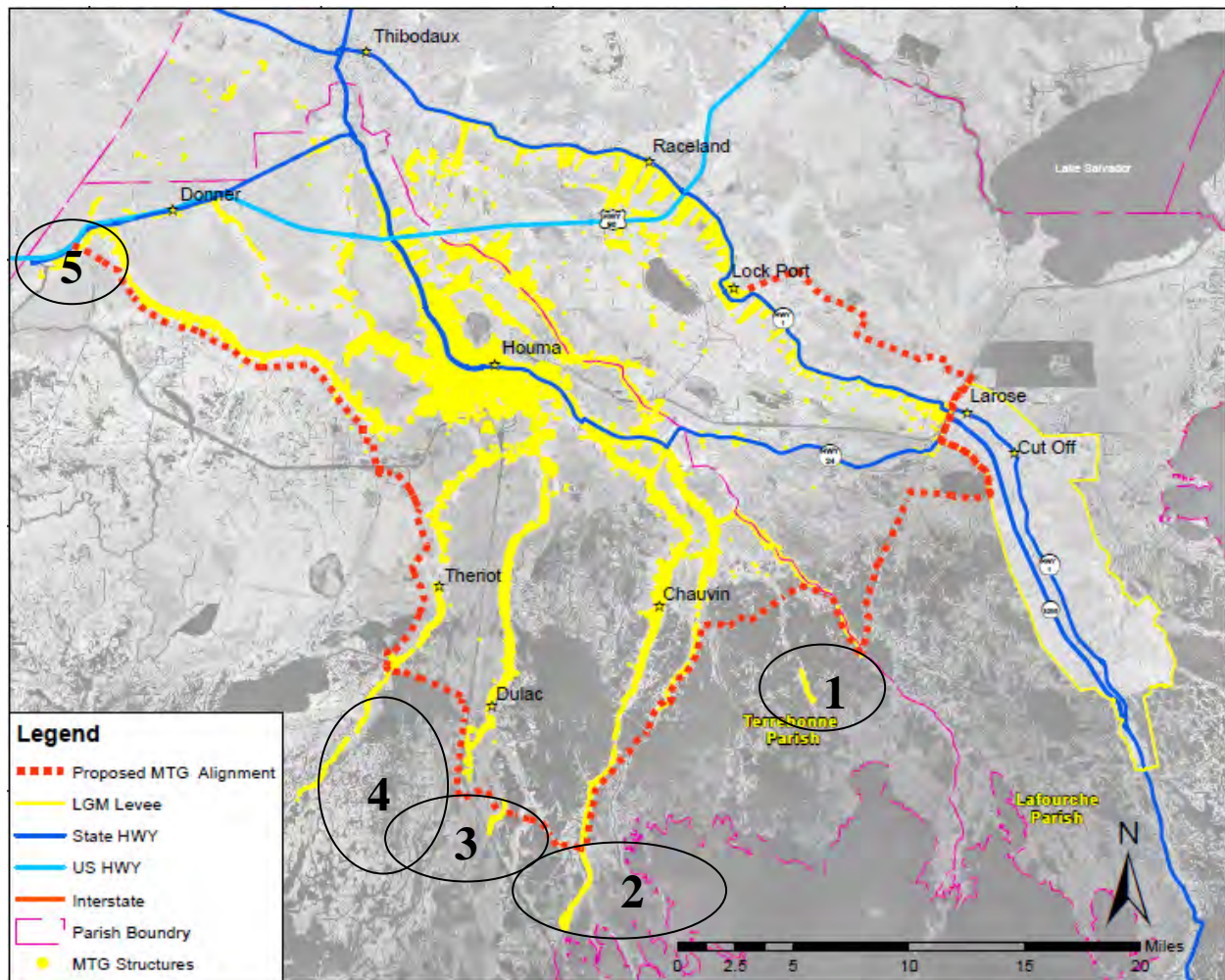
Figure 4-1. Location of Structures Outside the Morganza to the Gulf Levee System

4.3 Preliminary Evaluation of Alternative Levee Alignments

Following the 2007 WRDA authorization, a set of four Morganza to the Gulf alternatives were evaluated based on post-Katrina/Rita interim design guidelines using LACPR data and methods. The purpose of the 2008 analysis was to determine whether or not there was still a Federal interest in the project and whether a feasibility-level PAC study should be initiated. The four alternatives were developed in 2008 in coordination with the CPRA, TLCD, and a group of non-governmental organizations. The alternative alignments are shown in figures 4-2 through 4-4 and are as follows:

1. Authorized Alignment at Post-Katrina 1% AEP Elevations – Alternative 1 (2008) generally follows the authorized plan alignment; however, it includes a few minor modifications to Reach G that were made between 2003 and 2008. The Reach G modifications are described in section 4.4.

2. Reconnaissance Alignment at Post-Katrina 1% AEP Elevations – Alternative 2 (2008) is based on the "Reconnaissance" alignment from the 2002 Feasibility Report.

3. Multiple Lines of Defense Strategy Alignment at Post-Katrina 1% AEP Elevations – Alternative 3 (2008) is based on a "Multiple Lines of Defense Strategy (MLODS)," presented by a group of non-governmental organizations, which considers the storm

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buffering potential of natural landscape features such as wetlands as the first line of defense against storm surge and waves. This alignment was not previously considered in the 2002 Feasibility Report. The Alternative 3 alignment includes Theriot and Dulac ring levees. As compared to the other alternatives, the MLODS alignment is located closer to development and has more marsh located outside the levee system.

4. <u>Authorized Alignment at Pre-Katrina Elevations</u> – Alternative 4 (2008) follows the same alignment as Alternative 1 (2008). The authorized project levees, lock, floodgates, environmental facilities, and floodwalls were reengineered to meet post-Katrina design criteria, but not the post-Katrina 1% AEP design elevations. This alternative was evaluated in order to compare the Pre- and Post-Katrina versions of the 2002 Feasibility Study Recommended Plan.
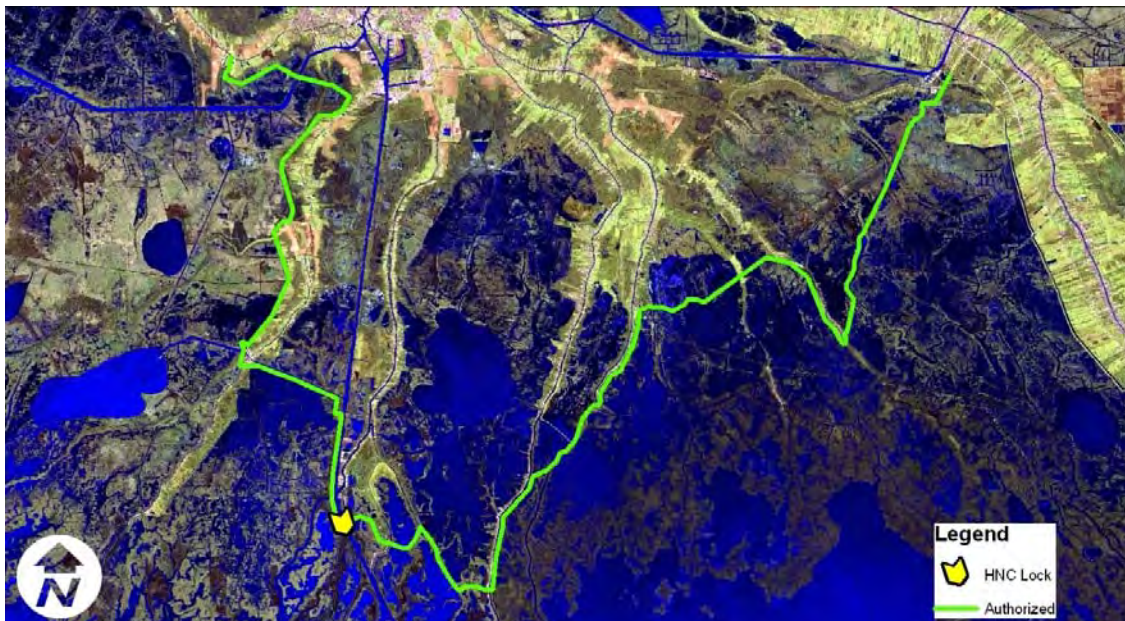
Figure 4-2. Alignment for 2008 Alternative 1 and 2008 Alternative 4

Figure 4-3. Alignment for 2008 Alternative 2

Figure 4-4. Alignment for 2008 Alternative 3

Note: Figure 4-4 (2008 Alternative 3) represents the MLODS alignment as originally proposed; however, the HNC lock would actually be located where the levee alignment intersects the HNC. In addition, a salinity control structure that could be overtopped during hurricane events would be located where the HNC lock is shown on the map.

Table 4-1 presents the results of the 2008 analysis of alignment alternatives. The 2008 analysis was based on the best available information at the time. Interim HSDRRS design criteria (USACE, 2007) available at the time were used. Some major differences between the 2008 analysis and the full feasibility analysis for the PAC report are described following the table.

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Table 4-1. Alignment Alternative Comparison of Elements

Feature	2008 Alternative 1	2008 Alternative 2	2008 Alternative 3	2008 Alternative 4
Hurricane and Storm Damage Reduction Design standards (year)	Post-Katrina (2008)	Post-Katrina (2008)	Post-Katrina (2008)	Pre-Katrina (2000)
B/C Ratio (Low to High Scenario)	0.67 to 1.07	0.60 to 0.98	0.49 to 0.79	0.59 to 1.40
Equivalent Annual Net Benefits in $Millions (Low to High Scenario)	($197) to $44	($223) to ($12)	($335) to ($136)	($98) to $99
Estimated Cost (High Scenario)	$ 8.6 B	$ 7.5 B	$ 8.3 B	$ 2.5 B
Levee Length	64.6 miles	55.2 miles	63 miles	64.6 miles
Total Cost per Mile	$133,000,000	$135,700,000	$131,300,000	$38,480,000
Levee Cost per Mile	$81,970,000	$81,270,000	$72,860,000	$17,430,000
Levee Height	Varies 18.5 to 28.5 ft	Varies 18.5 to 28.5 ft	Varies 18.5 to 23 ft	Varies 11 to 15 ft
Low: 14 to 18 ft	0 Miles	0 Miles	0 Miles	64.6 miles
Medium: 18.5 to 24.9 ft	33.3 Miles	29.9 Miles	63 Miles	0 Miles
Tall: 25 ft and over	31.3 Miles	25.3 Miles	0 Miles	0 Miles
Levee Base Width				
Lower Range	524	524	442	164
Upper Range	725	725	680	243
Embankment in Place (CY)	153,089,711	130,674,856	133,708,696	23,936,090
No. Lifts to Construct	Maximum of 5	Maximum of 5	Maximum of 5	Maximum of 3
Marsh Impacted(direct)	5,558 acres	4,953 acres	3,901 acres	1,562 acres
Bottomland Hardwoods Impacted (direct)	1,024 acres	1,358 acres	3,776 acres	331 acres
Right of Way Areas	14,527 acres	11,781 acres	13,381 acres	1,614 acres
Stability Berms	Both sides of levee	Both sides of levee	Both sides of levee	Unprotected side only
Nos. of Structures	40	36	40	39
HNC Lock Complex	1	1	1	1
250 ft Sector Gate	0	0	1	0
125 ft Sector Gate	3	3	3	3
56 ft. Sector Gate	10	8	9	10
46 ft. Tainter Gate	5	4	5	5
Sluice Gates	13	14	8	13
Fronting Protection	3	3	3	3
Highway Gates	5	3	10	5

*Benefits and costs are presented for a high damage scenario (high employment, highly dispersed land use, and high relative sea level rise). The without-project equivalent annual damages were estimated at $1.1 billion. Marsh and bottomland hardwood impacts are for direct construction of the levee only. Indirect impacts were not quantified for the 2008 preliminary analysis, but were considered by the interagency team.

The 2008 designs and costs are based on a typical levee cross-section to elevation +28 ft NAVD88 and representative structure designs. Engineering judgment was applied to scale each typical section higher or lower to match the design elevation provided for each specific hydraulic reach to estimate the required levee footprint, overbuild heights, and estimated lift schedule. The cost estimates (ARCADIS, 2008) include levees, floodgates, the HNC lock complex, tidal exchange/environmental control structures, pumping station fronting protection, road gates, utility relocations, real estate, Engineering and Design, Construction Management, and mitigation for direct impacts to marsh and bottomland hardwoods (wet and dry). A Cost and Schedule Risk Analysis Process was also performed according to guidelines set out by the USACE Walla Walla

District in a March 2008 document. Overall, the contingency for the 2008 estimate was calculated to be 23.5 percent at 80 percent confidence level. Since the 2008 analysis, additional borings have been taken to refine structure designs and cost estimates in the final alternatives analysis.

The 2008 economic benefits analysis was based on LACPR data and methods (LACPR, 2009) to calculate damages for the residential and non-residential categories, agricultural resources, transportation infrastructure and emergency cost savings and benefits during construction. The damages to boat fleets and business from saltwater intrusion were based on the 2002 feasibility report. The LACPR census block data was used instead of updated, detailed structure inventory data, which was not available at the time. Economic benefits were calculated using the LACPR GIS database instead of the USACE certified model for performing flood damage analysis, i.e. HEC-FDA.

Annualized benefits and costs were compared over a period of analysis from 2010 to 2081. The assumed base year, when the project would begin to provide the specified level of risk reduction, was assumed to be 2032 for Alternatives 1, 2, and 3 and 2020 for Alternative 4. In order to make the alternatives comparable, the Alternative 4 benefits stream was compounded 12 years from 2020 to 2032 to reflect a common base year (2032) for all four alternatives. Benefits during construction were calculated for all four alternatives. The benefit and cost values were converted to an equivalent time frame using October 2008 price levels and a Federal discount rate of 4-7/8 percent for the amortization period of 50 years.

Reflective of economic evaluations conducted under LACPR, the four alternatives were evaluated under two damage scenarios. The "high" damage scenario incorporates high employment, highly dispersed land use, and high relative sea level rise. The "low" damage scenario incorporates business-as-usual future development, compact land use, and low relative sea level rise.

Preliminary estimates of direct environmental impacts for each of the four alternatives and adjacent borrow sites were developed for the 2008 verification of the authorized alignment. The footprints of project features (levees, adjacent borrow pits, and the mitigation areas) were categorized by land form (wetlands, open water, or non-wet) and a spreadsheet was used to calculate impacts by wetland type (marsh, bottomland hardwood-dry, and bottomland hardwood-wet or "swamp"). All mitigation cost estimates in the PAC report are based on the following:

- Marsh mitigation at $80,000 per acre and a ratio of 1-to-1.

- Bottomland hardwood mitigation at $37,000 per acre and a ratio of 3-to-1, or $111,000 per acre, for bottomland hardwood-wet (swamp) and a ratio of 6-to-1, or $222,000 per acre, for bottomland hardwood-dry.

Wetland replacement ratios are for mitigation cost estimating purposes only; actual mitigation costs would be determined based on replacement of wetland habitat values and ratios could be higher or lower depending on specific habitat type (brackish, fresh, saline, etc).

As shown in table 4-1, Alternatives 1 and 4 (2008), which are both based on the authorized alignment, had the highest net benefits among the alternatives and indicated a potential for positive economic justification based upon the results for the high damage scenario. Alternatives 2 and 3 (2008) produced negative annual net benefits under both the high and low damage scenarios. This analysis demonstrates that both alternatives along the authorized alignment potentially remain economically justified, while the alternative alignments examined in Alternatives 2 and 3 (2008) are not likely to be economically justified. Alternatives 2 and 3 were also screened for the following reasons:

- Based on surge modeling, areas outside the levee system may experience higher statistical water levels than without the project. Alternatives 1 and 4 leave fewer assets outside of the levee alignment, most of which are camps or secondary residences. Alternatives 2 and 3 leave many more assets outside the levee system, which has greater implications for induced flooding impacts.

- Alternatives 1 and 4 provide much more internal storage in the case of levee overtopping or significant rainfall during a storm event. Alternatives 2 and 3 would likely require additional pumping capacity and would have higher residual risk in the case of levee overtopping.

- Although Alternatives 1 and 4 enclose more wetlands than Alternatives 2 and 3, the authorized alignment would be mostly constructed along existing raised features (e.g. ridges, local levees, etc.) and the addition of water control features would actually improve the hydrologic exchange in some areas. Although the shorter Alternative 2 and 3 alignments enclose fewer wetlands and have less direct marsh impacts, those alignments actually impact more bottomland hardwoods than the authorized alignment.

For the reasons stated above, the authorized alignment was reconfirmed as the alignment that best meets Federal objectives, i.e. most effective, efficient (cost-effective), complete, and acceptable, and was therefore carried forward for more detailed analysis.

5   Post-Authorization Changes to the Levee Alignment

As a result of new information obtained during the PED phase, several of the authorized levee reaches have been refined to reduce costs, reduce environmental and cultural resources impacts, and improve risk and reliability. For example, Hurricane Katrina and advances in storm surge modeling revealed that narrow, sharp indentations in the alignment can lead to stacking of surge, which increases risk and makes the levee less reliable. These refinements reduced the length of the 72-mile authorized alignment by approximately 11 miles, however, post-Katrina surge modeling demonstrated that the authorized project could potentially be flanked at either end. The alignment had to be extended 16 miles to the west and 21 miles to the east to complete the system. The result of these alignment modifications and extensions resulted in a 98-mile levee project as shown in Attachment 1.

5.1   Modifications to the Authorized Alignment

The following sections describe each of the post-authorization levee reach modifications starting with modifications to the reaches that were part of the authorized project (A, G, H, J, and L). No changes were made to Reaches B, E, F, I, or K.

5.1.1   Reach A

In October 2009, the USACE evaluated four alignment options for Reach A as follows:

- The 12-mile **A1** alignment is Reach A from the authorized project alignment. The authorized alignment follows the development line along Bayou Dularge.

- The 14-mile **A1A** alignment is a slight variation on the authorized (A1) alignment. A1A generally follows the A1 Alignment with an additional 2 miles of levee around an agricultural area, which is located along a natural ridge and has the potential for future development.

- The 10-mile **A2** alignment is located west of the authorized (A1) alignment and was considered because some landowners wanted more of their farmland inside the levee. The A2 alignment shortened the authorized (A1) alignment by 2 miles and reduced marsh and bottomland hardwood impacts.

- The 9-mile **A3** alignment is a compromise between A1 and A2. A3 is the shortest alignment, has the least marsh impacts, and bottomland hardwood impacts similar to A2.

Each alternative includes two 125-foot floodgates on the GIWW (reduced to one 125-ft sector gate in the final PAC alternatives), one 56-foot sector gate at Minors Canal, and 6-ft by 6-ft box culverts at four locations, except for the A2 Alignment which has only two box culvert locations also has three tainter gates associated with the sector gate. Subsequent to the analysis of the four options, an even more cost effective alignment was identified which is labeled "Current Alignment" in figure 5-1.
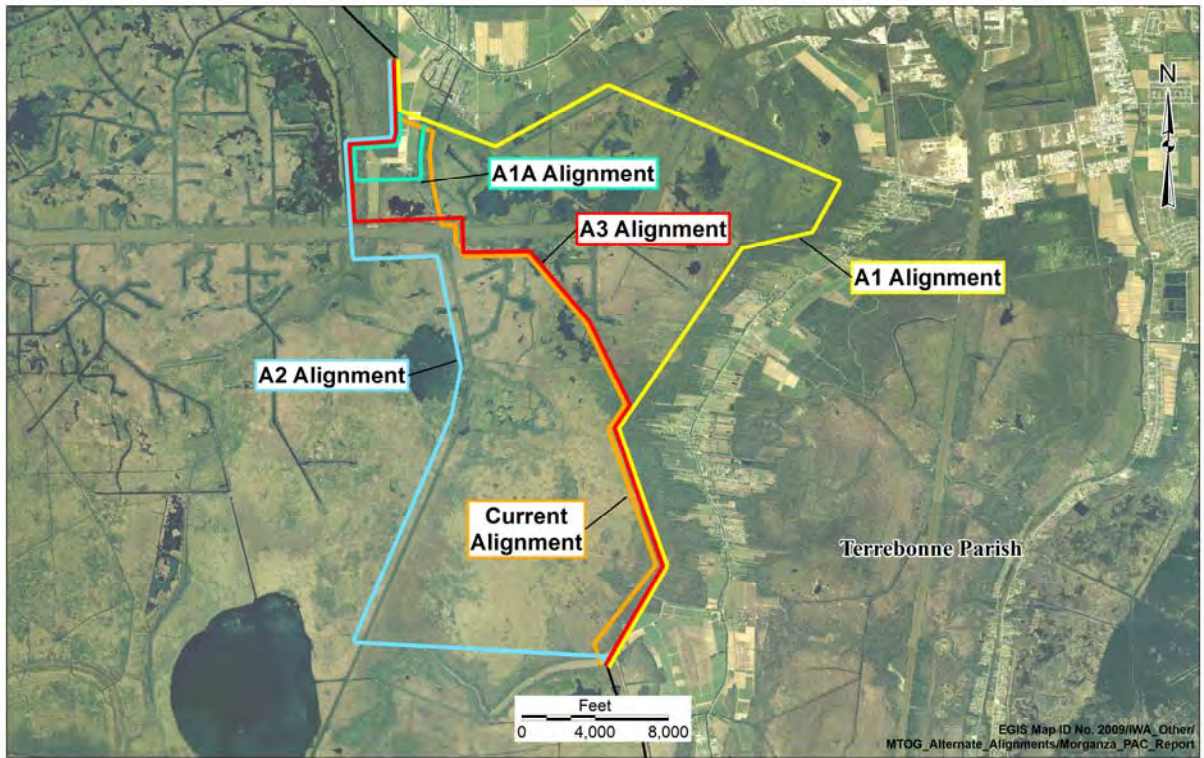
Figure 5-1. Levee Reach A Alignment Options

Environmental impacts were evaluated based on the amount of direct impacts to both marsh and bottomland hardwoods (wet and dry) and the approximate cost of mitigation as shown in table 5-1. Both Options A2 and A3 have the potential to impound more wetlands than Option A1, but box culverts are designed to reduce any potential indirect impacts. As compared to the authorized alignment (A1), the A3 alignment has more acres of bottomland hardwood (dry) impacts, but less bottomland hardwood (wet) impacts and marsh impacts.

Table 5-1. Direct Environmental Impacts for Reach A Options

Reach	Levee Impacts (acres)			Approximate Mitigation Costs ($ Millions)			
	Marsh	**BLH Dry**	**BLH Wet**	**Marsh**	**BLH Dry**	**BLH Wet**	**Total**
A1	312	33	168	$25	$7	$19	$51
A1A	312	68	222	$25	$15	$25	$65
A2	306	56	62	$25	$12	$7	$44
A3	256	60	67	$21	$13	$7	$41

BLH = bottomland hardwood

The estimated project cost was compared for each alternative alignment as shown in table 5-2. Cost estimate includes the cost of construction, real estate, and mitigation. Cost estimates assume levees are constructed from hauled-in material from off-site borrow.

Table 5-2. Cost Estimates for Reach A Options

Reach A Options	Estimated Total Cost ($ Millions)
A1	$767
A1 including A1A	$870
A2	$680
A3	$626

Originally, Option A3 was selected because it was the most cost effective of the four alignment options. The alignment was later modified to exclude the agricultural area with the potential for future development (see area protected by Option A1A in figure 5-1). The alignment designated as "Current Alignment" in figure 5-1 is the Reach A alignment carried forward into the Morganza to the Gulf PAC levee alignment.

5.1.2   Reach G

Five alignment options for Reach G were evaluated as shown in figure 5-2 (ARCADIS, 2011). The options included the authorized alignment from the 2002 feasibility report, an alignment developed during the PED phase, and three other alignments developed for the PAC report as follows:

- **Feasibility** alignment (7.5 miles) – Includes one road crossing and two drainage structures.
- **PED** alignment (5.3 miles) – Includes one road crossing, a 30-ft stop log, and two drainage structures.
- **PAC 1** alignment (4.6 miles) – Includes one road crossing, a 30-ft stop log, and two drainage structures.
- **PAC 2** alignment (4.3 miles) – Includes one road crossing, a 30-ft stop log, and three drainage structures.
- **PAC 3** alignment (4.9 miles) – Includes one road crossing and two drainage structures.

The PED alignment reduced the authorized (feasibility) alignment costs by shortening the alignment and reducing wetland impacts by crossing open water. The remaining PAC alternatives were attempts to optimize the alignment to achieve the most cost-effective number of structures balanced with the shortest alignment and least environmental impacts.
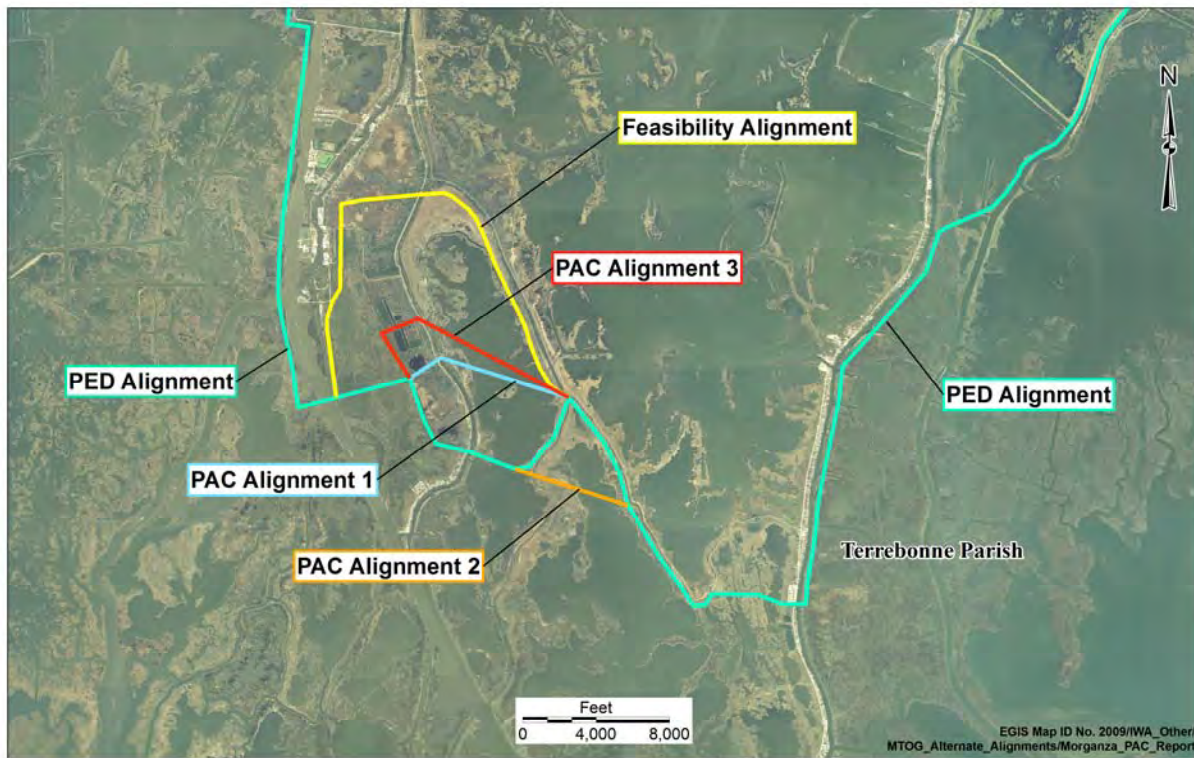
Figure 5-2. Levee Reach G Alignment Options

Environmental impacts were evaluated based on the amount of direct impacts to both marsh and bottomland hardwoods (wet and dry) due to the levee placement and the borrow pits as shown in table 5-3. Cost for the mitigation was based on a ratio, not on habitat value.

Table 5-3. Direct Environmental Impacts for Reach G Options

Reach G Options	Levee Impact (Acres)			Borrow Impact (Acres)			Total Impact (Acres)			Total Mitigation Cost ($ Millions)
	Marsh	BLH Dry	BLH Wet	Marsh	BLH Dry	BLH Wet	Marsh	BLH Dry	BLH Wet	
Feasibility	336	0	31	143	0	2	479	0	32	$42
PED	238	0	5	30	0	0	268	0	5	$22
PAC1	171	0	4	73	0	0	244	0	4	$20
PAC2	178	0	0	33	0	0	211	0	0	$17
PAC3	177	0	<1	56	0	0	233	0	<1	$19

BLH = bottomland hardwood

The estimated project cost was compared for each alternative alignment as shown in table 5-4. Cost estimates include the cost of construction, real estate, and mitigation.

Table 5-4. Cost comparison for Reach G Alignment Options

Reach G Options	Estimated Total Cost
Feasibility	$581
PED	$445
PAC 1	$407
PAC 2	$400
PAC 3	$410

The feasibility alignment was screened out because it has the highest cost, highest direct impacts, and two known cultural sites within the alignment. The PAC2 option was selected as the preferred alternative because it is the most cost effective alternative. The PAC2 option presents a tradeoff between direct and potential indirect impacts. Of all the options, the PAC2 option would have the least direct wetland impacts, but would enclose the largest amount of marsh and open water. The National Marine Fisheries Service (NMFS) and the rest of the Habitat Evaluation Team were initially concerned about the potential for indirect impacts to marsh and fishery access to wetlands and Essential Fish Habitat on the protected side; however, those concerns have been reduced by demonstrating minimal indirect impacts through systemwide modeling of environmental control structures.

5.1.3 Reach H, Segments 2 and 3 (H-2/H-3)

In 2005, the TLCD performed an alternative alignments analysis on Reach H, Segments 2 and 3 (Shaw, 2005). Agency representatives from USACE, U.S. Fish and Wildlife Service (USFWS), and NMFS had input on the alternative analysis, which considered engineering feasibility, environmental impacts, and construction costs. The following four alignments were evaluated (see figure 5-3):

- The Existing Alignment from the 2002 feasibility report, which follows the natural ridge.

- A Set Back Alignment, which moves the levee alignment out away from the ridge.

- An Existing Alignment Cross Over, which follows the Existing Alignment (see first bullet) from the south, but then crosses over to the northeast to join Reach I, eliminating the need to improve the Bush Canal levee and associated pump station in Reach I.

- A Set Back Alignment Cross Over, which follows the Set Back Alignment (see second bullet) from the south, but then crosses over to the northeast to join Reach I, eliminating the need to improve the Bush Canal levee and associated pump station in Reach I.
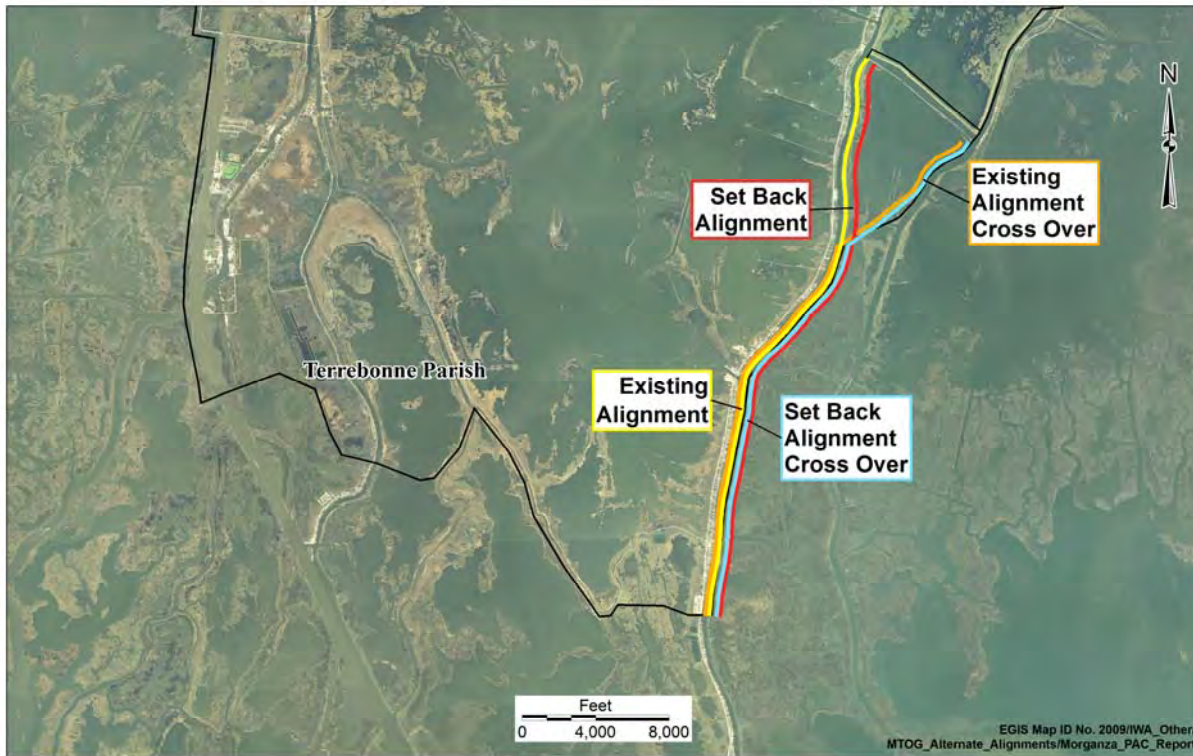
Figure 5-3. Levee Reach H Alignment Options

Additional alternatives were evaluated by combining the four alignments with different borrow assumptions (i.e., adjacent borrow, haul-in borrow, etc.). Adjacent borrow was found to be the most cost effective method. Table 5-5 shows a comparison of the four alignment alternatives assuming adjacent borrow.

Table 5-5. Estimated Costs and Direct Impacts for Reach H Options

Reach H Options	Total Impacts (acres)*	Estimated Costs ($ Millions)				
		Construct-ion Cost	Mitigation Cost	Total Cost	Bush Canal Savings	Total with Bush Canal Savings
Existing Alignment	368	$13	$7	$20	$0	$20
Set Back Alignment	453	$16	$9	$25	$0	$25
Existing Alignment Cross Over	291	$14	$5	$19	$2	$17
Set Back Alignment Cross Over	447	$15	$9	$24	$2	$22

*Includes ridge, marsh, spoil bank, and marsh impoundment.

Total costs of alternatives, taking Bush Canal savings into consideration, ranged from approximately $17 million to $72 million (an alternative with offsite borrow not shown in table 5-5). The Existing Alignment Cross Over alternative was selected as the new alignment since it had the lowest total cost of all the alternatives, either with or without the Bush Canal savings, of approximately $17 million or $19 million, respectively.

Both economically and environmentally it made sense to move the alignment to its current location. This decision was confirmed during the USACE Section 10/404 permit and State of Louisiana Coastal Use Permit evaluations. The TLCD was ultimately granted a Department of the Army permit for reaches H-2 and H-3 in November 2008.

5.1.4   Reach J, Segment 2 (J-2)

In 2004, the TLCD, in cooperation with the USACE, evaluated two alternatives for Reach J, Segment 2 as shown in figure 5-4 (GSE, 2005). In the cost projections prepared for the 2002 feasibility report, the proposed hurricane levee (J-2 Northern) was assumed to be located on natural levee soil landforms consisting of moderately strong to strong clays. An analysis of geotechnical and historic data along the existing Montegut Forced Drainage Levee system, however, indicated that as much as 70 percent of the J-2 Northern alignment, which lies outside (flood side) of the existing forced drainage levee alignment, would be placed on landforms consisting of relatively deep peat layers, overlying weak clay deposits. The unanticipated soil conditions led to an increase in projected costs associated with the planned alignment (J-2 Northern).

Figure 5-4. Levee Reach J-2 Alignment Options

As an alternative to the J-2 Northern alignment, the team evaluated an alignment following the existing Department of Natural Resources (DNR) marsh restoration levee. Table 5-6 provides a comparison of the J-2 Northern alignment and the alternative J-2 DNR alignment. The J-2 DNR alignment shortens the total levee length from the Humble Canal to the western terminus of Segment J-1 from approximately 9.5 to 5.3 miles. The J-2 DNR levee would reduce direct wetland impacts from 345 acres to 83 acres. The J-2 DNR levee alignment would also reduce all

bottomland hardwood impacts to zero. Indirect impacts would be minimal because of the existing levees and structures associated with the J-2 DNR marsh management system.

Table 5-6. Comparison of Reach J-2 Alignment Options

Item	J-2 Northern Alignment	J-2 DNR Alignment
Levee Length	9.5 miles	5.3 miles
Environmental Control Structures	1	3
Pipeline crossings	4	3
Footprint Area	556 acres	323 acres*
Wetlands Impacted	345 acres	83 acres
Additional Wetlands Restoration Area	0 acres	1,035 acres
Total Cost	$32 Million	$18 Million

*Within the 323-acre footprint, 138 acres are existing levee and borrow canal, so the DNR Alternate Alignment only adds 185 acres to the new footprint.

The J-2 DNR alternative was selected because it has less environmental impact and is less expensive than the original J-2 Northern alignment. The cost savings associated with the construction of the J-2 DNR alignment versus the construction of the original J-2 Northern alignment was estimated at over $14 million. The reduced cost for the construction of the more southern J-2 DNR alignment is primarily due to the fact that most of this levee would be constructed on existing levees. Approximately 20 percent of the alignment would be on stable levees and would not require reinforced geotechnical fabric or a sand base. Those portions of the new levee constructed on the J-2 DNR levee sections would require less embankment fill, as well as less sand fill than required in the authorized alignment.

5.1.5   Reach L

In October 2009, the USACE evaluated three alignment options for Reach L (see figure 5-5):

- **L1** is the authorized alignment, which consists of 5.4 miles of levee and two box culvert locations.
- The **L2 alignment** was developed during PED discussions as having potential to reduce Larose to Golden Meadow upgrade costs. L2 consists of 6.1 miles of levee and two box culvert locations.
- **The L3** alignment was requested by stakeholders, because it is similar to L2 in length and impacts and has the added economic benefit of including Apache Mineral property inside the alignment and further reducing Larose to Golden Meadow improvement costs. L3 consists of 6.1 miles of levee and one box culvert location.

Each alternative has one 56-ft sector gate with three 46-ft tainter gates and 6-ft by 6-ft box culverts at various locations.
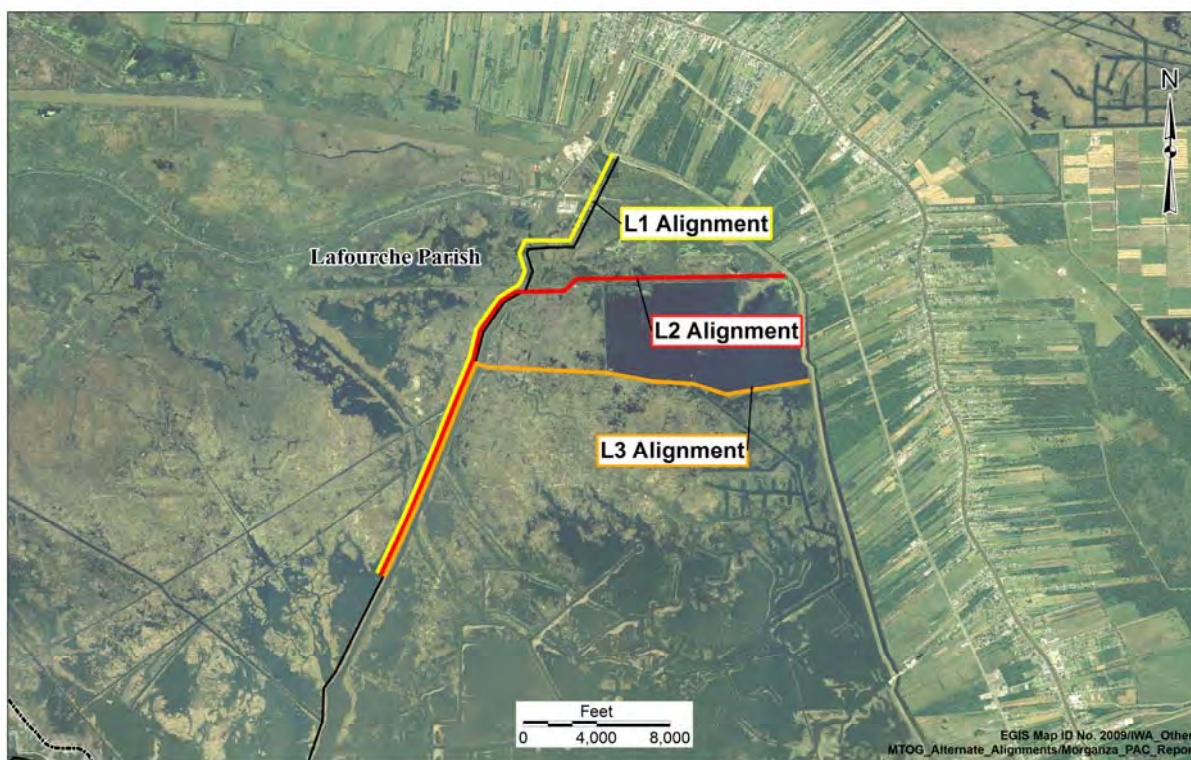
Figure 5-5. Levee Reach L Alignment Options

Environmental impacts were evaluated based on the amount of direct and indirect impacts to both marsh and bottomland hardwoods (wet and dry) and the approximate cost of mitigation as shown in table 5-7.

Table 5-7. Direct Environmental Impacts for Reach L Options

	Levee Impacts				**Approximate Mitigation Costs ($ Millions)**			
Reach L Options	**Levee length (mile)**	**Marsh* (acres)**	**BLH dry (acres)**	**BLH wet (acres)**	**Marsh#**	**BLH Dry**	**BLH Wet**	**Total**
L1	5.4	209	19	0	$17	$4	$0	$21
L2	6.1	240	0	29	$19	$0	$3	$22
L3	6.1	163	0	0	$13	$0	$0	$13

BLH = bottomland hardwood.

Option L3 assumes placement of the alignment inside existing levee, not south of it. With Option L2 there is a potential to enclose approximately 1,320 acres of wetlands more than L1. With Option L3 there is a potential to enclose approximately 1,970 acres of wetlands more than the L1 alignment. The estimated project cost was compared for each alternative alignment as shown in table 5-8.

Cost estimates include the cost of construction, real estate, and mitigation. All earthen levees would be constructed of hauled-in material from off-site borrow. Although Reach L3 is longer

45

than the authorized alignment (L1), it would reduce the length of the existing Larose to Golden Meadow levee that would need to be raised, resulting in an overall lower net cost.

Table 5-8. Cost Estimates for Reach L Options

Reach L Options	Estimated Costs ($ Millions)		
	Total Cost	**Larose Savings***	**Net Costs**
L1	$561	$0	$561
L2	$633	$207	$427
L3	$615	$297	$319

*Savings for portions of Larose to Golden Meadow levees that would not need to be raised. These savings may or may not be realized pending the outcome of the Larose PAC, which is not yet complete. Additional discussions on the Larose to Golden Meadow project are included in sections 5.2.2 and 6.5.2.

Option L3 was selected because it is the most cost effective alternative. As compared to the authorized alignment, the new alignment has less bottomland hardwood (dry) impacts and less marsh impacts. Option L3 encloses approximately 2,000 additional acres compared to the authorized alignment (L1), but box culverts are designed to reduce any potential indirect impacts.

5.2   Extensions to the Authorized Alignment

Based on relative sea level rise projections and higher post-Katrina surge levels, the authorized alignment had to be extended to the west and to the east in order to complete the Morganza to the Gulf project. Both PAC alternatives follow the same 98-mile levee alignment but at different levels of risk reduction (1% and 3% AEP).

5.2.1   Western Levee Extension (Barrier Reach)

Two alternative alignments were evaluated as part of a screening-level analysis—a "Barrier Alignment" and a "Northern Alignment." Both alignments originate at Minors Canal. The alternatives are as follows:

- The Barrier Alignment is 15.4 miles and generally follows the edge of development along the south side of the Black Bayou ridge (LA 182/Old US 90).

- The Northern Alignment is 15.6 miles and runs west along Bayou Black Ridge for approximately 2 miles, then turns north and follows Savane Road (Parish Road 23) up to the Little Bayou Black Ridge. The alignment then follows the southern development boundary along Bull Run Road (Chacahoula ridge) northwest until it ties in to Highway 90.

As shown in figure 5-6, the Barrier Alignment incorporates additional assets and people along the Bayou Black ridge that are not included within the Northern alignment. Table 5-9 includes the number of people and the value of equivalent annual benefits based on the 2009 inventory of structures located between the Barrier and Northern alignments. Table 5-9 also provides the estimated cost for each alignment as well as the cost difference between the two alignments. The Northern Alignment has not been designed to full feasibility level, but would have the same levee width as the Barrier, but at a slightly lower elevation. The Northern Alignment would also require fewer structures than the Barrier alignment.
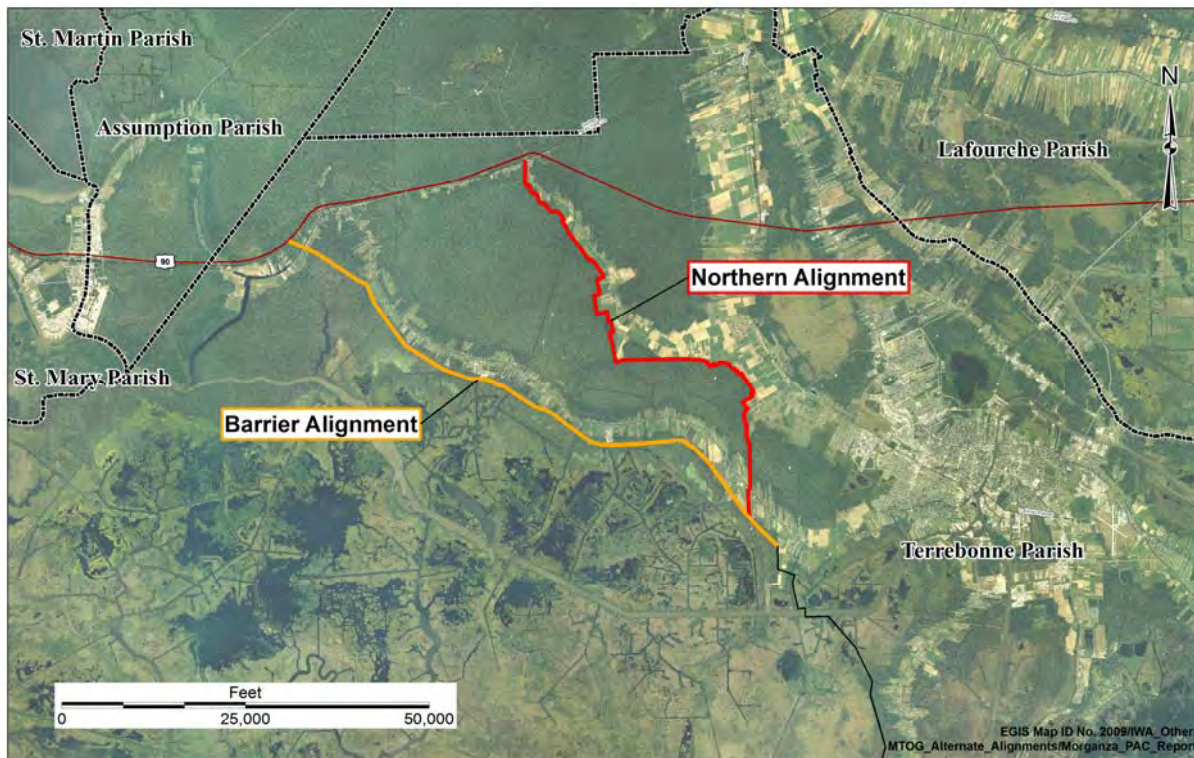
Figure 5-6. Barrier and Northern Levee Alignment Options

Table 5-9. Incremental Costs and Benefits between Northern and Barrier Levee Alignments

Options	Estimated Cost ($millions)	Incremental Total Benefits ($millions)	Incremental Equivalent Annual Benefits ($millions)	Incremental Number of People
Northern Alignment	$710	N/A	N/A	N/A
Barrier Alignment	$965.2	$603.3	$28.1	3,400
Increment Between Barrier and Northern	$255.2	$603.3	$28.1	3,400

Note: Economic benefits calculated based on 2035 to 2085 period of analysis. Number of people estimated based on 2009 structure inventory. Construction cost does not include mitigation costs; see table below.

Table 5-10. Direct Wetland Impacts for Northern and Barrier Levee Alignment Options

Option	Total Footprint Acres	Marsh Acres	Swamp Acres	Other Acres (uplands, open water, etc.)	Estimated Mitigation Cost ($millions)
Barrier	691	86	518	87	$64.4
Northern	634	0	464	170	$51.5

Although the Barrier Alignment has a slightly larger footprint and impacts more wetlands as shown in table 5-10 and has a higher cost than the Northern Alignment, the Barrier was selected to complete the Morganza to the Gulf levee alignment because it reduces risk to 3,400 more people than the Northern Alignment. As well as leaving a large number of people unprotected, the Northern alignment could induce damages on Bayou Black areas to the south and west of the alignment.

5.2.2 Eastern Levee Extensions (Larose C-North and Lockport to Larose Reaches)

The eastern extent of the authorized Morganza to the Gulf project (Reach L) ties into the northwestern portion of the existing Larose to Golden Meadow project. The Larose to Golden Meadow ring levee system extends from the town of Larose, LA to a point 2 miles south of Golden Meadow, LA and provides hurricane and storm damage reduction to roughly 25,000 people living on both sides of Bayou Lafourche. Surge modeling for the 2002 feasibility report did not show flanking of the Morganza project from the east across Bayou Lafourche, but new modeling shows that the Morganza project could be flanked if the existing Larose to Golden Meadow levee elevations are not brought up to Morganza-compatible design standards.

The Larose to Golden Meadow project is currently undergoing a PAC analysis similar to the Morganza to the Gulf PAC analysis and future levee elevations for the Larose ring levee system have not yet been determined, however, none of the Larose PAC alternatives include raising the Larose levees to the 1% AEP risk reduction level. The highest level of risk reduction being considered is a 2% AEP plan. In the event that Congress does not re-authorize and fund improvements to the Larose to Golden Meadow ring levee to bring it up to a level of risk reduction comparable to the Morganza project, a 7-mile levee following a portion of the Larose to Golden Meadow levee alignment has been added to the Morganza PAC project (labeled as C-North and GIWW reaches on figure 5-7 but collectively referred to as "Larose C-North Reach" throughout the rest of this report).
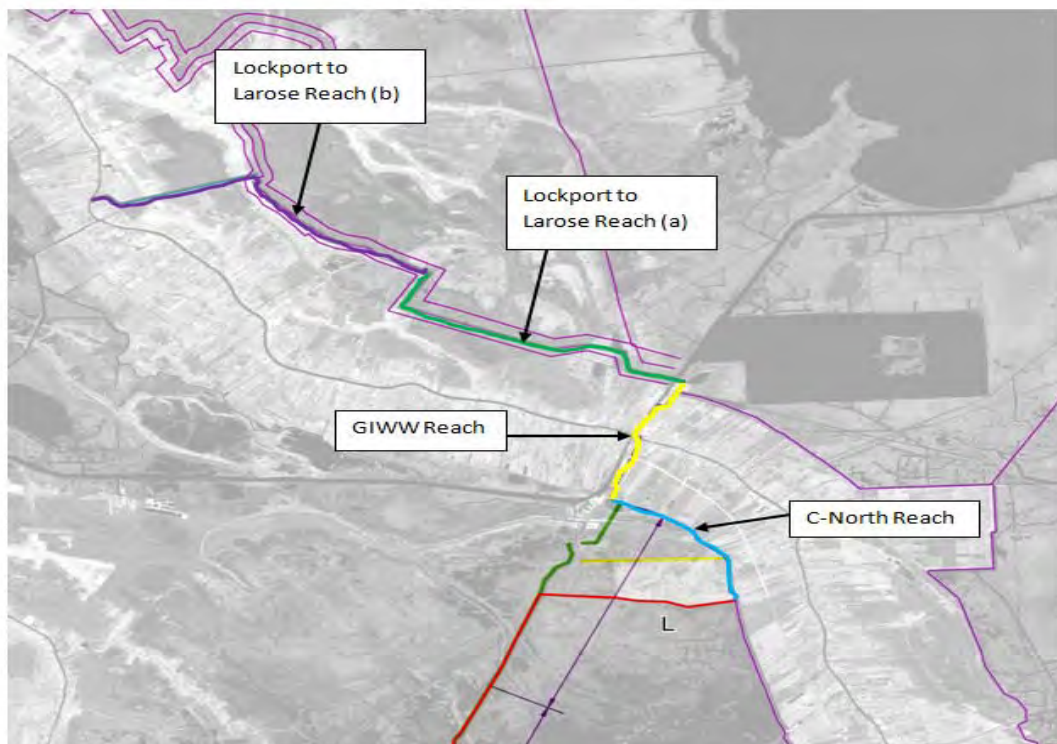
Figure 5-7. Barrier and Northern Levee Hydraulic Reaches

To complete the Morganza to the Gulf system, the alignment had to be further extended from the Larose C-North Reach up to Lockport, LA (see figure 5-7). The Lockport to Larose Reach follows an alignment formerly proposed under the Donaldsonville to the Gulf feasibility study. This levee reach was included because the Bayou Lafourche ridge, which is approximately 6 ft in elevation, provides some protection from surges coming up from the Barataria Basin, but could be overtopped in the future with relative sea level rise.

6 Evaluation and Comparison of Alternatives

In accordance with the WRDA 2007 implementation guidance, the PAC report compares the following two alternatives in detail:

- **3% AEP Alternative (pre-Katrina 100-yr alternative)** - The 3% AEP alternative is a hurricane levee system that provides risk reduction for water levels that have a three percent probability of occurring each year. This alternative is generally based on the pre-Katrina authorized project, but is modified to be consistent with post-Katrina design standards. Levee elevations for future conditions (year 2085) range from +13 to +20 ft NAVD88. Structure elevations range from +15 to +25 ft NAVD88.

- **1% AEP Alternative (post-Katrina 100-yr alternative)** - The 1% AEP alternative is a hurricane levee system that provides risk reduction for water levels that have a 1 percent probability of occurring each year. This alternative has the same intended level of risk reduction as the pre-Katrina authorized project, but is modified to be consistent with post-Katrina design standards. Levee elevations for future conditions (year 2085) range from +19.5 to +26.5 ft NAVD88. Structure elevations range from +17.5 to +33 ft NAVD88.

The 3% and 1% AEP alternatives both follow the same 98-mile levee alignment previously described in section 5, Post-Authorization Changes to the Levee Alignment.

6.1 Comparison of 3% and 1% AEP Levee and Floodgate Designs

The same storms used to model the without-project condition were simulated to estimate water levels and waves along the proposed levee project alignment. Outputs included surge and wave information at 320 locations along the proposed levee alignment. Table 6-1 summarizes the approximate surge and wave elevations used to design levee elevations by reach.

Table 6-1. Predicted With-Project Surge and Waves from 2035 (Base) to 2085 (Future)

Levee Reach(es)	Surge Still Water Elevations (ft NAVD88)		Wave Heights (ft NAVD88)	
	3% AEP	**1% AEP**	**3% AEP**	**1% AEP**
Barrier Reach and Reach A North of the GIWW	7 to 10	10 to 14	2 to 3	3 to 5
Reach A South of the GIWW and Reach B	9 to 11	12 to 14	2 to 3	4 to 5
Reaches E and F	11 to 12	15 to 17	2	4 to 5
Reach G	11 to 12	15 to 16	5	7
Reaches H, I, and J	12 to 14	16 to 18	5 to 6	7 to 8
Reaches K and L	12 to 13	16 to 18	3 to 4	5 to 6
C-North (section of Larose)	10 to 12	14 to 16	2 to 3	3
GIWW (section of Larose)	7 to 9	9 to 11	2	2 to 3
Lockport to Larose (a)	7 to 9	9 to 10	3 to 4	4 to 5
Lockport to Larose (b)	6 to 8	8 to 9	2 to 3	3 to 4

Note: Values are rounded to the nearest foot and are summarized from more detailed information in Engineering Appendix tables 40, 42, 51, 52, 53, 94, 95, 97, and 98. Values are for the Intermediate RSLR scenario. See section 6.9 for a discussion on the Low and High RSLR scenarios.

Levee dimensions are shown in table 6-2. Levee crown elevations were determined by ensuring that the elevation equaled or exceeded the required hydraulic design elevation given the Intermediate RSLR scenario at any given point in time between the base (2035) and future (2085) years. The design elevations vary by levee reach because of surge and wave differences due to storm path, wind speeds and direction, etc. The variation in design elevations is required to provide the same level of risk reduction along the entire length of the project. The 1% AEP alternative design elevations are generally 5.5 to 8 ft higher than the corresponding 3% AEP design elevations.

Table 6-2. Comparison of Levee Reach Dimensions for the 3% and 1% AEP Alternatives

Levee Reach	Approx. Length (miles)	Authorized Elevation (NGVD)	Range of Levee Design Elevations Between 2035 and 2085 (ft NAVD88)		Maximum Levee Toe to Toe Width (ft)	
			3% AEP Alternative	1% AEP Alternative	3% AEP	1% AEP
Barrier	16	N/A	10 to 13	15.5 to 20	174	329
A	8	10.5	10 to 13	15.5 to 20.5	174	329
B	5	12	11.5 to 13.5	17.5 to 20.5	355	610
E	5	14	14.5 to 15.5	21.5 to 23.5	440	725
F	4	14	14.5 to 15.5	22 to 23.5	270	490
G	6	15	16.5 to 17.5	22.5 to 24	270	550
H	8	15 to 16	18.5 to 20	24 to 26.5	330	500
I	6	14 to 15	18.5 to 20	24 to 26.5	319	570
J	9	14	18.5 to 20	24 to 26.5	337	660
K	5	12 to 14	16.5 to 17.5	22.5 to 25.5	400	635
L	6	10 to 11	16.5 to 17.5	22.5 to 25.5	400	635
Larose C-North	7	N/A	13.5 to 15.5	18 to 20.5	252	467
Lockport to Larose	13	N/A	8.5 to 12	10.5 to 15	282	282

Note the different datum for the authorized (NGVD) and current (NAVD88) elevations. The change in elevation due to datum differences varies by location, and is around 0.5 to 1.5 ft.

Table 6-3 lists floodgates along the levee alignment from west to east; the floodgate dimensions refer to the primary gate width. Structure elevations are higher than levees within the same reach because structure designs include 2 ft of additional height (structural superiority) to accommodate potentially higher RSLR and other uncertainties.

Hundreds of detailed engineering drawings showing phased construction site plans, cross sections, and foundation plans were developed for the PAC report. For most structural features, designs were prepared for a limited number of structures and the remainders were pro-rated.

Table 6-3. Floodgate Elevations for the 3% and 1% AEP Alternatives

Reach	Waterway	Structure Design Size/Type (subject to change during detailed design)	3% AEP Design Elevation (ft)	1% AEP Design Elevation (ft)
Barrier	Bayou Black	56-ft sector gate	15.0	22.0
	Shell Canal West	30-ft stop log gate	16.0	23.5
	Shell Canal East	56-ft sector gate	16.0	23.5
	Elliot Jones Canal	20-ft stop-log gate	16.0	23.5
	Humphreys Canal	20-ft stop-log gate	16.0	23.5
A (north of GIWW)	Minors Canal	56-ft sector gate	16.0	23.0
A	GIWW West (at Houma)	125-ft sector gate and nine 16-ft sluice gates	16.0	23.0
B	Marmande Canal	30-ft stop-log gate	16.5	23.0
	Falgout Canal	56-ft sector gate and nine 16-ft sluice gates	16.5	23.0
E-2	Bayou Dularge	56-ft sector gate	18.0	25.5
F-1	Bayou Grand Caillou	56-ft sector gate and nine 16-ft sluice gates	18.0	25.5
G-1	HNC	110-ft wide by 800-ft long lock with an adjacent 250-ft wide sector gate and ten 10-ft sluice gates	22.5	30.5
G-2	Four Point Bayou	30-ft stop-log gate	22.5	30.0
H-1	Bayou Petit Caillou	56-ft sector gate and six 16-ft sluice gates	22.5	30.5
H-2	Placid Canal	56-ft sector gate and six 16-ft sluice gates	24.0	31.5
H-3	Bush Canal	56-ft sector gate and nine 16-ft sluice gates	25.0	33.0
I-1	Bayou Terrebonne	56-ft sector gate	25.0	33.0
I-3	Humble Canal	56-ft sector gate	25.0	33.0
J-3	Bayou Pointe aux Chenes	56-ft sector gate	25.0	33.0
L	Grand Bayou	56-ft sector gate and nine 16-ft sluice gates	21.0	29.5
Larose C-North	Bayou Lafourche	56-ft sector gate	14.0	17.0
	GIWW East (at Larose)	125-ft sector gate and six 16-ft sluice gates	17.0	21.5

6.2   Multi-Lift Levee Construction

Levees were placed as close as practical to the natural ridges along existing or former bayous to take advantage of the stronger foundation conditions and reduce the probability of failure during and after construction. Sediments that comprise the natural ridges are stronger than the sediments in the lower surrounding areas, especially sediments that are in open water environments. Levee construction on top of the marsh sediments will be challenging from a stability and settlement viewpoint, especially for levee heights exceeding 15 ft. Multi-lift construction will help achieve the desired crown elevations. Geotextile reinforcement, berms, and flat berm slopes were used to achieve stable levees. Settlement analyses are an important part of the study due to the added construction cost associated with settlement during and after construction.

Generally, the approach for multi-lift levee construction consists of an initial preload lift to an elevation between +12 to +14 ft NAVD88 to provide a good base and working surface. Prior to construction on the second lift (first enlargement), the initial preload lift is expected to settle to approximately +10 ft NAVD88. As part of second lift construction, the existing preload levee would be degraded to approximately +4 ft NAVD88 and reinforcing geotextile installed. To achieve levee design elevations at target years 2035 and 2085, each reach requires two or three additional lifts between approximately 2020 and 2070. Levee lift elevations at the time of construction would generally be a few feet higher than the targeted design elevations to allow for levee settlement. After construction of each levee lift is completed, the levee would be covered in grass to increase its resilience in the case of wave overtopping.

As shown in table 6-4, the 1% AEP plan requires more than twice the material for levee construction as the 3% AEP alternative plan.

Table 6-4. Material Quantities by Levee Lift and Alternative

Levee Lift	Estimated Quantities (CY)	
	3% AEP Alternative	1% AEP Alternative
1st Lift/Preload	40,596,000	43,396,000
2nd Lift/1st Enlargement	14,458,000	65,929,000
3rd Lift/2nd Enlargement	4,581,000	10,725,544
4th Lift/3rd Enlargement*	1,224,000	6,182,000
Total	60,859,000	126,233,000

*Since C-North is the only reach with a 5th Lift/4th Enlargement, those quantities have been grouped with the 4th Lift/3rd Enlargement quantities for the rest of the levee reaches. All quantities rounded to the nearest 1,000.

Figures 6-1 and 6-2 are examples of lift schedules for different levee reaches and different levels of risk reduction. Figure 6-1 shows the lift schedule for Reach A/Barrier in the 3% AEP alternative, which has the lowest elevation and requires only three lifts to achieve the final design elevation (13 ft) in 2085. Reach A/Barrier would meet the 2035 design elevation (10 ft) after the first lift. Figure 6-2 shows the lift schedule for Reach J in the 1% AEP alternative, which has the highest elevation and requires four total lifts to achieve the final design elevation (26.5 ft) in 2085. Reach J would meet the 2035 design elevation (24 ft) after two lifts.
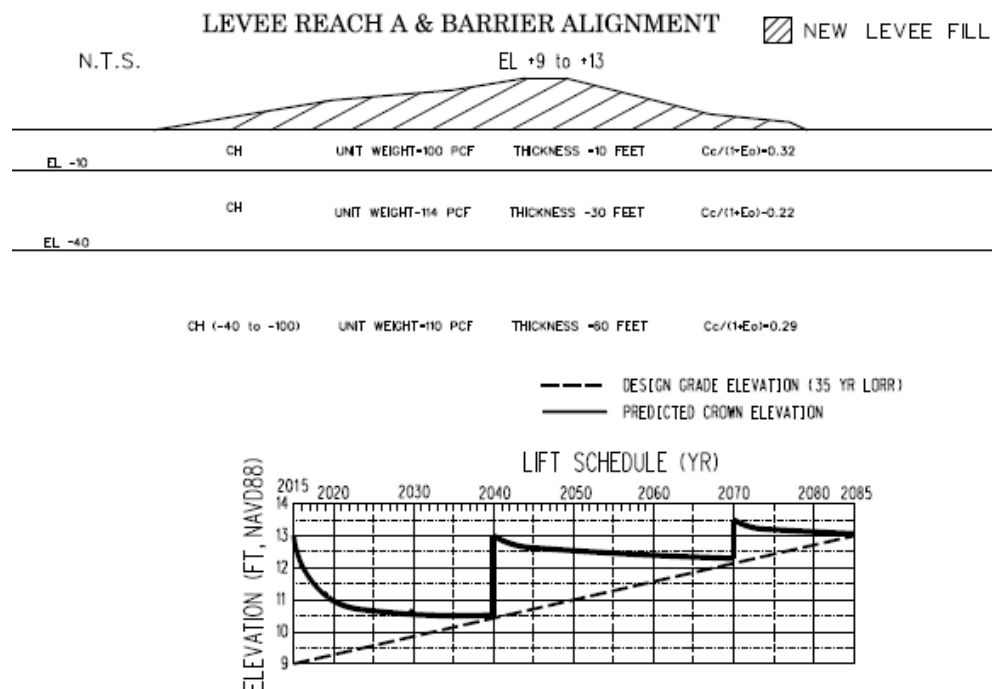
Figure 6-1. Example Lift Schedule for 3% AEP Alternative Reach A/Barrier
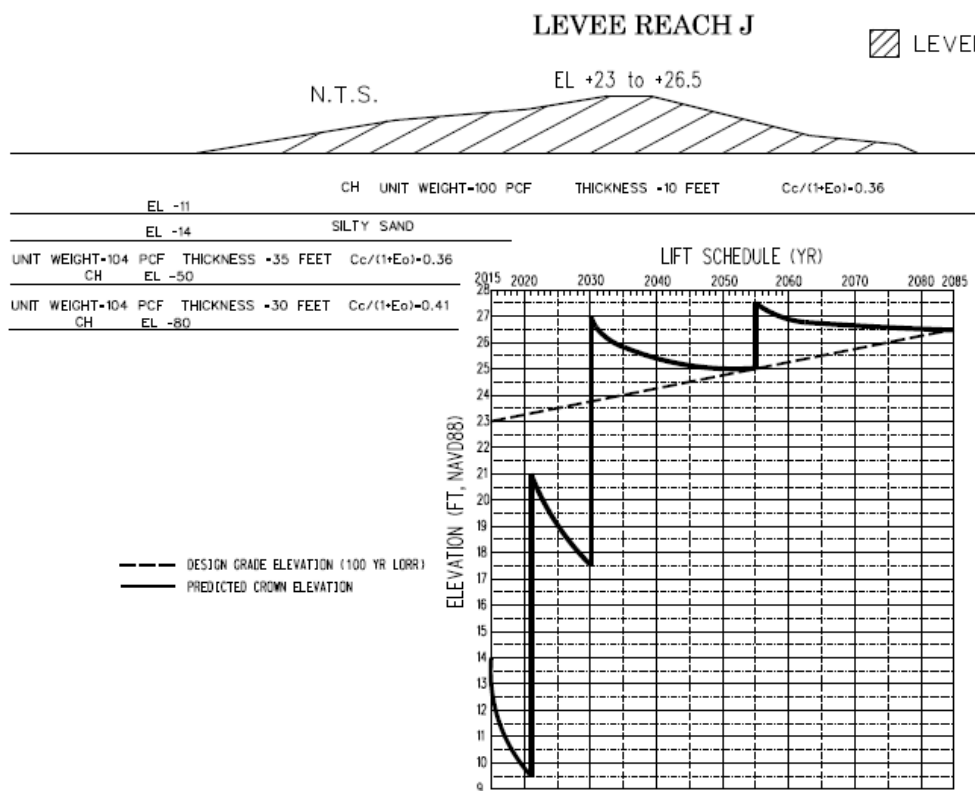
Figure 6-2. Example Lift Schedule for 1% AEP Alternative Reach E

6.3   Construction Schedule Assumptions

Preliminary construction schedules for alternatives are needed to calculate annual cost streams and benefit-cost ratios. Project benefits cannot start accruing until a "closed" risk reduction system is in place, which would require, at a minimum, all structures and first lift levees to be constructed. Assuming construction starting in 2015 and adequate funding (schedules were not constrained by potential funding levels), both alternatives would have a closed system by 2024. Based on reasonable construction durations that allow for levees to settle and compact between lifts, the alternatives could achieve their associated 3% or 1% AEP levels of risk reduction by 2035, however, as shown in table 6-5, the 3% AEP plan could achieve its base year design elevations almost 10 years sooner than the 1% AEP plan.

Table 6-5. Implementation Schedule Assumptions by Activity

Activities	Timeframe for 3% AEP	Timeframe for 1% AEP
Acquire Real Estate, Relocate Utilities, and Mitigate	2014 to 2023	2014 to 2025
Construct Lock, Floodgates, and Environmental Control Structures	2015 to 2024	2015 to 2024
Construct Levee Lifts to Achieve Base Year Elevations	2015 to 2026	2015 to 2035
Construct Levee Lifts to Achieve Future Year Elevations	2033 to 2072	2035 to 2075

The 1% AEP alternative takes longer to achieve its base year level of risk reduction, because more material is required to build the larger levee, and in some levee reaches, more than one lift is required to achieve the base year (2035) design elevations, as shown in table 6-6.

Table 6-6. Number of Levee Lifts Required to Achieve Base and Future Year Design Elevations

Reach	3% AEP Alternative		1% AEP Alternative	
	Base Year (2035)	Future Year (2085)	Base Year (2035)	Future Year (2085)
Barrier & A	1	3	2	4
B & E	2	3	3	4
F	2	4	3	4
G	2	3	3	3
H & I	2	3	3	4
J	2	4	3	4
K & L	2	3	3	4
Larose C-North	1	4	1	5
Lockport to Larose	1	2	2	3

After 2035, both alternatives would require additional levee lifts to achieve the future year (2085) design elevations because of continuing sea level rise and subsidence. The lift schedule is based on keeping the actual levee elevation at or above the design elevation between 2035 and 2085 allowing for several years of settlement after construction of each lift. The actual level of

risk reduction for each alternative would vary over time depending on the construction timing.

6.4   Environmental Impacts

The following sections briefly describe direct and indirect impacts of the alternatives. A more detailed wetland impact study of direct, indirect, and cumulative environmental impacts of the alternatives and associated mitigation requirements are included in the RPEIS.

6.4.1   Direct Impacts

The major direct impact of the project is the loss of wetlands within the project right-of-way. Construction of the 3% or 1% AEP alternatives would directly impact 3,213 or 4,113 wetland acres (combination of marsh, swamp, and bottomland hardwoods). The direct wetland losses are calculated based on the right-of-way limits (includes the levee footprint, the borrow canal and the widths of the offsets required for both levee stability and borrow pit stability) plus the extents of the proposed mitigation areas. The right-of-way limits are depicted in the RPEIS Mapbook Appendix.

Direct impacts can be determined with a reasonable level of certainty, although the exact number could vary depending on wetland loss prior to construction, which could be caused by sea level rise, subsidence, hurricanes, construction of non-Federal levees along the proposed alignment, or other factors. Since the PAC levee design is based on the Intermediate RSLR scenario, for the purpose of updating project costs and benefits, current estimates of direct wetland impacts for the levee footprints are also based on the Intermediate RSLR scenario. Impacts of the project would slightly decrease as sea level rises because there would be less wetland acres left to be impacted at the time of construction.

The direct impacts of the project on the aquatic habitat of Lake Boudreaux would be temporary. The project would directly impact a narrow band of riparian habitat along the banks of the Bayou Grand Caillou. Direct impacts can be reduced by placement of dredged material in open areas, where practicable.

6.4.2   Indirect Impacts

The authorized alignment builds on existing hydrologic barriers, such as natural ridges, roadbeds, or existing levees that have been built for other purposes such as forced drainage or marsh management. Of the estimated 72 miles of levee originally proposed in the authorized alignment, approximately 15 miles would cross part of the estuaries that are currently open to estuarine exchange. Of the estimated 98 miles of levee in the PAC alternatives, approximately 14 miles would cross open estuaries. The proposed project alternatives include numerous environmental water control structures to allow hydrologic exchange through the levees.

A Systemwide Model was used to determine the impacts of the proposed project on hydrology and salinity. Both the 3% and the 1% AEP alternatives would have similar indirect impacts. The validated model (McAlpin, 2009) for calendar year 2004 was modified to include three system or "plan" configurations and was used to compare the existing without-project conditions to with-project conditions. All three configurations represent operation during non-tropical storm conditions. During tropical storm conditions, all structures would be closed. A comprehensive analysis was performed on the water surface elevations, discharges, and salinity to obtain an approximate indication of the resulting behavior of the system if the proposed changes were to occur.

Structures were modeled as part of two groups: (1) floodgates on navigable waterways and (2) environmental water control structures. Structure groups were modeled in either the open or closed position. The three conditions or "plans" are described below.

Plan 1 - All structures in the open position. The purpose of modeling this condition is to determine the scale of hydrodynamic and salinity impacts of the Morganza project under everyday non-storm conditions. Plan 1 possesses minimal global salinity changes with the largest changes occurring in the marsh area south of Falgout Canal. This area is newly connected to Falgout Canal allowing for a new freshwater inflow to this area, which in turn reduces the salinity (about 3 ppt) with the largest benefit occurring during the winter months and minimal benefit occurring during the summer months. Globally, the salinity changes tend to be less than 1 ppt.

Plan 2 - All floodgates on navigable waterways in the open position and all environmental water control structures in the closed position. This condition would never occur under the current structure operation plan, but was modeled to isolate the effects of the environmental water control structures. The structure operation plan for storm surge is to leave all structures open during everyday non-storm conditions and to close them during high water levels due to storm conditions. Plan 2 has minimal global salinity changes (less than 2 ppt) with some increased salinity possible in local areas newly cutoff by the proposed levee system. Plan 2 has some areas that possess no connection to the remainder of the domain (due to closed environmental water control structures) and therefore would remain stagnant with constant water levels and salinity.

Plan 3 - All structures in the open position with the exception of the HNC structure and lock in the closed position. This condition represents operation of the HNC lock complex for salinity control and would occur whenever certain salinity criteria are met at designated monitoring stations. Plan 3 has noticeable salinity changes along the HNC. Salinity increases along the southern portion (~4 ppt) and lowered north of the HNC structure. The Falgout Canal and Lake Boudreaux areas would be freshened as the closed HNC structure forces the freshwater flow to divert along other avenues, thereby freshening the surrounding areas.

Sensitivity simulations demonstrated the importance of the two GIWW structures. Reducing the size of the western structure reduces the freshwater inflow able to enter the Morganza levee system and thereby increases the salinity in the study area. Conversely, reducing the size of the eastern GIWW at Larose structure reduces the amount of freshwater able to leave the system and therefore decreases the salinity in the study area. While navigational concerns require certain structure sizes for these two areas, those simulations exhibit the type of control the new levee system would provide operators.

Indirect impacts described in the Draft PAC/RPEIS were based on existing sea level conditions because the H&H modeling results were not available for higher sea level rise scenarios. Additionally, there is a high level of uncertainty associated with the rate of sea level rise and how the structures would be operated in the future to compensate for sea level rise. Federal and State agencies that reviewed the Draft PAC/RPEIS noted that in the future there is a potential for adverse indirect and cumulative impacts to wetlands, fisheries, water quality, and navigation due to increased frequency and duration of water control structure closures. In response to these concerns, the Final PAC/RPEIS includes a quantitative analysis of a range of potential indirect impacts to wetlands for the constructible features, and a qualitative analysis of the potential indirect and cumulative impacts for the entire project.

6.5   Potential Induced Flooding

Given the modeling resolution at the time, the potential for induced flooding outside the levee was not identified in the 2002 feasibility report, however, post-Katrina surge modeling results indicate that the project could increase water levels in areas immediately outside the risk reduction system during storm events. When comparing the results of the ADCIRC runs for the without-project to the with-project conditions for existing 1% AEP water levels, the with-project water levels under a storm event are approximately 2 to 3 ft higher as shown in figures 6-3 and 6-4.
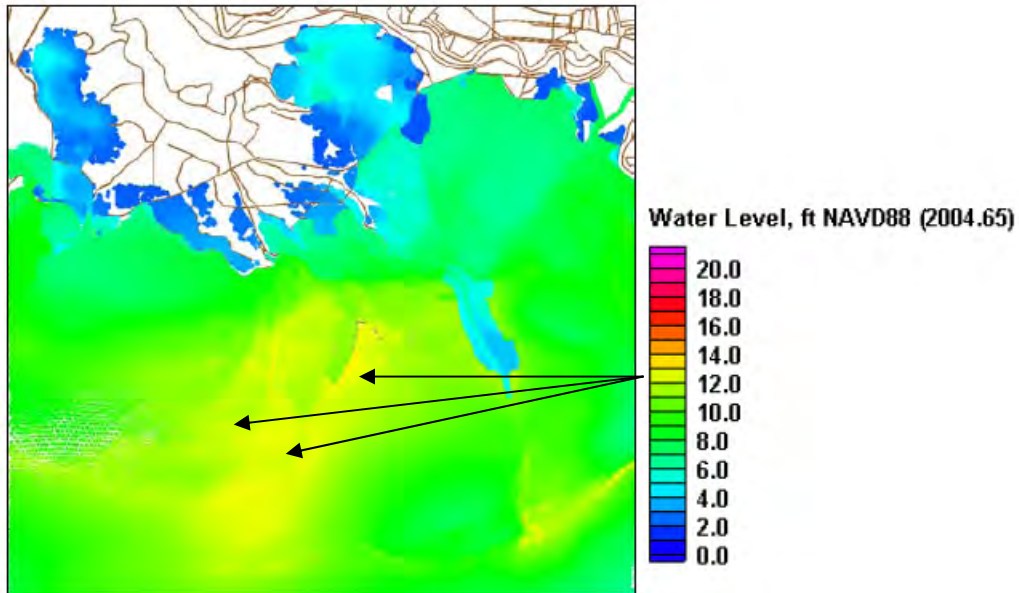
Figure 6-3. 1% AEP Existing Without-Project Water Levels
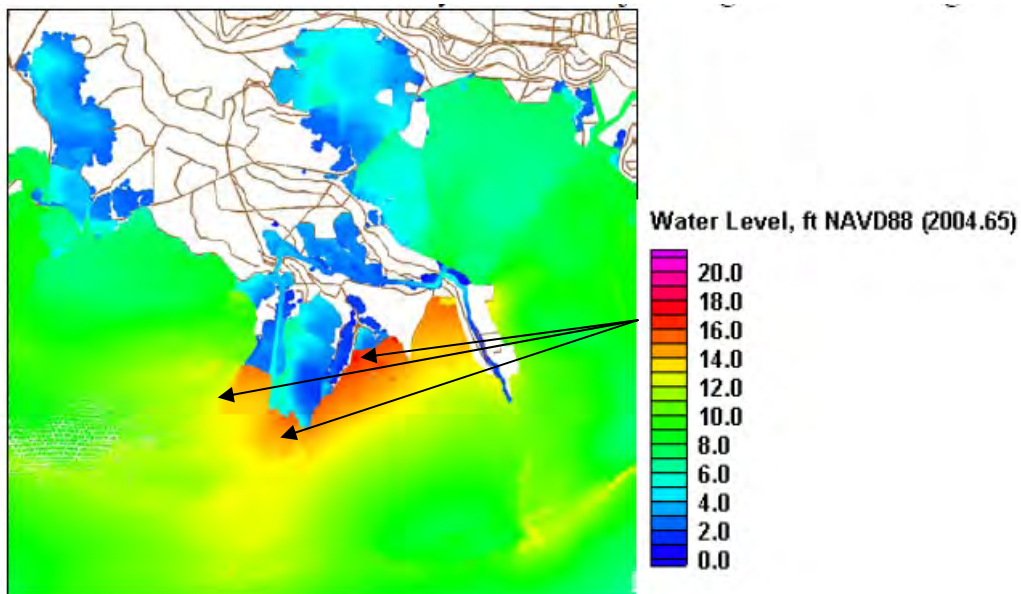
Figure 6-4. 1% AEP Existing With-Project Water Levels

6.5.1   Impacts on Structures Outside of the Risk Reduction System

An indirect impact of the construction of the project is the potential to raise water levels outside the levees by several feet during storm events causing induced flooding to several communities located outside of the proposed levee alignment. These areas include portions of the communities of Gibson, Bayou Dularge, Dulac, and all of Cocodrie and Isle de Jean Charles (see red areas in figure 6-5). Approximately 1,000 structures would remain outside of the Morganza to the Gulf risk reduction system. Although areas outside the levee system would already receive damages under the without-project conditions, the alternatives could increase damages during some events.
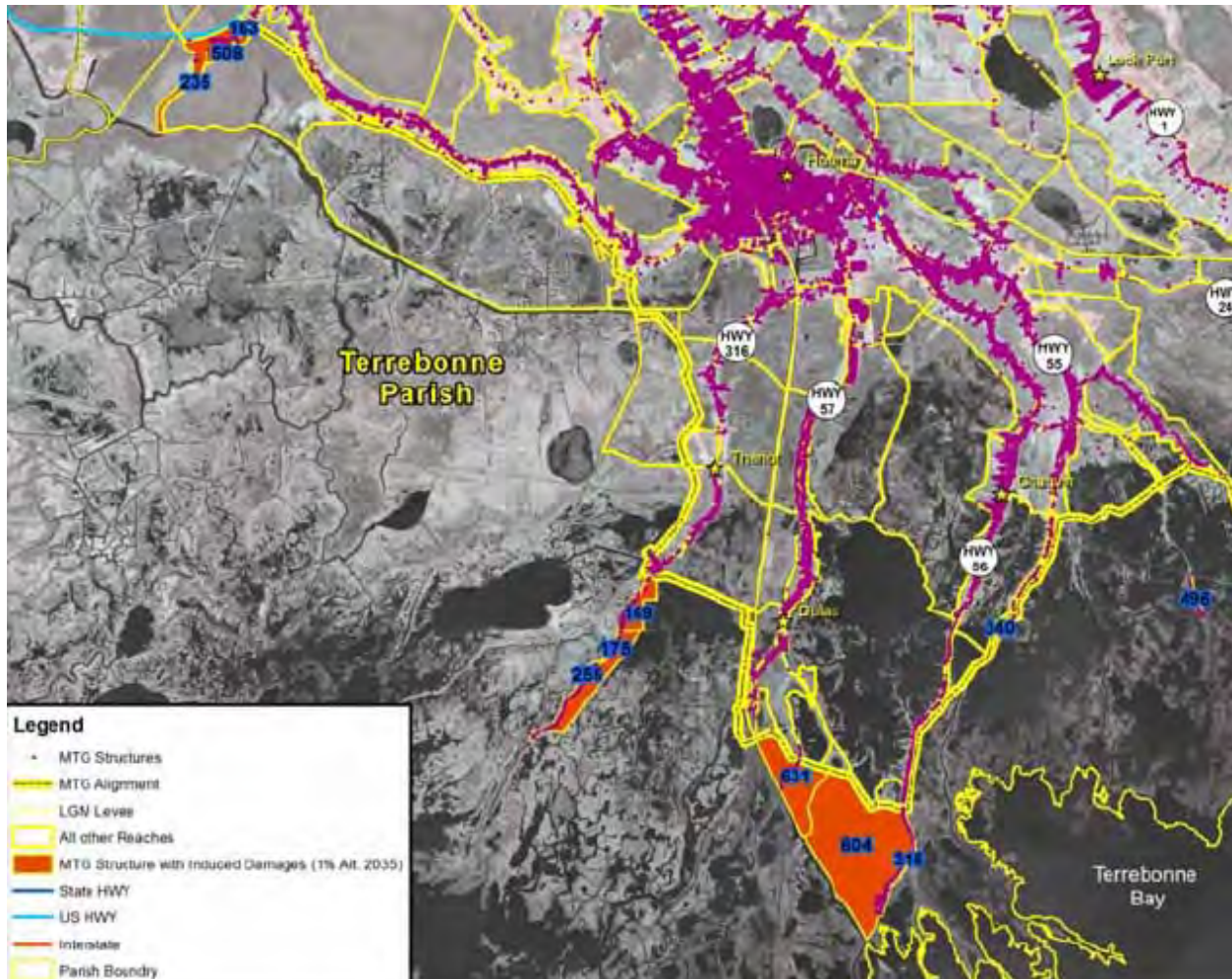
Figure 6-5. Economic Reaches With Possible Induced Damages

Induced damages were initially accounted for in the economic analysis as a negative value in the benefit calculation. For the base condition (2035), induced damages to areas outside the project levee reduced the expected annual benefits by between $14.5 and $22 million. Based on 2035 stage-probabilities, the probability that the target stage (equal to top of levee elevation or elevation where significant damages begin if no levee) would be exceeded within a given year with the Morganza project increases from 18 to 46 percent in Bayou Dularge and from 12 to 68 percent in Isle de Jean Charles. Over a 10-year period, the with- and without-project differences are less significant in these high risk areas. Local levee exceedance risk would increase from 87 percent to 100 percent in Bayou Dularge and from 71 percent to 100 percent in Isle de Jean Charles.

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In order to prevent increased risk to people and structures, which are already located in high risk areas, a preliminary nonstructural plan has been developed. Presently, detailed information regarding the differences in frequency, depth, and duration of the flooding between the future without-project and future with-project conditions is not available. This detailed information typically would be assessed in light of the uses to which the particular land is zoned, and the appropriate mitigation methods, if any, would be implemented to address the effects of the Federal project. To ensure the public is informed of all potential impacts of the project, and to prevent schedule delays, for purposes of this report, the worst case scenario (most expensive option) has been assumed, which would be a 100 percent buy-out of all of the structures in the impacted areas. Should this scenario prove to be the appropriate mitigation method, approximately 2,500 people would need to be relocated to areas behind the Federal levee system.

The benefits and costs of the buy-out plan have been incorporated into the total project cost and benefits analysis. The buy-out plan increases equivalent annual benefits for the 3% and 1% AEP alternatives by approximately $39 million and $57 million, respectively. The total real estate cost associated with this acquisition is estimated to be approximately $305 million as shown in table 6-7. The estimate of acquisition costs for residential structures includes the depreciated value of the improvement, the value of the land, moving costs, differential housing payment, payment of last resort, and administrative costs. The estimate of acquisition for the non-residential structures includes the depreciated value of the improvements, land value, moving costs, reestablishment costs, necessary and reasonable incidental costs, and administrative costs. Real Estate estimated an average Uniform Relocations Assistance cost for residential structures and one for non-residential structures, which was applied to all structures to be acquired.

Table 6-7. Preliminary Nonstructural Compensation Plan

	No. of Structures	Cost
Residential Acquisition	876	$248,712,800
Non-Residential Acquisition	134	$56,402,500
Total	1,010	$305,115,300

The potential induced damages and mitigation for economic damages would be further addressed during detailed design and supplemental NEPA documents. Individual investigation and mitigation for each structure, if appropriate, will be done during PED. Additional factors (height of structures vs. induced stages, type of residential structure, social concerns, etc.) would have to be investigated under PED. Each structure would have to be evaluated under PED to determine if mitigation is appropriate. Further modeling will be performed during PED to determine whether there is a potential taking. A Takings Analysis will be prepared during PED to address this issue; at that time, it will be determined what real estate interest, if any, would be acquired.

Socioeconomic considerations associated with the potential buyouts are described in RPEIS section 5.2.13, and Environmental Justice is discussed in RPEIS section 6.14.8 and RPEIS Appendix J.

6.5.2 Morganza to the Gulf Impacts on the Larose to Golden Meadow Project

Impacts on the existing Larose to Golden Meadow levee that would result from construction of the Morganza to the Gulf levee were evaluated. As previously shown in figures 6-3 and 6-4, stages on the western side of the Larose ring levee will be higher with the Morganza project in place than without Morganza. Morganza does not affect levee sections on the east side of the Larose to Golden Meadow ring levee. Initial Morganza to the Gulf levee lifts may not impact the performance of Larose to Golden Meadow, but future lifts would need to be coordinated to ensure that the Morganza project does not induce damages on Larose and that costs are allocated appropriately between projects.

Costs for potential future levee extensions/lifts and to eliminate potential induced damages on Larose have been added to the Morganza project at this time, however, final determination on cost allocation between the projects has not been made. The 2% AEP Larose levee design elevations "with" and "without" Morganza in place were used to determine the increment of inducements attributable to the Morganza project (table 6-8). The Morganza inducements were quantified for the Larose PAC report using the ADCIRC model. Consistent with LACPR modeling assumptions, the Morganza project was modeled as a non-overtopping levee in the ADCIRC model; therefore, results do not differentiate between the 3% and 1% AEP Morganza alternatives.

Table 6-8. Comparison of Larose Levee Elevations With and Without Morganza

Larose Levee Section	Approx. Section Length (miles)	Larose Authorized Elevations (ft)	2085 2% AEP Larose To Golden Meadow Levee Elevations (with wave berm)		Levee Height Increment* (ft)
			Without Morganza (ft)	With Morganza (ft)	
C-South	3.6	11.4	20	23	3
B-North	6.3	11.4-13.4	20	21.5	1.5
B-South	1.6	13.4	19.5	20.5	1
A-West	4.1	13.4	20	21	1

*For the Larose sections with induced stages, the increment is the difference between 2% AEP elevations with and without Morganza in place. Section C-North is part of both the Larose to Golden Meadow and Morganza to the Gulf projects.
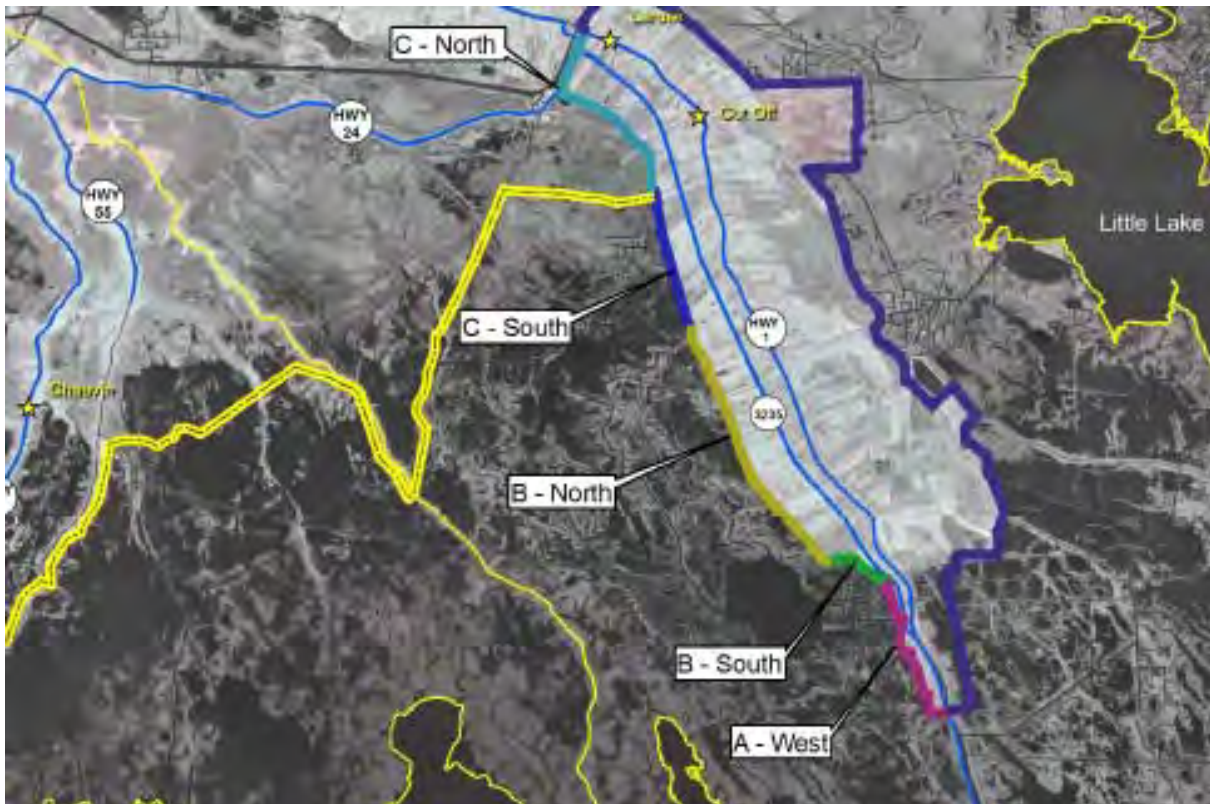
Figure 6-6. Morganza Reach L Tie-in to Western Side of Larose to Golden Meadow Ring Levee

Of the four levee reaches shown in table 6-8 (and in figure 6-6), the largest inducement is on Larose Section C-South. Table 6-9 compares the Larose still water elevations with and without Morganza for the same section. As shown in the table, the increment increases as the event becomes less probable (i.e. the larger the hurricane surge, the larger the inducement). Since the Larose to Golden Meadow project currently provides less than a 2% AEP level of risk reduction, the 3-ft stillwater and corresponding levee elevation increment was used rather than the 4.6 ft increment for Section C-South. The corresponding increments for the other reaches shown in table 6-7 were also used to develop costs.

Table 6-9. Comparison of Larose Still Water Elevations With and Without Morganza

Probability	Stillwater Elevation at Larose Section C-South		Increment (ft)
	Without Morganza (ft NAVD88)	With Morganza (ft NAVD88)	
2% AEP	11.2	14.3	3.1
1% AEP	14.6	18.2	3.6
0.2% AEP	18.3	22.9	4.6

Note that variations in side slopes (1 on 6 for Morganza; 1 on 3 for Larose) create variations in levee design elevations at same location.

If Congress does not re-authorize and fund improvements to the Larose to Golden Meadow ring levee to bring it up to a level of risk reduction comparable to the Morganza project, the costs to the Morganza project to mitigate the induced flooding could be around $125 million.

6.6   Comparison of Alternative Project and OMRR&R Costs

Levees and floodwalls are the largest component of the project costs and the biggest difference in project costs between the alternatives as shown in table 6-10.

Table 6-10. Project First Costs by Civil Works Feature

Work Breakdown Structure No. & Civil Works Feature Description	Estimated Cost($Millions)	
	3% AEP Alternative	**1% AEP Alternative**
02 Relocations	274	291
05 Locks	530	622
06 Fish & Wildlife Facilities	619	941
11 Levees & Floodwalls	2,467	5,351
15 Floodway Control & Diversion Structures	763	1,068
Construction Estimate Totals:	**$4,652**	**$8,273**
01 Lands and Damages	339	355
30 Planning, Engineering & Design	574	1,006
31 Construction Management	385	631
Project Cost Totals:	**$5,950**	**$10,265**

Note: Costs are in October 2012 price levels rounded to the nearest $million. Contingencies vary by category and range from 21 to 30 percent for the 3% AEP plan and from 26 to 35 percent for the 1% AEP plan.

The annual OMRR&R cost estimates used to compare the alternatives are shown in table 6-11. Annual OMRR&R costs are based on grass cutting for earthen levees and routine OMRR&R on all structures within the levee system, such as replacing flap gates, pumps, etc. The OMRR&R costs for the lock structure and all major floodgates (56-ft sector gates and larger) also include dewatering, which is estimated to be done once every 10 to 15 years.

Table 6-11. Comparison of Annual OMRR&R Costs for Alternatives

Alternative	Levee Grass Cutting		Structure OMRR&R ($millions)	Total OMRR&R ($millions)
	(acres)	($millions)		
3% AEP	3,500	$2.2	$3.2	$5.4
1% AEP	6,500	$4.0	$3.2	$7.2

Note: OMRR&R cost estimates are in 2012 dollars, include a 25 percent contingency, and are rounded up to the nearest $100,000. Mitigation OMRR&R costs are not included in this table; those costs are added to the 1% AEP plan only (see table 7-4).

Annual levee OMRR&R costs are based on over 3,500 acres of grass cutting for the 3% AEP alternative and over 6,500 acres of grass cutting for the 1% AEP alternative. Grass cutting is estimated to be needed 16 times per year. In the months of March to October, grass would likely need to be cut every 3 weeks. In the months of November to February, grass would likely need to be cut every 4 to 5 weeks.

6.7   Net Benefits and Benefit-Cost Ratios

Net benefits are based on the following benefit categories: residential and commercial (structure/content/vehicles), industrial (structures/contents), highways, streets, debris removal and cleanup, water supply, boats, and avoided structure raising costs. Costs and benefits for each alternative are shown in table 6-12.

Table 6-12. Cost and Benefit Comparison of PAC Alternatives

(OCTOBER 2012 EFFECTIVE PRICE LEVEL, 3.75% INTEREST RATE)	3% AEP Alternative ($Millions)	1% AEP Alternative ($Millions)
First Costs	$5,950	$10,265
Equivalent Annual W/O Project Damages	906	906
Equivalent Annual With Project Damages (Residual Damages)	447	136
Equivalent Annual Benefits (Damages Reduced)	459	770
Equivalent Annual Benefits During Construction	198	253
Total Equivalent Annual Benefits	656	1,023
Total Annual Costs	442	716
Equivalent Annual Net Benefits	214	307
Benefit-to-Cost Ratio	1.48	1.43

6.8   Other Socioeconomic Benefits/Impacts

The following sections describe additional socioeconomic benefit and impact categories that are not factored into the benefit-cost ratio, including regional economic development, other social effects, and agricultural resources. For more information, refer to the Economic Appendix.

6.8.1   Regional Economic Development

The Regional Economic Development analysis measures the interdependence among industries and workers in an economy. This analysis uses a matrix representation of a region's economy to predict the effect of changes in one industry on others. The greater the interdependence among industry sectors, the larger the multiplier effect on the economy. Changes to government spending are projected to drive new levels of sales, employment, and income for each industry.

The construction of the Morganza to the Gulf levee system would yield significant increases in employment and gross regional product not only to the parishes of Terrebonne and Lafourche, but to Metro New Orleans and beyond. The 3% AEP alternative would generate an estimated $5.8 billion in gross regional product and 85,000 worker-years of labor annually during the construction of the levee system. The 1% AEP alternative would generate an estimated $10.2 billion in gross regional product and 155,000 worker-years of labor.

6.8.2   Other Social Effects

The Other Social Effects analysis considers the potential social ramifications of USACE actions so that decision makers and stakeholders are able to evaluate the social implications of each alternative and choose an alternative that will be judged as complete, effective, and fair.

The No Action Alternative would not reduce storm surge risk to residents of the Morganza evaluation area. Therefore, there is a high potential for extensive hurricane and tropical storm damage to continue occurring in the area. The relative sea level rise that has been taking place in the Morganza to the Gulf area, coupled with the anticipated population growth, is expected to magnify the flooding problems in the future. As a result, subsequent flooding events could cause even more damage to housing units, public facilities, and commercial structures than has previously been experienced. Under this alternative, the area would remain vulnerable to flooding, and long term resiliency would be hampered by the continued local efforts necessary to prepare for, and react to, flood events.

The 1% AEP alternative would result in the greatest potential for reduced flooding in the Morganza study area. This alternative would reduce the damages to housing units, public facilities, and commercial structures for 1 percent annual chance (and more probable) events. The area's social vulnerability would be reduced under this alternative, and thus, the potential for long-term growth and sustainability would be enhanced. Also, under this alternative, the area would be at a reduced probability of incurring the costs associated with clean-up, debris removal, and building and infrastructure repair as a result of flood events.

The 3% AEP alternative would also reduce the risk of flooding in the Morganza study area. However, this alternative would only provide risk reduction for 3 percent annual chance (and more probable) events. The area would still experience damage associated with less probable (more damaging) events.

Socioeconomic considerations associated with the potential buyouts are described in RPEIS section 5.2.13, and Environmental Justice is discussed in RPEIS section 6.14.8 and RPEIS Appendix J.

6.8.3   Agricultural Benefits

Agricultural benefits due to inundation reduction have been quantified as a number of acres rather than a dollar value. Between 2035 and 2085, the average annual agricultural benefits range from approximately 610 to 1,680 acres for the 3% AEP alternative and 770 to 1,860 acres for the 1% AEP alternative. Even if a high estimate of the net revenue generated by an average annual acre were used in the analysis, the total agricultural benefits would only equal approximately 1 percent of the total inundation reduction benefits to structures, contents, and vehicles for each of the project alternatives. Thus, estimates of agricultural benefits were not included in the net benefit computations.

6.9   Relative Sea Level Rise Scenario Sensitivity Analysis

Since this project will be constructed over 40 or more years, there will be opportunities to re-evaluate relative sea level rise. If over time it appears that the actual RSLR rate is higher than expected, additional lifts can be added to levees, and the levee alignment may need to be further extended to the west and to the east. If RSLR rates are lower than expected, then final levee lifts will not need to be constructed, although structures may remain overbuilt. For example, to achieve the 1% AEP levee elevations at 2085 for the Low RSLR scenario, levee reach elevations

could be reduced by 1 to 1.5 ft compared to the Intermediate RSLR elevations. For the High RSLR scenario, levee reach elevations would have to be increased by 3 to 6.5 ft.

Engineering data required for a complete economic analysis of the High and Low RSLR scenarios have not been generated; however, parametric designs and costs were used to compare the volumes and costs for the RSLR scenarios (not including levee extensions that may be needed for the High RSLR scenario). Table 6-13 summarizes the volume and cost increases or decreases. Cross-sectional areas were calculated for each of the levee reaches. The length of each reach was used to calculate the volumes of the levee reaches for each RSLR value. For each alternative, the Low RSLR volume was subtracted from the Intermediate RSLR volume to get the volume reduction, and the Intermediate RSLR volume was subtracted from the High RSLR volume to get the volume increase.

Table 6-13. Potential Change in Volume and Costs for Relative Sea Level Rise Scenarios

Alternative	RSLR Scenario	Average Cross-Sectional Area (sq.ft.)	Volume Difference Compared to Intermediate RSLR (cu.yd.)	Cost Difference Compared to Intermediate RSLR
1% AEP	Low (1.7 ft)	6,200	-3,600,000	-$98,500,000
	Intermediate (2.4 ft)	6,400	N/A	N/A
	High (4.8 ft)	9,000	49,100,000	$1,300,000,000
3% AEP	Low (1.7 ft)	2,300	-2,200,000	-$58,200,000
	Intermediate (2.4 ft)	2,400	N/A	N/A
	High (4.8 ft)	3,200	13,900,000	$376,500,000

If the High RSLR future scenario becomes as reality, the potential first cost increases to the project are estimated at $1.3 billion for the 1% AEP alternative and $377 million for the 3% AEP alternative. These first cost increases translate to 13 and 6 percent average annual cost increases. If the Low RSLR future scenario occurs, the potential savings to project first costs are $99 million for the 1% AEP alternative and $58 million for the 3% AEP alternative. These first cost decreases translate to approximately 1 percent decreases in average annual costs.

Stage probability functions for the alternative sea level rise scenarios have not been developed, making it impossible to precisely compute damages and benefits. In order to approximate damages and benefits for the Low RSLR scenario, the HEC-FDA modeled Intermediate RSLR damages for 2035 were held constant through the end of the period of analysis (as a "surrogate" for the Low RSLR scenario). The effect of this assumption is to allow no additional sea level rise beyond 2035. While the Intermediate RSLR increase up to 2035 would be greater than that associated with the Low RSLR scenario (stages for the 1% AEP would be approximately 0.2 ft higher on average), the amount of Intermediate RSLR by 2035 would be less than that associated with the 2085 Low RSLR scenario (stages for the 1% AEP would be approximately 1 ft lower on average). In general, over the period of analysis it would be appropriate to characterize the "surrogate" Low RSLR scenario as representing less increase than the Intermediate sea level rise scenario. It is believed that lower increases would be associated with lower without-project damages and lower benefits, therefore the without-project damages and benefits for the

"surrogate" Low RSLR scenario would be lower than the Intermediate sea level rise scenario. Using this conservative estimate of benefits for Low RSLR (a reduction of 16 percent for both the 1% and 3% AEP project alternatives compared to the Intermediate RSLR scenario), and adjusted annual costs as previously described, would result in benefit-cost ratios of approximately 1.2 to 1 and 1.3 to 1, respectively, for the 1% and 3% AEP project alternatives.

Approximation of the damages and benefits for the High RLSR scenario is more problematic given available data. It is expected that both damages and benefits would be higher with the High RSLR scenario. Consequently, Intermediate RSLR benefits are assumed as a conservative estimate of the High RSLR scenario. Using this conservative estimate of benefits for high sea level rise, and adjusted annual costs as previously described, would result in benefit-cost ratios of approximately 1.3 to 1 and 1.4 to 1, respectively, for the 1% and 3% AEP project alternatives.

6.10 Summary of Alternatives Comparison

As compared to the 3% AEP alternative, the 1% AEP alternative has:

- **Higher net benefits**. Both plans have positive benefit-cost ratios, but net benefits (excess benefits over costs) for the 1% AEP alternative plan are higher than the net benefits of the 3% AEP alternative plan.

- **Lower residual risk**. The 3% AEP alternative has a higher probability of overtopping and/or levee breaches than the 1% AEP alternative and therefore has higher residual damages than the 1% AEP alternative.

- **Greater flexibility**. The 1% AEP structures would be constructed at higher elevations than the 3% AEP structures to accommodate higher RSLR levels in the future. Although the total cost of the 1% AEP alternative is significantly higher than the 3% AEP alternative, not all funding and expenditures are required up front since earthen levees would be constructed in multiple lifts.

7   Description of the 1% AEP Post-Authorization Plan

The 1% AEP alternative has been identified the plan with the greatest net benefits of the two alternatives. Section 7 describes the 1% AEP alternative in more detail, however, much of this section would also apply to the 3% AEP alternative. The 3% AEP alternative has the same features as the 1% AEP alternative (i.e. same levee alignment, same number and types of structures). The primary differences in these two alternatives are the levee and structure dimensions resulting in different levels of risk reduction, direct impacts, net benefits, and costs.

7.1   Plan Description

As a comprehensive approach to reduce hurricane and storm risk in portions of Terrebonne and Lafourche Parishes, the 1% AEP post-authorization plan is a hurricane and storm damage reduction levee system designed to provide 1% AEP surge risk reduction based on post-Hurricane Katrina criteria. The levee system consists of 98 miles of grass-covered earthen levees tying into US 90 near the town of Gibson in Terrebonne Parish and Hwy 1 near Lockport, LA in Lafourche Parish (see the project map in Attachment 1). Levee elevations for base conditions (2035) range from 10.5 to 24 ft NAVD88, and final levee elevations (2085) range from 15 to 26.5 ft NAVD88 with final levee widths from 282 to 725 ft.

Structures include 1 lock, 22 floodgates on navigable waterways (3 on Federally-maintained navigation channels and 19 on other canals and bayous), 23 environmental water control structures, 9 road gates, and fronting protection for 4 existing pumping stations. Structures on Federally maintained navigation channels include the HNC lock and floodgate (250-ft sector gate) and two floodgates on the GIWW (125-ft sector gate east of Bayou Lafourche and 125-ft sector gate west of Houma). Fourteen 56-ft sector gates and five 20- to 30-ft stop log gates are located on various waterways that cross the levee system. Structure elevations range from 17 to 33 ft NAVD88.

As a sponsor funded additional work item, the HNC lock complex sill depth may be deepened from -18 to -23 ft NAVD88 in anticipation of future deepening of the HNC.

Levees would be covered in grass to increase resilience in the case of wave overtopping. All of the transitions between levees and floodwalls would be armored with reinforced concrete scour protection.

The post-authorization project would directly impact 4,113 acres of wetlands. Mitigation plan requirements are discussed in section 7.7 of the PAC report and in the RPEIS.

7.2   HNC Lock Complex

The largest structure in the Morganza to the Gulf project is the HNC lock complex, which consists of a 110-ft wide by 800-ft long lock with an adjacent 250-ft wide floodgate. The lock complex would have a 30.5 ft NAVD88 top elevation and either a -18.0 ft NAVD88 sill elevation consistent with current authorized channel depth, or a -23 ft NAVD88 sill elevation to account for a potential future channel depth (if funded as an additional work item by the non-Federal sponsor). When a storm approaches from the Gulf of Mexico, the lock and floodgate are closed, shutting the canal off to storm surge. During low water periods, the floodgate is closed, and marine traffic is locked through, reducing the quantity of salt water intrusion north of the project site.

7.2.1   Description of the HNC Lock Complex

Figure 7-1 is a conceptual drawing of the HNC lock complex. Features shown in figure 7-1 appear in bold in the following text:

- The HNC lock complex is generally oriented in a north-south direction approximately 3,000 ft south of the intersection of the HNC with Bayou Grand Caillou and is located in a **bypass channel** adjacent to the HNC on its west side.

- The lock structure consists of two lock gate monoliths (**gulf side lock gates and inland lock gates**), which house two sets of sector gates, and five U-frame **lock chamber** monoliths. A floodgate monolith adjoins the **gulf side lock gate** monolith and houses a **sector gate**, which is separated from the **gulf side lock gates** to the west by 59 ft. The five lock monoliths and the floodgate monolith are made of cast-in-place, reinforced concrete, and are pile supported.

- T-walls extend from both sides of the lock and floodgate to tie into the proposed Morganza to the Gulf hurricane system at **levee reach F-1** to the west and **levee reach G-1** to the east, transitioning to levee elevations +23.5 and +24 ft NAVD88 (in year 2085), respectively. Within the T-walls, there are a total of ten 5-ft wide by 10-ft high sluice gates—four between the floodgate and Levee Reach F-1, two between the lock and floodgate, and four between the lock chamber and closure dam.

- Guide walls and guard walls with steel sheet pile dolphin cells help align marine traffic to the lock and floodgate on both the north and south approaches.

- The lock and floodgate sector gates are normally remotely operated from a single **control house**, which is located at the south end of the lock chamber, on its east side. Below the **control house** is a visitor's platform from where the lock operations can be observed. On the east side of the lock, there is an earthen reservation area that is built up above existing grade to +8 ft NAVD88. The lock can be accessed from this reservation area by either of two bridges that connect the lock gate monoliths to the reservation or by a driveway on the tie-in T-walls. Access to the floodgate is from the gulf side lock gate monolith or by boat. The reservation area contains an office building, an operation and maintenance building, a generator building, a perimeter road to access both ends of the lock, and a boat launch and dock. The maintenance dewatering structural systems for the lock and floodgate monoliths are stored on a nearby platform. Site utilities (electricity and potable water) are routed beneath the HNC and tie into the reservation area. Sewage is treated by a package unit on site.

- A **closure dam** closes the existing HNC channel near the confluence of Bayou Platte and the HNC. The dam is underlain by a grid of soil-cement columns installed with the dry method of deep-soil mixing. The closure dam is a rock dam constructed to + 8 ft NAVD88 with a T-wall on top that provides protection to +30.5 ft NAVD88.

The project would be constructed "in-the-dry," inside an earthen cofferdam excavation. Once the major project features are complete, the site would be re-watered and the last sections of the bypass channel dredged. Marine traffic would be rerouted through the lock and floodgate, and the HNC would be permanently closed by the rock dam.

Figure 7-1. Conceptual Drawing of HNC Lock Complex

7.2.2 Deepening the Lock Complex as a Sponsor Funded Additional Work Item

The non-Federal sponsor has requested that, in lieu of constructing the HNC lock sill to an elevation of -18 ft NAVD88 as would be required by the currently authorized HNC project depth, the USACE construct the lock sill to an elevation of -23 ft NAVD88. Although this additional depth is not required for implementation of the Morganza to the Gulf project, the USACE could consider undertaking this additional work on the non-Federal sponsor's behalf if the non-Federal sponsor pays all incremental costs of this additional work, currently estimated at $32 million as shown in table 7-1. The USACE received a letter from the non-Federal sponsor on 18 May 2012 agreeing to pay for the full incremental cost of the deeper sill, as well as any incremental OMRR&R costs.

Table 7-1. Benefits and Costs for Authorized vs. Locally Preferred HNC Lock Complex Sill Depths

Feature	HNC Lock Complex Design Options Based on HNC Depths		Difference With and Without Sponsor Funded Additional Work Item
	As Currently Authorized	As Possibly Authorized in the Future	
Channel Depth	-15 ft	-20 ft	5 ft deeper
HNC Lock Complex Sill Elevation (3 ft below channel depth to allow for over-dredge)	-18 ft	-23 ft	
HNC Lock Complex Wall Elevation	30.5 ft	30.5 ft	Same
HNC Lock Complex Cost*	$622 million	$654 million	$32 million

*Costs are in October 2012 price levels. October 2012 cost estimates were escalated by a factor of 1.011 (1.1%) from originally estimated costs, which were in October 2011 price levels; costs include contingencies; the total cost increment could be higher than the HNC lock complex cost increment if the deeper sill requires higher PED and S&A costs.

The sponsor funded additional work item does not change the Morganza to the Gulf project's hurricane and storm damage risk reduction benefits, nor would it substantially change environmental impacts. The benefit of the deeper sill is that it would have the potential to meet future navigational needs for increases in channel depths.

The deeper sill adds approximately $32 million, which is a minor increase to the total project cost, so it would only negligibly lower the Morganza PAC benefit-cost ratio, and the project is still economically justified. The sponsor funded additional work item would prevent having to replace the HNC lock complex in the event that the HNC is deepened in the future. Conversely, if the lock sill elevation is built to -18 ft NAVD88 as part of the Federal plan, and the HNC is later deepened, most of the lock complex would need to be demolished and replaced. A detailed cost estimate to demolish and replace the -18 ft NAVD88 sill depth lock with a -23 ft NAVD88 sill depth lock has not been developed, but it is estimated that a fairly substantial portion of the original project cost (estimated 50 to 75 percent, or approximately $250 to $350 million) would be required to build the deeper lock complex. There would also be demolition costs to tear out portions of the old -18 ft NAVD88 sill HNC lock complex.

The OMRR&R of the HNC lock complex and the GIWW floodgate features that provide for inland waterway transportation is a Federal responsibility; however, the non-Federal sponsor will be responsible for any incremental OMRR&R costs associated with a deeper sill, which may require higher dredging costs. At this time, the annual OMRR&R costs in the PAC report are not at a level of detail to be able to differentiate between the annual OMRR&R cost difference with and without the deeper sill. Incremental OMRR&R costs will be developed during the follow on PED phase.

7.3　Other Structures

In addition to the HNC lock complex, dozens of structures are located throughout the levee system including floodgates on navigable waterways, environmental water control structures, road/railroad gates, and fronting protection for existing pumping stations.

7.3.1　Floodgates on Navigable Waterways

In addition to the HNC lock and 250-ft sector gate described in the previous section, the post-authorization Morganza to the Gulf project includes 21 other floodgates on navigable waterways ranging in size from 20-ft stop-log gates to 125-ft sector gates. Figure 7-2 is a conceptual drawing of a 56-ft sector gate and figure 7-3 is an example site plan for the same type of floodgate.
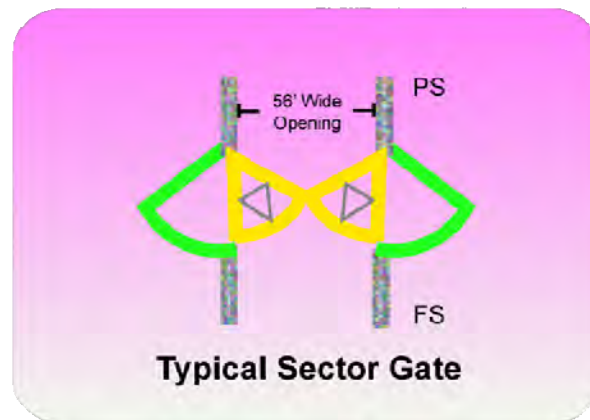
Figure 7-2. Conceptual Drawing of a Typical Sector Gate
PS = Protected Side; FS = Flood Side

Structures were designed to provide enough width for vessel passage and enough flow cross sectional area at the gate sites so that the constricted flow velocities would neither exceed the navigational criteria nor result in channel scouring problems. The assumption for the PAC report is that all currently navigable waterways must remain so after the project is put in place, however, the number/sizes of gates may be able to be reduced during PED with additional data on navigation.
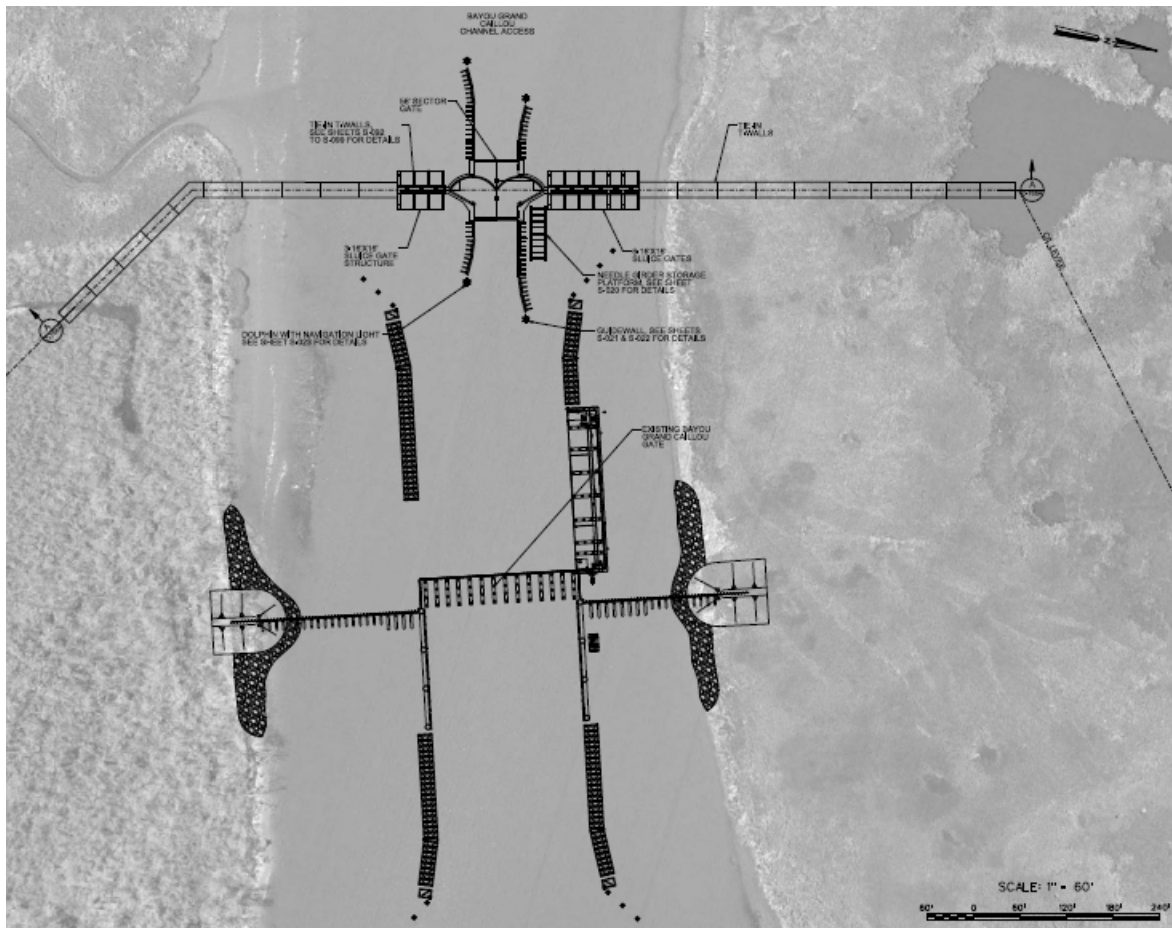
Figure 7-3. Example 56-ft Sector Gate Site Plan (Bayou Grand Caillou)

7.3.2 Environmental Control Structures

Environmental control structures are located at 23 locations within the levee system. The purpose of the environmental control structures is to provide hurricane and storm damage reduction during storm conditions and to match existing drainage patterns during non-storm conditions. Environmental control structures consist of box culverts and sluice gates allowing tidal exchange. The number of 6-ft by 6-ft or 5-ft by 10-ft culverts at each location varies from one to nine. The typical configuration is six 6-ft by 6-ft box culverts as noted in the conceptual drawing in figure 7-4.
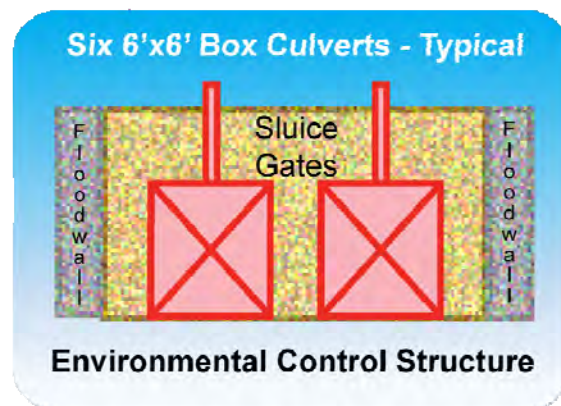
Figure 7-4. Conceptual Drawing of a Typical Environmental Control Structure

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Refer to the Attachment 1 project map for the environmental control structure locations and to Table 146 in the Engineering Appendix for the number and size of structures at each location.

7.3.3 Pump Stations

Fronting protection is provided for several existing pumping stations, including the Madison, Pointe aux Chenes, Elliot Jones (Bayou Black), and Hanson Canal pump stations. Features associated with the construction of fronting protection include T-walls and butterfly gate valves as shown in figure 7-5. All fronting protections would be constructed on the flood side of the existing protection. Based on site visits, the discharge pipes extend far enough that additional pipes are not needed. Butterfly valves would be opened to allow pumping discharge for interior drainage or closed to prevent backflow during storm conditions.
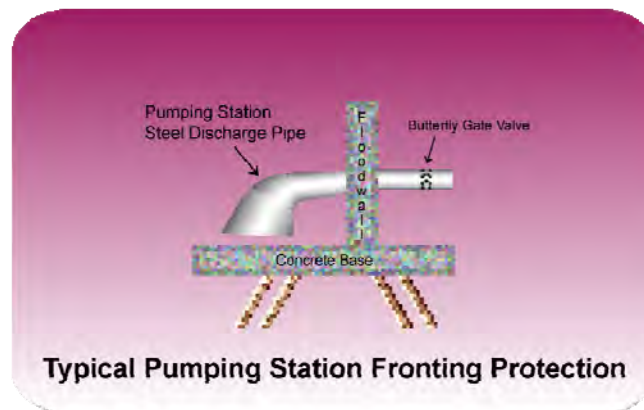
Figure 7-5. Conceptual Drawing of Typical Pumping Station Fronting Protection

7.3.4 Roadway Gates

The levee alignment impacts several state and parish highways and roads and the Union Pacific Railroad. At the points of intersection with the levee, the options are either to construct ramps over the levee or permanent floodgates. The decision to use a swing gate or ramp is based on a variety of factors, including cost, relocations, existing structures, environmental impacts, and access requirements (e.g. consideration of evacuation routes). The use of gates versus ramps at each location will be revisited during PED as designs are further refined. Roadway structures would remain open except for times of anticipated flooding, which at that time they would be closed and remain closed until the flood threat subsided. Roadways affected by this project include LA Highway 1, LA Highway 25, LA Highway 182 (Bayou Black Drive), LA Highway 315 (Bayou Dularge Road), Four Point Road, LA Highway 56, LA Highway 55, LA Highway 665 (Point aux Chenes Road), LA Highway 24, LA Highway 3235, and a private road on NAFTA property. All roadways and the railroad would have a swing gate, except LA 182 which would have a ramp. For LA 182, alternate access for locals will need to be made available during the construction of the earthen ramp, which will need to be raised each time the levee is raised. The features associated with construction of each roadway gate structure are a steel swing gate, concrete monolith, and traffic control devices.

7.4   Structure Operation Plans

In response to comments on the Draft PAC/RPEIS from Federal and State resource agencies, the USACE, non-Federal sponsors, and Habitat Evaluation Team modified preliminary structure operation plans to clarify opening and closure triggers for each structure. The preliminary operation plan assumptions for all water control structures, which are included in section 4.3.8 of the RPEIS, were developed for the sole purpose of estimating indirect impacts for the constructible features. Detailed operation plans for each structure will be prepared during PED and in future NEPA documents for the programmatic structures.

The number of times closure occurs each year depends on the frequency of tropical storm events and location of the structures. Table 7-2 summarizes recent historical closures and frequency of closure by location and year. Most closure durations were for less than 48 hours. The longest closure was during Tropical Storm Alex in 2010 when the Humble Canal and Little Caillou floodgates were closed 10 to 12 days.

Table 7-2. Number of Gate Closures Between 2001 and 2012

Gate	Total Number of Closures from 2001 to 2012	Maximum Number of Closures Per Year
Bayou Terrebonne	45	8
Little (Petit) Caillou	29	9
Lower Bayou Dularge	5	2
Upper Little Caillou Barge	4	1
Humble Canal	9	5

Source: Terrebonne Levee and Conservation District

Under future conditions, structure closure frequencies could increase if closure triggers are not adjusted to account for sea level rise. For example, under existing conditions, if the HNC floodgate were to be closed based on a 2.5 ft stage (for illustrative purposes only), closure could occur approximately 1.5 days per year. If the 2.5 ft trigger remained the same through 2085,

- Low RSLR could require closure 5 days per year by 2035, and 168 days per year by 2085.

- Intermediate RSLR could require closure for 15 days per year by 2035, and 354 days per year by 2085.

- High RSLR could require closure for 24 days per year in 2035, and 365 days per year in 2085.

Refer to section 10.3.3, Indirect Environmental Impact and Mitigation Plan Uncertainties, for a discussion of unresolved issues associated with operating structures under future sea level conditions.

7.5   Real Estate Plan

The Non-Federal Sponsor has responsibility for and receives credit for obtaining Lands, Easements, Rights-of-Way, Relocations, and Disposal Areas (LERRDs). The Real Estate Plan, which is provided as an appendix to this report, presents the real estate requirements and costs for the PAC Report, in accordance with E.R. 405-1-12 (Chapter 12).

7.5.1   Lands, Easements, and Rights-of-Way

The majority of the acreage affected by the project consists of marsh or wooded wetlands. Other lands impacted include woodland, agricultural (cane land), industrial waterfront on the GIWW, residential waterfront lots at the community of Waterproof, and mixed-use waterfront lots on Bayou Petit Caillou and Bayou Dularge ("Mixed-use" refers to recreational "camps" or residential waterfront lots). The Right-of-Way in Lafourche Parish consists mostly of marsh and open water. The majority of the staging areas for construction of this project would be located within the Right-of-Way for the levee footprint or existing Right-of-Way.

The Real Estate Plan references LERRDs for three different types of acquisitions: 1) The LERRDs required for construction and OMRR&R of the project (affecting approximately 580 landowners), 2) the LERRDs required for future lift borrow (affecting approximately 325 landowners), and 3) the LERRDs required in areas affected by induced flooding (affecting approximately 1,010 landowners).

Estates required for the project would include Fee, Excluding Minerals (With Restriction on Use of Surface), perpetual Flood Protection Levee Easements, Temporary Work Area Easements, and Temporary Access Easements (Non-Material Deviation from Standard Estate).

7.5.2   Utility Relocations

Facility/Utility Relocation data was collected and detailed by the USACE New Orleans District, Engineering Division, Design Services Branch Relocations Team. A separate Relocations Report, containing relocation costs, was submitted as a reference to the Engineering Appendix of the PAC Report.

There are multiple pipeline crossings throughout the project. Some pipeline owners have directionally drilled pipeline crossings for currently planned local levees and with anticipation of this project in mind. During the next phase of design, as more information becomes available, alternatives for pipeline crossings will be further evaluated. Directional drilling is the preferred alternative, but many times more expensive than other methods. Other alternatives may include bridging over the levee or laying the pipe over the levee slope and crown support with temporary saddles or short piles until it can be incorporated into future lifts that keep the pipe above the storm still water level. Abandoned pipelines should be removed, but in some cases may be grouted and capped on both ends within the levee right-of-way. Oil and gas wells would not be relocated; instead, the levee alignment would be changed, or T-walls used, during the project Plans and Specifications phase to avoid them.

In addition to underground lines, the proposed highway gate closures and ramp would require relocation of overhead lines (poles, electric, cable, telephone). For the floodwalls, it includes passing the gas and water lines through a sleeve. For the ramp at LA 182, it includes directionally drilling the gas and water lines.

7.6   Borrow Materials

Levees will be constructed using a combination of sidecast and hauled-in borrow materials. Adjacent sidecast was used for the pre-load section only. Haul in scenarios were used for the initial (first) lift and projected subsequent lifts. Borrow pits are oversized to offset the potential for encountering organics, expected losses, etc.

7.7   Mitigation Plan

USACE policy is to ensure that adverse impacts to significant resources have been avoided or minimized to the extent practicable and that remaining, unavoidable impacts have been compensated to the extent justified. The appropriate application of mitigation is to formulate an alternative that first avoids, then minimizes, and lastly, compensates for unavoidable adverse impacts. In the development of the project, features that were incorporated to avoid and minimize potential adverse environmental effects included, where practical, the placement of levees at locations that would avoid or minimize effects on wetlands or other significant features of the project area. The sizing of floodgates and environmental control structures was done to minimize indirect impact to wetlands in the interior of the system.

7.7.1   Mitigation Determinations

An interagency Habitat Evaluation Team was formed to use Wetland Value Assessment (WVA) methodology to assess the quality of wetlands of the area, make a determination of the effects various aspects of the project on future conditions, and calculate the amount of mitigation required to compensate for impacts caused by the constructible features of the project. A description of the WVA methodology, analysis, and assumptions made by the HET may be found in Appendix F of the RPEIS.

Environmental mitigation features for the project authorized in WRDA 2007 included creation of 1,352 acres of marsh habitat. At this time, mitigation features for the post-authorization project have only been identified for the constructible features (i.e. levee reaches F-1, F-2, and G-1; the HNC lock complex; and the Bayou Grand Caillou floodgate). Mitigation for direct and indirect impacts of the constructible features includes approximately 394 acres intermediate marsh restoration, 358 acres brackish marsh restoration, and 883 acres saline marsh restoration. For the remaining programmatic features, mitigation costs and land requirements were estimated, but the exact number of acres will be determined in the future as more specific designs are completed and impacts are assessed in future supplemental NEPA documents.

Mitigation is discussed in more detail in section 6.19 of the RPEIS. Also refer to section 10.3.3 of the PAC for unresolved issues related to Indirect Environmental Impacts and Mitigation Plan Uncertainties.

7.7.2   Compensatory Mitigation Alternatives

For the constructible features, compensatory mitigation alternatives considered the purchase of mitigation credits from an approved mitigation bank and USACE constructed in-kind mitigation. The WRDA of 2007 requires that the USACE first consider using commercial mitigation banks to provide compensation for environmental impacts to wetlands. The USACE determined that the use of mitigation banks for the constructible features was not feasible for the following reasons: (1) No mitigation banks for intermediate, brackish, or saline marsh are available in the vicinity of the project area; (2) project structures would be constructed using clay material dredged from areas adjacent to the proposed structures; however, the overburden consists of

approximately 5 ft of organic material unsuitable for use as construction material. To reduce project costs, the USACE proposes to use this organic material to create/restore coastal marsh habitat to compensate for losses resulting from the project. If the amount of the overburden material is insufficient, additional material would be obtained from offsite sources. There are two mitigation banks in the area that would be considered for the programmatic features. They may potentially provide credits for fresh marsh, cypress/tupelo gum swamp, and bottomland hardwoods.

7.7.3   Proposed Mitigation Actions

The proposed mitigation actions include construction of marsh to restore eroded and subsided wetlands in the project area. The mitigation work plan includes the following features, which are further described in the RPEIS:

- **Containment Dikes:** Dikes would be used at each marsh restoration site to contain placed earthen materials until the materials have consolidated and wetland vegetation has become established.

- **Dike Degradation:** The dikes around mitigation sites and cells would be designed to slowly deteriorate and subside to the level of the adjacent marsh substrate, thereby promoting the tidal exchange of water.

- **Target Elevations:** The target elevations of placed and consolidated fill at each site would be determined through geotechnical analyses. The final result of the material placement would be a combination of wetlands and shallow open water habitat within the site. Slurry would be allowed to overflow over existing emergent marsh vegetation within the proposed disposal areas, but would not be allowed to exceed a height of about one foot above the existing marsh elevation.

- **Vegetation:** The establishment of vegetation on marsh areas would provide stability and reduce erosion.

- **Access Corridors:** Access corridors to mitigation sites would be a maximum of about 200 feet wide and would cross over uplands, wetlands, and shallow open water as necessary.

- **Flotation Access Corridors:** Channels would be excavated as needed in shallow open water areas to allow construction equipment to access sites.

- **Existing Levee Access Corridors:** If construction equipment and discharge pipelines are placed across or along the crown of existing levees in the project vicinity, the levees may be refurbished using borrow material from adjacent shallow open water to facilitate their use as access corridors for construction equipment and discharge pipelines.

- **Staging Areas:** The construction or designation of staging areas may be necessary for construction equipment and for the unloading of pipeline and other equipment necessary to perform disposal operations.

- **Board Roads:** Temporary board roads may be constructed along access corridor alignments and staging areas wherever emergent marsh exists. Board roads would be removed when work is completed.

7.7.4 Mitigation OMRR&R

The non-Federal sponsor would be responsible for OMRR&R of functional portions of work as they are completed. On a cost-shared basis, the USACE would monitor completed mitigation to determine whether additional construction, invasive species control, and/or planting are necessary to achieve mitigation success. The USACE would undertake additional actions necessary to achieve mitigation success in accordance with cost sharing applicable to the project and subject to the availability of funds. Once the USACE determines that the mitigation has achieved initial success criteria, monitoring would be performed by the non-Federal sponsor as part of its OMRR&R obligations. If, after meeting initial success criteria, the mitigation fails to meet its intermediate and/or long-term ecological success criteria, the USACE would consult with other agencies and the non-Federal sponsor to determine whether operational changes would be sufficient to achieve ecological success criteria. If instead, structural changes are deemed necessary to achieve ecological success, the USACE would evaluate and take appropriate actions, subject to cost sharing requirements, availability of funding, and current budgetary and other guidance; as well as coordination with the local non-Federal sponsor and resource agencies.

7.8   Plan Costs

Table 7-3 shows the cost breakout of the plan features and/or categories.

Table 7-3. Costs by Civil Works Feature

Work Breakdown Structure No. & Civil Works Feature Description	Estimated Cost ($Millions)
02 Relocations	291
05 Locks	622
06 Fish & Wildlife Facilities	941
11 Levees & Floodwalls	5,351
15 Floodway Control & Diversion Structures	1,068
Construction Estimate Totals:	**$8,273**
01 Lands and Damages	355
30 Planning, Engineering & Design	1,006
31 Construction Management	631
Project Cost Totals:	**$10,265**

Note: Costs are in October 2012 prices and rounded to the nearest $million. Lock costs do not include sponsor funded additional work item.

The **02 Relocations** category refers to utility relocations described in section 7.5.2.

The **05 Locks** category includes the cost of the HNC lock complex as described in section 7.2.1.

The **06 Fish & Wildlife Facilities** category includes both the cost of the environmental control structures described in section 7.3.2 and a portion of the wetland mitigation costs. Of the $941 million shown in table 7-3, approximately $513 million is for the environmental control structures and $428 million is for the remaining compensatory wetland mitigation after all of the levee overburden is used for mitigation. The top 5 ft of borrow for first lift levees that is not suitable for levee building (approximately 12,305,000 cubic yards) would be available for marsh creation. Using Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) program marsh creation assumptions and water depths of between -2.25 and -3.5 (varies by reach), the 12,305,000 cubic yards could create approximately 1,175 acres of marsh. Since more

than 1,175 acres of marsh mitigation is needed, the 1,175 acres were subtracted from the total marsh acres needed (approximately 4,000 acres) to calculate the remaining mitigation costs.

The **11 Levees & Floodwalls** category, which is the largest cost component, includes all levees, and floodwalls (i.e. T-walls). Also included are structures that tie directly into the levees and floodwalls such as road/railroad gates and fronting protection for pump stations.

The **15 Floodway Control & Diversion Structures** category refers to the floodgates described in section 7.3.1. Also included in the category are road gates that are adjacent to floodgates.

The **01 Lands and Damages** category includes land payments as well as administrative costs and incremental costs associated with acquiring the real estate interests. These estimates include costs of acquiring mitigation lands for both environmental mitigation and the preliminary buyout plan to mitigate for induced damages flood side of the levee.

The **30 Planning, Engineering & Design** category includes such costs as project management, engineering, planning, designs, investigations, studies, reviews, value engineering and engineering during construction. The estimated cost for Planning, Engineering, & Design is 12 percent of the total construction costs.

The **31 Construction Management** category (also sometimes referred to as Supervision & Administration) refers to costs associated with management of the project during construction.

7.9   Annual OMRR&R Costs

The project authorization for the Morganza to the Gulf project stipulates that all costs of Operation, Maintenance, Repair, Rehabilitation and Restoration (OMRR&R) are the 100 percent responsibility of the non-Federal Sponsor, with the exception of the HNC lock complex and GIWW structures. In accordance with that statutory requirement, when the project or a portion of the project construction is complete, the USACE is required by law to provide the non-Federal Sponsor with a notice of completion of construction and the commencement of the period of OMRR&R. For example, once a levee lift for a particular reach is constructed and covered in grass, maintenance (i.e. mowing) would start. Based on the assumed implementation schedule, the first feature of the project to be completed would be first-lift levees on reaches F-1, F-2, and I-1 by 2016. Table 7-4 presents the estimated annual OMRR&R costs.

Table 7-4. Annual OMRR&R costs for the 1% AEP Plan

Project Feature	Federal	Non-Federal	Total
Structures	1,590,000	1,570,000	3,160,000
Levees	0	3,970,000	3,970,000
Mitigation	See note	See note	<270,000
Total	**$1,700,000**	**$5,700,000**	**$7,400,000**

Note: Annual OMRR&R costs for structures and levees are in October 2012 prices with a 25 percent contingency and are rounded up to nearest ten thousand dollars. See description of annual OMRR&R costs for mitigation below.

Structures. The Federal OMRR&R costs in table 7-4 are for the GIWW and HNC lock complex structures. With the sponsor funded additional work item, the HNC lock complex OMRR&R costs could be slightly higher; any additional costs attributable to the deeper sill (e.g. higher dredging costs) would be the responsibility of the non-Federal sponsor. The OMRR&R for all structures, with the exception of the GIWW and HNC lock complex structures, is the responsibility of the non-Federal Sponsor.

Levees. Levee OMRR&R costs in table 7-4 are based on mowing 98 miles of earthen levees and dewatering and refurbishing numerous floodgates and a lock every 10 to 15 years. Future levee lifts are considered construction costs, not OMRR&R costs. The OMRR&R for levees is the responsibility of the non-Federal Sponsor.

Mitigation. In accordance with the project's statutory authority, the proposed mitigation actions will include construction, with the non-Federal Sponsor responsible for OMRR&R of functional portions of work as they are completed. On a cost shared basis, USACE will monitor completed mitigation to determine whether additional construction, invasive species control and/or additional plantings are necessary to achieve mitigation success. USACE will undertake additional actions necessary to achieve mitigation success in accordance with cost sharing applicable to the project and subject to the availability of funds. Once USACE determines that the mitigation has achieved initial success criteria, monitoring will be performed by the non-Federal sponsor as part of its OMRR&R obligations. If, after meeting initial success criteria, the mitigation fails to meet its intermediate and/or long-term ecological success criteria, USACE will consult with other agencies and the non-Federal Sponsor to determine whether operational changes would be sufficient to achieve ecological success criteria. If, instead, structural changes are deemed necessary to achieve ecological success, USACE will instruct the non-Federal sponsor to implement appropriate adaptive management measures in accordance with the contingency plan and subject to OMRR&R cost sharing requirements, availability of funding, and current budgetary and other guidance.

8   Plan Implementation

The following sections describe division of plan responsibilities, work in kind credit, and non-Federal sponsor financing.

8.1   Federal and Non-Federal Cost-Sharing

The CPRAB and TLCD have indicated their desire to act as non-Federal sponsors for this project and to perform work-in-kind in order to provide interim flood protection, expedite project completion and satisfy a portion of the non-Federal cost share. The non-Federal share of project costs is approximately $3,592,785,000 (October 2012 dollars), which would be provided over a 50 to 60 year construction period. The non-Federal sponsor would assume responsibility for OMRR&R of the hurricane and storm damage reduction system, except for those features identified as Federal responsibilities in section 8.2 of this report.

The non-Federal sponsor is required to provide all LERRDs for the project. The non-Federal sponsor requests that the entire remaining non-Federal share (other than LERRD requirements) be provided as work-in-kind rather than cash. Any levees or structures that are part of the Morganza to the Gulf post-authorization project and conform to USACE standards could be possible work-in-kind. The construction cost of this potential work in kind is estimated to be approximately $2,947,262000 (October 2012 dollars). The estimates in table 8-1 are subject to change as the design progresses and features become more refined.

Table 8-1. Federal and Non-Federal Cost-Sharing Summary

Cost Share Feature	Project Costs ($1000s)
Total Project Cost	$10,265,100
Federal Share (65 percent of Total Project Cost)	6,672,315
Non-Federal Share (35 percent of Total Project Cost)	3,592,785
Value of Proposed Work-in-Kind (non-Fed share minus LERRDs)	2,947,262
LERRDs	645,523
Additional Cash Required	0

Costs are in October 2012 price levels and are rounded to nearest hundred thousand dollars. Does not include sponsor funded additional work item.

8.2   Federal Responsibilities

The Federal government will be responsible for planning, engineering, design, and construction of the project in accordance with the applicable provisions of Public Law 99-662 (WRDA of 1986). The Government, subject to Congressional authorization and the availability of funds and using those funds provided by the non-Federal sponsor, shall expeditiously construct the project, applying those procedures usually applied to Federal projects, pursuant to Federal laws, regulations, and policies. In accordance with WRDA 2007, Section 1001(24), the Government shall operate, maintain, repair, rehabilitate, and replace the two floodgate structures along the GIWW and all components of the HNC lock complex, at no cost to the non-Federal sponsor.

8.3   Non-Federal Responsibilities

Federal implementation of the 1% AEP project would be subject to the non-Federal sponsor agreeing to comply with applicable Federal laws and policies, including but not limited to the following:

a. Provide 35 percent of total project costs as further specified below:

   1. Provide the required non-Federal share of design costs in accordance with the terms of a design agreement entered into prior to commencement of design work for the project;

   2. Provide, during the first year of construction, any additional funds necessary to pay the full non-Federal share of design costs;

   3. Provide all lands, easements, and rights-of-way, including those required for relocations, the borrowing of material, and the disposal of dredged or excavated material; perform or ensure the performance of all relocations; and construct all improvements required on lands, easements, and rights-of-way to enable the disposal of dredged or excavated material all as determined by the Government to be required or to be necessary for the construction, operation, and maintenance of the project;

   4. Provide, during construction, any additional funds necessary to make its total contribution equal to 35 percent of total project costs;

b. Shall not use funds from other Federal programs, including any non-Federal contribution required as a matching share therefor, to meet any of the non-Federal obligations for the project unless the Federal agency providing the Federal portion of such funds verifies in writing that expenditure of such funds for such purpose is authorized;

c. Not less than once each year, inform affected interests of the extent of protection afforded by the project;

d. Agree to participate in and comply with applicable Federal floodplain management and flood insurance programs;

e. Comply with Section 402 of the Water Resources Development Act of 1986, as amended (33 U.S.C. 701b-12), which requires a non-Federal interest to prepare a floodplain management plan within one year after the date of signing a project cooperation agreement, and to implement such plan not later than one year after completion of construction of the project;

f. Publicize floodplain information in the area concerned and provide this information to zoning and other regulatory agencies for their use in adopting regulations, or taking other actions, to prevent unwise future development and to ensure compatibility with protection levels provided by the project;

g. Prevent obstructions or encroachments on the project (including prescribing and enforcing regulations to prevent such obstructions or encroachments) such as any new developments on project lands, easements, and rights-of-way or the addition of facilities which might reduce the level of protection the project affords, hinder operation and maintenance of the project, or interfere with the project's proper function;

h. Comply with all applicable provisions of the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970, Public Law 91-646, as amended (42 U.S.C. 4601-4655), and the Uniform Regulations contained in 49 CFR Part 24, in acquiring lands, easements, and rights-of-way required for construction, operation, and maintenance of the project, including those necessary for relocations, the borrowing of materials, or the disposal of dredged or excavated material; and inform all affected persons of applicable benefits, policies, and procedures in connection with said Act;

i. For so long as the project remains authorized, operate, maintain, repair, rehabilitate, and replace (OMRR&R) the project or functional portions of the project, including any mitigation features, at no cost to the Federal Government, in a manner compatible with the project's authorized purposes and in accordance with applicable Federal and State laws and regulations and any specific directions prescribed by the Federal Government (except the HNC lock complex and the GIWW floodgate features of the project for which the responsibility for OMRR&R is assigned to the Government under Section 1001(24) of WRDA 2007);

j. Give the Federal Government a right to enter, at reasonable times and in a reasonable manner, upon property that the non-Federal sponsor owns or controls for access to the project for the purpose of completing, inspecting, operating, maintaining, repairing, rehabilitating, or replacing the project;

k. Hold and save the United States free from all damages arising from the construction, operation, maintenance, repair, rehabilitation, and replacement of the project and any betterments, except for damages due to the fault or negligence of the United States or its contractors;

l. Keep and maintain books, records, documents, or other evidence pertaining to costs and expenses incurred pursuant to the project, for a minimum of 3 years after completion of the accounting for which such books, records, documents, or other evidence are required, to the extent and in such detail as will properly reflect total project costs, and in accordance with the standards for financial management systems set forth in the Uniform Administrative Requirements for Grants and Cooperative Agreements to State and Local Governments at 32 Code of Federal Regulations (CFR) Section 33.20;

m. Comply with all applicable Federal and State laws and regulations, including, but not limited to: Section 601 of the Civil Rights Act of 1964, Public Law 88-352 (42 U.S.C. 2000d) and Department of Defense Directive 5500.11 issued pursuant thereto; Army Regulation 600-7, entitled "Nondiscrimination on the Basis of Handicap in Programs and Activities Assisted or Conducted by the Department of the Army"; and all applicable Federal labor standards requirements including, but not limited to, 40 U.S.C. 3141- 3148 and 40 U.S.C. 3701 – 3708 (revising, codifying and enacting without substantial change the provisions of the Davis-Bacon Act (formerly 40 U.S.C. 276a et seq.), the Contract Work Hours and Safety Standards Act (formerly 40 U.S.C. 327 et seq.), and the Copeland Anti-Kickback Act (formerly 40 U.S.C. 276c et seq.);

n. Perform, or ensure performance of, any investigations for hazardous substances that are determined necessary to identify the existence and extent of any hazardous substances regulated under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), Public Law 96-510, as amended (42 U.S.C. 9601-9675), that may exist in, on, or

under lands, easements, or rights-of-way that the Federal Government determines to be required for construction, operation, and maintenance of the project. However, for lands that the Federal Government determines to be subject to the navigation servitude, only the Federal Government shall perform such investigations unless the Federal Government provides the non-Federal sponsor with prior specific written direction, in which case the non-Federal sponsor shall perform such investigations in accordance with such written direction;

o. Assume, as between the Federal Government and the non-Federal sponsor, complete financial responsibility for all necessary cleanup and response costs of any hazardous substances regulated under CERCLA that are located in, on, or under lands, easements, or rights-of-way that the Federal Government determines to be required for construction, operation, and maintenance of the project;

p. Agree, as between the Federal Government and the non-Federal sponsor, that the non-Federal sponsor shall be considered the operator of the project for the purpose of CERCLA liability, and to the maximum extent practicable, operate, maintain, repair, rehabilitate, and replace the project in a manner that will not cause liability to arise under CERCLA; and

q. Comply with Section 221 of Public Law 91-611, Flood Control Act of 1970, as amended (42 U.S.C. 1962d-5b), and Section 103(j) of the Water Resources Development Act of 1986, Public Law 99-662, as amended (33 U.S.C. 2213(j)), which provides that the Secretary of the Army shall not commence the construction of any water resources project or separable element thereof, until each non-Federal interest has entered into a written agreement to furnish its required cooperation for the project or separable element.

r. Shall not use any project features or lands, easements, and rights-of-way required for such features as a wetlands bank or mitigation credit for any other project;

s. Pay all costs due to any project betterments or any additional work requested by the sponsor, subject to the sponsor's identification and request that the Government accomplish such betterments or additional work, and acknowledgement that if the Government in its sole discretion elects to accomplish the requested betterments or additional work, or any portion thereof, the Government shall so notify the Non-Federal Sponsor in writing that sets forth any applicable terms and conditions.

8.4   Implementation Sequencing and Requirements

Subject to project authorization, funding, and regulatory approval, construction is scheduled to begin in 2015 as shown in table 8-2. The schedule assumes a complete risk reduction system in place by 2024, with additional levee lifts so that the entire system meets 1% AEP standards by year 2035, and final levee lifts to account for sea level rise and subsidence completed by around 2070. The Morganza to the Gulf project requires construction authorization and the appropriation of construction funds. A continuous funding stream is needed to complete this project within the anticipated timeline, which requires continuing appropriations from Congress and the State of Louisiana in order to fund the detailed design phase and fully fund construction contracts.

Table 8-2. Implementation Milestones

Milestone	Dates
Public Review Draft	Jan 2013
Civil Works Review Board	May 2013
State and Agency Review	May – June 2013
Sign the Report of the Chief of Engineers	2013
Chief's Report sent to the Assistant Secretary of the Army (Civil Works)	2014
PED Phase Continues	2014 – 2015
Real Estate Acquisition and Construction Contracts Advertising and Award	2014 – 2015
Project Construction Starts	2015

The Morganza to the Gulf project does not start to function as a system or achieve its benefits until all first lift levees and structures are in place. Preliminary construction sequencing is based on availability of information required for construction, such as detailed plans and specifications, identified borrow sources, and environmental clearances. The HNC lock complex is one of the first work items. Preconstruction Engineering and Design on the lock was initiated in January 2000, and the lock is a critical component of both the Morganza to the Gulf risk reduction project and the authorized LCA project, Convey Atchafalaya River Water to Northern Terrebonne Marshes and Multipurpose Operation of Houma Navigation Lock (see section 10.1.7, LCA Program Uncertainties).

Given the size and complexity of the Morganza to the Gulf project, and the fact that not all borrow sources have been identified, most of the RPEIS is at a broad, programmatic level; however, the RPEIS includes a more in-depth analysis of features for which borrow sources have been identified and that could be constructed in the near future, including the HNC lock complex, the Bayou Grand Caillou floodgate, and levee reaches F and G-1. Borrow materials from construction of the lock would be used to construct adjacent levee reaches F and G-1. For these features, the RPEIS provides sufficient detail so that no further environmental clearances would be needed upon signing a Record of Decision. The remaining features would require supplemental NEPA documents before they could be constructed.

The non-Federal sponsor has requested that it be allowed to design and construct portions of the Morganza to the Gulf project as work-in-kind. The non-Federal sponsor would focus their effort on earthen levee construction (multiple lifts) concentrated between Reach E-2 and Reach L. The non-Federal sponsor would also construct floodgates on some typical bayous within the same geographical area, such as Bush Canal, Placid Canal, Bayou Pointe aux Chenes, Bayou Terrebonne, Humble Canal and Bayou Petit Caillou. Details regarding specific features and schedules for work-in-kind will continue to be coordinated between the USACE and the non-Federal sponsor throughout the design and construction phase of the project.

Districts are eligible to initiate PED upon the Division Engineer's transmittal of the final report to HQUSACE. PED normally begins before the completion of the Chief's Report if funds are available and a Design Agreement has been executed. In accordance with EC 1165-2-208, an in-kind Memorandum of Understanding (MOU) can be executed to allow work-in-kind (design only) even prior to execution of the Design Agreement. The USACE and non-Federal sponsor

are currently in the process of negotiating a design work-in-kind MOU; however, any work performed in advance of the MOU is not eligible for credit.

Once construction funds are appropriated for this project, the non-Federal sponsor and the Department of the Army will enter into a PPA. After the signing of the PPA, the non-Federal sponsor can begin to acquire the necessary land, easements and rights of way to construct the project. Since project features cannot be advertised for construction until the appropriate real estate interests have been acquired, obtaining the necessary real estate in a timely fashion is critical to achieving the project schedule. At the completion of construction, project features would be turned over to the non-Federal sponsor, which would then be fully responsible for their OMRR&R.

9 Review of the PAC Report and RPEIS

In compliance with USACE policies and NEPA, input on projects is solicited from technical reviewers, other government agencies, and the public. The following sections briefly describe coordination and review of the Morganza to the Gulf PAC report and RPEIS.

9.1 Types of Reviews

The PAC report and RPEIS have been subjected to three major types of technical review as described in the August 2011 Review Plan (USACE, 8/2011). The first review, District Quality Control, assures accountability for the technical quality of the PAC report. District Quality Control incorporates a team of reviewers similar in experience and structure to the project team, but who have not worked on the project prior to the review.

The second component, Agency Technical Review, is similar in structure to District Quality Control, but is performed outside of the New Orleans District by other USACE districts in coordination with the Mississippi Valley Division and the National Planning Center of Expertise for Coastal Storm Damage Reduction. The Agency Technical Review team includes about 15 reviewers from various technical fields.

Finally, an Independent External Peer Review (IEPR) Panel acts as a completely outside evaluation of the economic, engineering, and environmental analysis performed for the study, including the panel's assessment of the adequacy and acceptability of the economic, engineering, and environmental methods, models, and analyses used. The IEPR panel began reviewing the preliminary draft PAC report in June 2012 and provided their final report in March 2013.

In addition to the technical reviews described above, the report must undergo USACE legal and policy reviews at the draft and final report stages. Final policy comments on the Draft reports were received in February 2013, but additional legal and policy reviews will continue until the Final PAC/RPEIS is provided to the public during the 30-day State and Agency review period.

Once all required reviews are complete, the report will ultimately be reviewed by the Assistant Secretary of the Army (Civil Works) and coordinated with the Office of Management and Budget as appropriate for submission to Congress.

9.2 Value Engineering

In accordance with ER 11-1-321, the appropriate Value Engineering action has been completed for the PAC phase of this project. The Value Engineering study conducted in November 2010 (SVS, 2010) is valid for up to 6 years and must be re-addressed as appropriate beyond that period. All proposals indicating savings greater than $1 Million impacting plan formulation have been addressed in a document indicating whether the Value Engineering recommendation has been accepted and resolved, rejected, or will be evaluated at a later time, e.g. during final design. A number of recommendations should be deferred to design, and additional Value Engineering workshops will be completed on detailed designs. At least two Value Engineering workshops will be needed—one for levees and one for structures, but given the complexity of the HNC lock complex, it is estimated that a total of five workshops will be needed at a cost of $90,000 for each workshop or an estimated $450,000 for Value Engineering on the total project.

9.3   Planning Model Certification

EC 1105-2-412, Assuring Quality of Planning Models, requires certification of planning models used in USACE studies. Planning models are models and analytical tools that planners use to define water resources management problems and opportunities, to formulate potential alternatives to address the problems and take advantage of the opportunities, to evaluate potential effects of alternatives, and to support decision-making. EC 1105-2-412 does not cover engineering models used in planning, which are certified under a separate process. A summary and status of model certification for the PAC are shown in table 9-1.

Table 9-1. Planning Model Certification Status

Name	Purpose	Status of Certification
HEC-FDA Version 1.2.5a	Calculate flood damages and benefits for structures, contents and vehicles and part of emergency cost damages and benefits.	Certified
Economic Benefit Spreadsheet Tools	Calculate emergency cost damages and benefits (debris removal and highways/streets) and analytical tool to use statistical bootstrapping to adjust CSVRs.	Approved for use on 24 April 2013
RECONS	Capture the impacts to income, output, and employment for the regional economy that are associated with construction spending on each of the alternatives.	Certified
Wetland Value Assessment Models - Coastal Marsh Module Version 1.0	Evaluate coastal marsh impacts and benefits for the project alternatives and mitigation.	Approved on 28 February 2012*
Wetland Value Assessment Bottomland Hardwood and Swamp Models	Evaluate swamp and bottomland hardwood impacts and benefits for the project alternatives and mitigation.	Approved on 8 November 2011*

 *WVA approval memos are included in Appendix F of the RPEIS.

9.4   Public Review of the Draft PAC/RPEIS

Since the start of the feasibility study and the scoping meeting in May 1993, there has been an ongoing public outreach program to the Terrebonne and Lafourche Parish communities through public meetings, media day events, briefings to Parish Council Board Meetings, and other public presentations and discussions. The CPRAB and TLCD have expressed their support for the project and their intent to act as the non-Federal sponsors. Close coordination with the CPRAB and the TLCD has been maintained throughout the planning process and they are involved in the key decisions discussed and developed by the team.

The RPEIS has generated significant interagency interest with the full involvement of the Louisiana CPRA, CPRAB, DOTD, TLCD, SLLD, and the Morganza Habitat Evaluation Team. The Morganza Habitat Evaluation Team consists of representatives from CPRA, USEPA, USFWS, NMFS, NRCS, USGS, LDWF, LDNR, and other Federal and state agencies.

The Draft PAC/RPEIS documents were open to public comment from January 4, 2013 to February 19, 2013. A public meeting was held in Houma, LA on January 31, 2013. In addition to individuals and the non-Federal sponsors (CPRA and TLCD), the following agencies and NGOs commented on the Draft RPEIS:

- United States Environmental Protection Agency (USEPA)

- United States Fish and Wildlife Service (USFWS)

- National Marine Fisheries Service (NMFS)

- United States Coast Guard

- Natural Resources Conservation Service (NRCS) State Conservationist in Alexandria, LA and Assistant State Conservationist in Lafayette, LA

- Louisiana Department of Environmental Quality (LDEQ)

- Louisiana Department of Natural Resources (LDNR)

- Louisiana Department of Wildlife and Fisheries (LDWF)

- National Wildlife Federation

- Lafourche Parish Council

- Gulf Intracoastal Canal Association (GICA)

- Louisiana Environmental Action Network (LEAN)

- Louisiana Audubon Council

- Gulf Restoration Network

- Atchafalaya Basinkeeper

- Delta Chapter, Sierra Club

USFWS concerns and recommendations are also documented in the USFWS Coordination Act Report, which is part of the RPEIS.

The most common concern in the top 10 comment themes (representing over 50 percent of the comments) are the potential environmental impacts of the project and associated mitigation requirements. Federal and State agencies noted that in the future there is a potential for adverse indirect and cumulative impacts to wetlands, fisheries, water quality, and navigation assuming there would be increased frequency and duration of water control structure closures as sea level rise accelerates. In response to these concerns, the Final PAC/RPEIS includes a quantitative analysis of the range of potential indirect impacts, including a mitigation plan, for the constructible features, and a qualitative analysis of the potential impacts for the entire project.

The remaining 9 of the top 10 comment themes (from most common to least common with each theme representing less than 10 percent of the total comments) were related to communication, coordination, and consideration of socioeconomic/cultural impacts; plan realignment or reformulation; design standards, RSLR, & constructability; project economics; nonstructural measures; the relationship of Morganza to other plans or projects, such as the State Master Plan and Louisiana Coastal Area (LCA) projects; GIWW floodgate size change, modeling, and impacts; non-Federal contributions; and general support for the project.

Refer to RPEIS section 8 for a more detailed discussion of Public Involvement, and RPEIS Appendix E, Public Comments, for all public and agency comments received during the 45-day public review period along with USACE responses.

10 Conclusions

The plan to provide hurricane and storm damage reduction to portions of Terrebonne and Lafourche Parishes in south Louisiana as updated in this PAC report is based on a thorough analysis and evaluation of all practicable alternatives, in view of applicable economic, engineering, and environmental criteria. Based on the data analyzed, of the two alternatives the 1% AEP alternative has the highest net benefits.

10.1   Planning Assumptions and Uncertainties

Given the complexity of this type of water resources project, analysts and decision makers do not have access to perfect information. Making simplifying assumptions in the face of large uncertainties is part of the normal planning process; however, decision makers need to be aware that conditions may change and planning assumptions could become invalid over time.

Key sources of risk and uncertainty are described in this section. Some types of risk and uncertainty can never be fully eliminated; however, measures or actions can be taken to reduce some risk and uncertainty, such as taking additional soil borings during detailed engineering design, monitoring sea level rise and subsidence to ensure the project continues to perform as designed into the future, monitoring and adaptively managing wetland mitigation projects, developing and rehearsing emergency operation and evacuation plans, and educating the public on residual risks of the levee system.

10.1.1 Non-Federal Levees

The without-project condition accounts for the existence of non-Federal levees within the Morganza project area (approximately 92 miles total; not all contiguous). Their performance was modeled using fragility curves. Most of the local levees are located close to development and are only intended to prevent tidal flooding from small tropical storm events, so they provide some benefit for the more frequent, lower flooding events, but tend to perform poorly at the 3% and 1% AEP surge events for which the Federal hurricane and storm damage risk reduction alternatives were formulated.

In the with-project condition, the performance of the local levees is not quantified. Once the Federal levee is in place, local levees are no longer subjected to direct hurricane surge. If a hurricane event is large enough to cause a breach in the project levees, then it is assumed the local levees would breach as well; therefore, the impact of not including the local levees in the analysis is minor.

Approximately 9 of the 92 miles of local levees are located along the Morganza to the Gulf PAC alignment. The non-Federal sponsor designed and constructed those 9 miles of levees with the intent of meeting HSDRRS standards so that the levees could eventually be incorporated into the Morganza Federal levee project. Per USACE policy, all local levees built without a signed PPA are part of the without-project conditions. Based on preliminary review of non-Federal levee designs, the assumption was made for the PAC report that the 9 miles of local levees would meet HSDRRS criteria and could be incorporated as a foundation to the Federal levees in those reaches. The approval process to incorporate the non-Federal levees into the Federal project would require detailed evaluation of plans and specifications and additional soil borings. Since that detailed review has not yet occurred, final determination of their suitability for integration into the Federal project cannot be made at this time. If the non-Federal levees are found to be incompatible with the Federal project, the Morganza PAC total project cost estimate may

increase based on additional costs to degrade and rebuild the non-Federal levees, or to realign the Federal project.

10.1.2 Economic Risk Analysis

Benefits modeled in HEC-FDA are expected to sufficiently replicate economic benefits of the alternatives. The risk-based analysis meets the intent of EC 1105-2-101 because all key variables, parameters, and components of the hurricane and storm damage reduction study have been subjected to probabilistic analysis. The key variables include stage-damage functions; stage- probability functions; length of record used to determine stage-probability confidence bands; and structural/geotechnical performance of levees.

Rainfall is not included in the HEC-FDA model because rainfall damages would be the same with or without the proposed Federal project. Inhabited areas are generally behind local drainage levees, and local drainage improvements are outside the scope of the project purpose and authority. In the uninhabited wetlands areas, environmental water control structures are sized to address issues related to isolated ponding areas that are created by the construction of the proposed Federal levee.

The without-project HEC-FDA model includes fragility curves that describe the reliability of the local levees. These fragility curves were developed using limited available data, stability analyses, and erosion evaluation. Wave overtopping was not considered; if the local levee does not fail, no damages are computed by the HEC-FDA model, even though wave overtopping could result in without-project damages. These assumptions resulted in overstating the performance of the local levee system and thus reduced the without-project damages that were used to compute project benefits.

The economic analysis performed for the with-project conditions represent the Federal levee performance by a single point fragility curve that has a zero percent probability of failure until a wave overtopping rate of 2 cfs/ft is reached, at which time, failure is considered likely. For all events where the wave overtopping is less than 2 cfs/ft, no damages are computed by the HEC-FDA model. The failure point chosen on the Federal levee would overstate residual damages and therefore understate benefits.

Recent full scale wave overtopping simulation research at Colorado State University (CSU) and simulation research projects on levees were performed to determine the need for armoring. This specific analysis forms the basis for the assumed performance of the proposed Federal grass-covered earthen levees. The upper estimate in the CSU tests was 2.0 cfs/ft. Levees could fail if armoring is not present and overtopping is greater than 2.0 cfs/ft. With the likelihood more certain, the failure probability for 2.0 cfs/ft was set at 95 percent.

A sensitivity analysis was performed on an economic reach for the with-project Federal levee with three test cases. Test Case A has a typical fragility curve developed from geotechnical and hydraulic analyses, Test Case B is the single point failure with the 100 percent probability at a surge elevation that results in 2 cfs/ft wave overtopping, and Test Case C is a single point failure with the 100 percent probability surge elevation at the top of levee. The sensitivity analysis demonstrated that the single point failure is a proxy for a typical fragility curve that meets the intent of the ER 1105-2-101 to address risk and obtain economic consequences. The with-project economic analysis using a single point fragility curve with a failure point at 2 cfs/ft wave overtopping understates the performance of the levee, calculating higher residual damages and therefore understating the benefit-cost ratio.

In the HEC-FDA model, once the wave overtopping reaches 2 cfs/ft, the Federal levees fail and interior stage becomes the without-project stage and residual damages are computed. A breach analysis for the with-project Federal levee demonstrated the interior stage with a breach is likely to be lower than the exterior stage and the without-project stage; the interior/exterior relationships used in the Federal levee analysis therefore understates the performance of the levee again understating the benefit-cost ratio.

In reviewing each of these assumptions, the USACE Risk Management Center concluded that the methodology used in this study resulted in underestimating the without-project damages. The analysis performed on the with-project conditions understated the performance of the Federal levees as well as overstated the residual damages due to the interior/exterior relationships used in the economic model.

10.1.3 Sea Level Rise and Subsidence (Relative Sea Level Rise)

There is widespread consensus in the international scientific community that sea levels will continue to rise and could accelerate. Coastal Louisiana's low elevation, high rate of subsidence, and increased rate of wetland loss make it vulnerable to changes in climate. Sea level change is defined and evaluated in terms of "relative sea level rise," which includes the effects of global and local sea level change as well as local subsidence. Although subsidence and sea level rise are independent processes, the net effect of the two processes results in local relative sea level rise, which is what matters for planning.

Unlike the 2002 report, the PAC structure inventory was not lowered to account for subsidence; rather, the surge and wave model accounts for both sea level rise and subsidence. Project designs in the PAC are based on the Intermediate RSLR scenario, which equates to increased water levels of 0.7 ft in the PAC base year (2035) and 2.4 ft in the PAC future year (2085). In the 2002 feasibility report analysis, to account for future sea level rise and subsidence, portions of the structure inventory fronted by healthy marsh were lowered by approximately 0.5 ft in the base year (2008) and 2.3 ft in the future year (2057). Portions of the structure inventory fronted by unhealthy marsh were lowered by approximately 0.7 ft in the base year and 3.1 ft in the future year. Note: The 2002 feasibility report evaluated a different 50-year period of analysis than the PAC report.

Based on storm surge and wave modeling for this study, the impact of sea level change on surge and waves is non-linear. Surge propagation over broad, shallow, wetland areas is highly sensitive to sea level rise. As wetlands deteriorate, it is likely that water depths will increase further and sea level rise will impact surge levels to an even greater extent, compared to the base case estimates. Waves also generally increased significantly for all sea level rise cases. The wave height increases are significant, but less dramatic than the surge increases.

For a discussion of uncertainties associated with RSLR and indirect environmental impacts, refer to section 10.3.3, Indirect Environmental Impact and Mitigation Plan Uncertainties.

10.1.4 Geotechnical Considerations for Multi-Lift Levee Construction

Multiple lifts are necessary to achieve and maintain levee design elevations over the 50-yr period of analysis. Levee designs represent the size/configuration required to attain the specified level of risk reduction (1% or 3% AEP) by the base year (2035) and the end of the analysis period (2085) accounting for settlement and the Intermediate RSLR scenario. Given the long construction period for this project, the final levee lifts could be adjusted to account for higher or lower levels of actual RSLR.

Constructing multiple levee lifts in a coastal environment with unstable sediments can present challenges. The project area contains large amounts of open water and many bayous, which results in heterogeneous soil foundations. As a result, parts of levees built on existing ridges or levees are expected to have much less settlement than connecting features. The USACE has a long history of working in the weak, compressible soils of southeast Louisiana and has incorporated this knowledge into the PAC feasibility-level levee and structure designs. About one-third of the subsurface exploration required for construction of the project has been completed. Data from each soil boring had to be extrapolated over a large area, which can result in inexact assumptions; however, efforts were made to provide a conservative, yet realistic design, that does not underestimate or overestimate costs because of limited data, or place significantly more emphasis on data that was retrieved from the bored locations. These designs are the basis for a representative PAC cost estimate, but are not intended to be the final engineering designs. Additional geotechnical exploration and testing is planned during the detailed design phase (plans and specs).

10.1.5 Levee Armoring

The selected method for increasing resilience of the Morganza to the Gulf levees in the case of wave overtopping is grass. The with-project economic analysis included the reliability of the constructed project with HSDRRS standards for grass and no additional armoring. Armoring with more resilient material, such as high performance turf reinforcement, involves a cost and performance tradeoff analysis between levee elevation (and associated overtopping rates) and armoring. For example, where sufficient overtopping storage exists behind the levee, levee elevations could be lowered if more robust armoring, e.g. concrete mats, were used to prevent erosion in the case of overtopping. For the Morganza PAC, a test case was run on one such levee reach to determine whether or not it would be more cost effective to add armoring and reduce the height of the levee. The preliminary results did not show a cost savings. The determination of whether or not additional armoring is necessary or cost effective should be made on a reach-by-reach basis; therefore, additional analysis on armoring should be conducted during PED.

10.1.6 LCA Program Uncertainties

Several LCA projects authorized by WRDA 2007 are located within the Morganza study area, including but not limited to: (1) Convey Atchafalaya River Water to Northern Terrebonne Marshes and Multipurpose Operation of Houma Navigation Lock (2) Modification of Davis Pond Diversion and (3) Land Bridge between Caillou Lake and Gulf of Mexico. By letters dated 20 August 2012 and 16 October 2012, CPRAB has notified USACE that it desires to suspend study and design on these projects. The decision of CPRAB to suspend these projects results in some degree of uncertainty regarding implementation of these projects as part of the authorized Federal LCA.

The only LCA project with a direct link to a feature of the Morganza project is the Convey Atchafalaya River Water to Northern Terrebonne Marshes and Multipurpose Operation of Houma Navigation Lock project. The HNC lock complex to be constructed as part of the Morganza to the Gulf project for storm surge and salinity control could also be operated for ecosystem restoration purposes, such as distribution of freshwater. Proposed operational changes for LCA ecosystem restoration purposes, and associated impacts, are documented in the *Final Integrated Feasibility Study and Environmental Impact Statement for the Convey Atchafalaya River Water to Northern Terrebonne Marshes and Multipurpose Operation of*

Houma Navigation Lock (USACE, 2010). The LCA plan relies on the operation of the HNC lock for environmental purposes after 2025 and proposes the modification of the operational plan for the lock complex structure authorized under Morganza to the Gulf, in order to maximize potential environmental benefits, both in terms of avoiding saltwater intrusion and optimizing flow distribution. For the multipurpose operation to occur, the LCA project would need an OMRR&R plan that considers operation of the lock beyond the current authorization of the Morganza to the Gulf project. A detailed multipurpose OMRR&R plan including these environmental purposes has not yet been developed.

10.1.7 Sponsor Funded Additional Work Item to Address Future HNC Depth

Considering the uncertainties associated with potential future deepening of the HNC, the benefits of building a more flexible lock complex are significant for the Nation. The non-Federal sponsor would assume all incremental costs and incremental OMRR&R of the sponsor funded additional work item. The overall benefits of implementing the sponsor funded additional work item outweigh the additional costs that result from the deeper sill depth. Thus, the New Orleans District has included the -23 ft NAVD88 sill elevation as part of the 1% AEP post-authorization plan as a sponsor funded additional work item. Significant coordination with the resource agencies has been undertaken on both the 1% AEP alternative and the sponsor funded additional work item. No issues have been raised at this stage in the planning process that would preclude implementation of either project.

10.1.8 Price Levels and Interest Rates for Economic Evaluations

Price levels and interest rates change over time and impact the economic viability of projects. The long‑term implementation approach carries with it significant risks for Federal and local funding shortfalls. The long implementation time also increases the probability that the levees could be damaged by storms before they are completed.

Benefits and costs for the economic analysis of the *initial array* of alternatives are in October 2008 price levels and based on a 4.875 percent interest rate. Benefits and costs for the economic analysis of the *final array* alternatives was initially performed using October 2011 price levels, but was later updated to October 2012 price levels and a 3.75 percent interest rate.

10.2 Residual Risk

No alternative plan can eliminate risk and uncertainty, and tradeoffs must be made among engineering performance, economic performance, and project costs. These benefit and cost tradeoffs are often expressed as local and Federal project costs, flood damage reduced, and residual risk. It is vital that the non-Federal sponsor and residents understand these tradeoffs and the residual risks in order to fully participate in an informed decision-making process. Figure 10-1 illustrates how everyone has a role in reducing risk and residual risk can never fully be eliminated.

Figure 10-1. Flood Risk Management: Buying Down Risk

At the top of the steps in Figure 10-1 is the initial risk of flooding. Individuals who purchase flood insurance buy down some of their initial risk. The next step shows how parish and state governments can further reduce flood risk by proactively setting and enforcing building codes and zoning regulations. Note that levees are only one of many steps to reduce risk. Even with the Morganza project in place, some risk of flooding remains, as well as other residual risks such as wind damage. An important step is for parish and state governments to develop evacuation plans and for individuals to heed them. If everyone contributes to the effort, there will be a smaller amount of risk at the bottom of the stair steps.

10.2.1 Insurance, Building Codes, and Zoning to Reduce Property Damage Risk

The National Flood Insurance Program enables property owners to purchase insurance as a protection against flood losses in exchange for state and community floodplain management regulations that reduce future flood damages. Local building ordinances, reflecting the floodplain regulations mandated by participation in the National Flood Insurance Program, require that new residential or commercial structures be constructed with first-floor elevations placed 1 foot above the base flood elevation (BFE) as indicated in the prevailing flood insurance rate maps (FIRMs), published by FEMA. As FEMA periodically updates its flood insurance rate studies and associated FIRMs, the BFEs are expected to change over time.

Prior to the impacts of Hurricane Juan in 1985, Terrebonne Parish residents often requested variances on minimum structural elevations. After the storm, the Flood Appeals Board was disbanded, and since then residents have not been allowed to vary from required elevations. General practice in the study area based on local parish recommendations is to build 2 ft above

the FEMA recommendation. In some cases these elevations are 4 ft higher than observed in Juan (1985). Terrebonne Parish is the most active county in the nation in pursuing non-structural flood control. They have elevated over 1,000 structures since 2001, and bought out some structures largely through grant programs. Substantially damaged structures from any storm are required to elevate. The Parish has been updating zoning laws to work towards a sustainable community. The National Flood Insurance Program's (NFIP) Community Rating System (CRS) has a voluntary incentive program that recognizes and encourages community floodplain management activities that exceed the minimum NFIP requirements; the Terrebonne Parish CRS number is the best in the state and results in lower insurance rates.

The economic analysis in this report was conducted consistent with this expectation as the lower portion of the first-floor level of future (without-project) structures are added to the structure inventory at elevations that are at or above the stage associated with the 1% ACE (equivalent to the BFE). In this way, structures placed into service in the future are compliant with flood plain regulations. Also, these structures are expected to be subject to flood risk for events less probable than the 1% ACE. Under with-project conditions, the FEMA-established BFEs in future updates of the FIRMs are not expected to be as high as would have been established under without-project conditions, owing to the performance of the structural plans in place. The potential damage to these residential or commercial structures is expected to be higher for storm surge events that exceed the performance of the structural alternative in place. This conclusion is based upon the observation that the BFE under without-project conditions is likely to be higher than under with-project conditions. Also, under the without-project condition, structures subject to repetitive loss were elevated in future analysis years, while structures behind the proposed levee project in the with-project condition were not. As a result, residual risk is higher under with-project conditions relative to without-project conditions should the Federal levee not perform as intended.

The 1% AEP standard is important within the National Flood Insurance Program because a levee certified to the 1% AEP (100-year) design level could reduce National Flood Insurance Program insurance premiums within the levee system. The 1% AEP levee design is intended to meet FEMA 100-year levee certification requirements. Project design and performance by levee reach would be verified during PED to assure compliance with FEMA certification criteria.

10.2.2 National Levee Safety Program

The National Committee on Levee Safety (NCLS) was established by the WRDA of 2007, specifically Title IX, also known as the "National Levee Safety Act of 2007." Members of the NCLS were invited to a hearing of the House Transportation and Infrastructure Subcommittee on Water Resources and the Environment in May 2009, where the recommendations for a National Levee Safety Program were addressed. In November 2010, the NCLS was invited to testify before the Senate Committee on Environment and Public Works on levee safety issues. A National Levee Safety Program hinges on shared responsibility at all levels of government and from the public. For more information, visit www.leveesafety.org.

10.2.3 Evacuation Planning and Outreach to Reduce Loss of Life

As population grows, the population at risk to flooding is also expected to increase. The growth in population and housing that has been observed within the study area in past decades, as documented in U.S. Census data, is expected to continue under both without-project and with-project conditions. Between 2010 and 2085, the population at risk in Terrebonne and Lafourche parishes is expected to grow from 133,700 to 165,000 residents. These projections of population

and housing growth were based upon the results of a macroeconomic analysis conducted by Moody's Economy.com that links population growth (and associated demand for housing and commercial services) to an underlying growth in employment demand in the region.

The 1% AEP plan will not result in a change in the level of employment within the study area since the presence of a storm surge risk reduction system will not promote further growth within the major employment sectors comprised of oil and gas exploration and development, commercial fishing, and agriculture. Rather, the structural plans are expected to preserve economic activities that exist, or would be expected to exist in the future, under without-project conditions.

Insofar as the structural plans do not induce development of population and housing, there is no further change in the number of persons exposed to residual risk than would have existed under without-project conditions. Yet, in cases where the surge events exceed the design of the structural projects, the additional population that is counted within the study area beyond the base year will be exposed to increased flood hazard.

All coastal parishes, Terrebonne and Lafourche in particular, exercise aggressive evacuation plans which are mandatory by state law and coordinated by the State of Louisiana according to the state's emergency operations plan that is put into effect 72 hours prior to expected landfall of tropical storms. Whether the current effectiveness of evacuation response of the affected population (generally in the 80 to 95 percent range for Hurricane Gustav) would change with the project in place is uncertain; it is possible that the percentage of the population evacuating could be lower than is currently the case. An increase in population in the study area in the future corresponds to an increase in the number of individuals that may remain in the study area after an evacuation order is issued, and thus an increase in the number of individuals exposed to residual risk associated with storm surges that exceed the project design. This means that for the approximately 8,900 persons added to the population between the years 2035 (the base year of the project) and 2085, most of whom would evacuate in advance of an approaching storm, an additional 445 to 1,780 persons would face life-loss risks in the event of project exceedance. Under such conditions, the parishes' emergency recovery plans would become operational to facilitate cleanup and repair of public facilities, to ensure public safety as population returns to the area in managed phases, and to coordinate relief services with state and federal officials. While the requirement to respond to the needs of additional population under such catastrophic conditions places additional burden on parish law enforcement, relief personnel and facilities, and financial resources, the plans themselves are not expected to be altered to accommodate the change in scope of emergency operations.

Within the last 10 to 20 years, there have only been two known drowning deaths during hurricanes, including one in 2008 during Ike (the only drowning since 1928). The community is inherently prepared for hurricanes as a consequence of a water-based culture (fishing, hunting, oil industry, etc). Many of those in the oil industry that work on off-shore platforms receive early forecasts. Unlike in the case of Hurricane Katrina and New Orleans, the proposed Federal levee alignment is an average of 25 miles from the main population center (Houma), with ring levees and large basins available for storage.

Terrebonne and Lafourche Parish officials have taken an active role in communicating the need to evacuate to the public regardless of the presence of levees or not. Post-Hurricane Katrina willingness to evacuate has greatly increased, which is typical after major hurricanes. If time passes without another major hurricane event, willingness to evacuate may decline, but officials

indicate that they will continue to educate the public and encourage evacuation after the construction of the proposed levee alignment.

10.2.4 Compliance with Executive Order 11988 and Policy Guidance Letter 25

Executive Order 11988, 24 May 1977, Floodplain Management outlines the responsibilities of Federal agencies in the role of floodplain management such that each agency shall evaluate the potential effects of actions on floodplains and should not undertake actions that directly or indirectly induce growth in the floodplain, unless there is no practical alternative. USACE Policy Guidance Letter 25 further specifies that structural plans will be formulated to protect "existing development and vacant property that is interspersed with existing development… and for vacant property that is not interspersed with existing development if it can be demonstrated that the vacant property would be developed without the project and benefits are based on savings in future flood proofing costs or reduction in damages to future development." Compliance with Executive Order 11988 is accomplished by following the eight general procedures outlined in ER 1165-2-26 (30 Mar 1984), paragraph 8, subparagraphs (a – h). The manner in which these procedures were observed for the Morganza to the Gulf 1% AEP plan is summarized below:

a. Determine if the proposed action is in the base flood plain.

The proposed action is located in the base flood plain.

b. If the action is in the base flood plain, identify and evaluate practicable alternatives to the action or to location of the action in the base flood plain.

In addition to the No Action Alternative, several action alternatives within the base flood plain were evaluated in the original 2002 feasibility study and the current PAC study, including both structural and nonstructural alternatives (refer to section 4 of the main PAC report). Nonstructural alternatives were not found to be feasible in the 2002 feasibility report (refer to section 4.2 of the main PAC report). Since the majority of the study area is in the base flood plain, all structural alternatives impact the base flood plain to some extent. In the PAC analysis, the authorized alignment was reconfirmed as the alignment that best meets Federal objectives, i.e. most effective, efficient (cost-effective), complete, and acceptable. Alternative locations of the proposed action outside of the base flood plain would not achieve the flood risk reduction goals of the action, and thus were not considered.

c. If the action must be in the flood plain, advise the general public in the affected area and obtain their views and comments.

The general public was advised of the proposed action during the 2002 feasibility report public meetings and 45-day Draft and 30-day Final public review periods, and more recently, during a 45-day public review and comment period for the 2013 Draft PAC Report and RPEIS. A public meeting for the Draft PAC Report was held in Houma, LA on January 31, 2013.

d. Identify beneficial and adverse impacts due to the action and any expected losses of natural and beneficial flood plain values. Where actions proposed to be located outside the base flood plain will affect the base flood plain, impacts resulting from these actions should also be identified.

The proposed action is designed to enhance flood plain values in the form of flood risk reduction. Beneficial and adverse impacts to the natural and aquatic flood plain values due to the proposed action are addressed in the environmental impact statement that supports the PAC report.

e. If the action is likely to induce development in the base flood plain, determine if a practicable non-flood plain alternative for the development exists.

The action is not expected to induce development in the base flood plain since the action is not expected to increase the availability of natural resources which is the driver of employment and population. Also, the limits to development are observed as most of the areas behind the proposed Federal levee are wetlands and/or open water areas subject to tidal influence that will remain subject to tidal influence, or they are behind pumped drainage levees that are already developed. Therefore, no practicable non-flood plain alternatives have been identified.

f. As part of the planning process under the Principles and Guidelines, determine viable methods to minimize any adverse impacts of the action including any likely induced development for which there is no practicable alternative and methods to restore and preserve the natural and beneficial flood plain values. This should include reevaluation of the "no action" alternative.

The proposed action is expected to have beneficial and adverse impacts to the base flood plain and measures to avoid, minimize and mitigate those adverse impacts are identified in detail in the environmental impact statement that supports the PAC report.

g. If the final determination is made that no practicable alternative exists to locating the action in the flood plain, advise the general public in the affected area of the findings.

The general public in the affected area was advised of the proposed action through a news release followed by a 45-day review and comment period for the Draft PAC Report and RPEIS. A public meeting for the Draft PAC Report was held in Houma, LA on January 31, 2013.

h. Recommend the plan most responsive to the planning objectives established by the study and consistent with the requirements of the Executive Order.

The Final PAC report identifies the 1% AEP hurricane and storm damage risk reduction system as the alternative that has the highest net benefits and is most responsive to planning objectives by maximizing the Federal investment and minimizing residual risk. The 1% AEP plan is designed to minimize the impact of floods on human safety, health and welfare, and it preserves the natural and beneficial values of the base flood plain to the greatest extent possible through the use of environmental control structure and floodgates that will remain open except during tropical storm events.

10.2.5 Narrative Future Scenario

Some risk and uncertainty factors associated with the Morganza to the Gulf project are difficult to quantify. The following future scenario for the year 2035 is included to help communicate risk.

By 2035, after two decades of additional wetland degradation and sea level rise, water levels in the Gulf of Mexico and waterways throughout the area may be almost a foot higher than they are today, resulting in even higher storm surges than have been seen in the past. Should Houma and surrounding communities in Terrebonne and Lafourche Parishes be threatened by a hurricane, over 181,000 individuals would be faced with the decision of when to begin voluntary evacuations. Approximately 50 hours before the onset of tropical storm winds, the State of Louisiana and local officials would begin mandatory evacuations.

If the Morganza to the Gulf hurricane levee system were never built, the 100-year surge event (which has a 1 percent chance of occurring each year) could overtop and breach most of the existing local levees resulting in over 18 ft of flooding in some areas and potentially impacting over 134,000 structures, including homes, businesses, boats, and automobiles that could not be evacuated. Because of the lack of natural drainage in some areas, flood water would likely remain in low-lying areas for 2 weeks or more, as seen in the aftermath of Hurricane Katrina.

Alternately, if a system of earthen levees ranging in elevations of 10.5 to 24 ft were in place in 2035, it is expected that the project would most likely be able to withstand a 1 percent chance event and structures within the levee system would remain undamaged by storm surge (structures would still be subject to wind damage). Even with the 1% AEP Plan, the project area could still flood in extremely rare events, but would have reduced risk of flooding up to the 500-year or 0.2 percent annual chance flood event.

10.3   Unresolved Issues

Given the size and complexity of the Morganza to the Gulf project, some issues could not be fully resolved at a feasibility-level of detail within the PAC timeframe. Issues that should be resolved at a greater level of detail as the project moves forward include the potential for induced flooding outside the levee system; the future level of risk reduction for the Larose to Golden Meadow project; and future operation of floodgates and water control structures as sea level rises. These plan components would be further addressed through detailed design and supplemental NEPA documents.

10.3.1 Induced Flooding Outside the Project

An indirect impact of the construction of the project is the potential to raise water levels outside the levees by several feet during storm events causing induced flooding to several communities located outside of the proposed levee alignment. These areas include portions of the communities of Gibson, Bayou Dularge, Dulac, and all of Cocodrie and Isle de Jean Charles.

 Presently, information is not available on the specific details on the differences in frequency, depth, and duration of the flooding between the future without-project and future with-project conditions. This detailed information typically would be assessed in light of the uses to which the particular land is zoned, and the appropriate mitigation methods, if any, would be implemented to address the effects of the Federal project. Because of the vast scope of this project and the limited amount of available information at this time, the USACE did not look at each affected parcel individually in order to determine potential impacts to property rights from the proposed Federal action that may give rise to compensation. For example, without more information, it is impossible to rule out the possibility of takings for all of the structures in these communities. Due to this concern, the USACE for purposes of this report, has assumed the

worst case compensation scenario (most expensive option), a 100 percent buy-out of all of the structures in the impacted areas. The cost and associated benefits with this scenario has been incorporated into the 1% AEP post-authorization plan (see PAC section 6.5.1).

Socioeconomic considerations associated with the potential buyouts are described in RPEIS section 5.2.13, and Environmental Justice is discussed in RPEIS section 6.14.8 and RPEIS Appendix J.

10.3.2 Larose to Golden Meadow Project Future Elevations

The future-without condition for the Larose to Golden Meadow levee system is uncertain since the Larose PAC analysis is ongoing and future levee elevations for the existing Larose ring levee system have not yet been determined. If the Morganza project is re-authorized to the 1% AEP level of risk reduction, but the Larose project is (a) not re-authorized; (b) re-authorized to less than a 2% AEP level of risk reduction; or (c) is authorized but not supported by a financially capable Non-Federal Sponsor willing to execute a PPA, the Morganza project would have added costs to both offset induced stages on the existing Larose system and to complete the Morganza system to ensure no overtopping of the Larose C-North levees that could impact the Morganza risk reduction area. Therefore, the Morganza to the Gulf PAC analysis assumes no further upgrades to the Larose to Golden Meadow system to ensure that all potential costs to complete the Morganza system are considered.

10.3.3 Indirect Environmental Impacts and Mitigation Plan Uncertainties

As identified in comment letters from several Federal and State agencies that reviewed the January 2013 Draft RPEIS document, creation of impounded wetlands, disruption of sediment deposition and sheet flow, circulation changes, induced development in wetlands, and other indirect impacts to wetlands have been and continue to be controversial issues within the environmental community of southern Louisiana. For example, existing road dumps and canal spoil banks, in combination with construction of the proposed levees, may create small unintentional impoundments that could result in adverse impacts to enclosed wetlands. Such problems exist within the proposed Barrier Reach levees, Reach A levees, the Larose reaches, and other areas. Any such problems will be addressed during the PED of those levee reaches and will be documented in a supplemental NEPA document.

Of greater concern to the agencies and NGOs is the potential for adverse indirect and cumulative impacts to wetlands, fisheries, water quality, and navigation assuming increased frequency and duration of water control structure closures due to future accelerated sea level rise. Although the project includes environmental control structures throughout the levee to maintain tidal ebb and flow, continuing to maintain ecosystem hydrology could be challenging in the future if the intermediate or high relative sea level rise scenarios become a reality. Indirect impacts as described in this report are preliminary, highly speculative, and subject to change. One of the reasons that quantification of project indirect impacts is highly speculative is that wetland loss rates will be impacted by relative sea level rise either with or without the Morganza project in place. Preliminary structure operation plan assumptions and indirect impacts for the constructible features have been determined using the best available information. Indirect impacts for the programmatic features will be addressed in supplemental NEPA documents (refer to RPEIS section 3.5.3 for more information). Mitigation plans will be subject to monitoring and adaptive management.

10.4   Cost and Schedule Risk

The cost and schedule risk analysis is intended to provide project leadership with contingency information for scheduling, budgeting, and project control purposes, as well as to provide tools to support decision making and risk management as projects progress through planning and implementation. These conclusions were reached by identifying and assessing risk items for use in the risk analysis. The quantitative impacts of these risk items are then analyzed using a combination of professional judgment, empirical data, and analytical techniques. The total project cost contingency is computed using the Crystal Ball software.

10.4.1 Project Confidence and Cost Contingencies

Based on the cost and schedule risk analysis, the recommended total project contingency is approximately 35 percent for the structural features and 26 percent for the levees and all other features, based on an 80 percent confidence level. This contingency of almost $2.3 billion was applied to the estimate for the Morganza to the Gulf post-authorization 1% AEP plan.
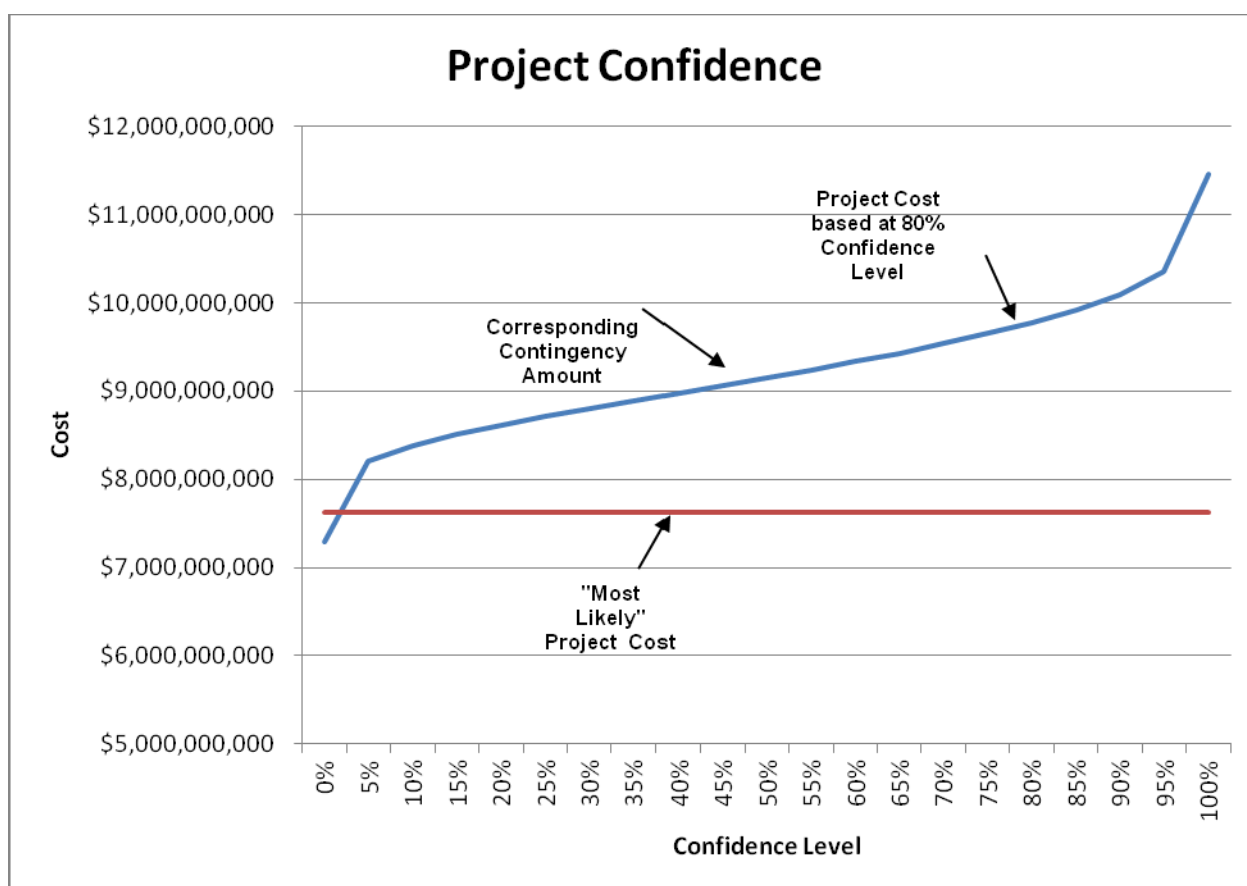
Figure 10-2. Graph of Project Cost vs. Confidence Level
Note: Real estate costs not included because real estate contingencies were computed separately.

10.4.2 High Risk Cost and Schedule Items

An important outcome of the cost and schedule risk analysis is the communication of high risk areas which have a high potential to affect the project cost and/or schedule. For the Morganza to the Gulf PAC project, the high risk cost items are the structural and geotechnical uncertainty, steel cost, fuel cost, unidentified borrow pit for hauled in material, and construction

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modifications. Some of these risk items can be mitigated, reducing the risk of an increased project cost. Mitigation measures for structural and geotechnical uncertainty cannot be determined until further studies and engineering data are obtained.

The risk and cost contingency associated with the current level of geotechnical information is considered acceptable for the purpose of the PAC report; there is always some geotechnical uncertainty associated with limited data available during feasibility studies, and the post-authorization feasibility-level design is based on a typical amount of site data for a feasibility report. The Morganza to the Gulf plan contained in the PAC report is not intended to be the final design, but to be a basis for a cost estimate which would be within 25 percent of the actual cost. A detailed cost risk analysis has been conducted, and although there is a risk that costs could be underestimated, there is also the risk that costs could be overestimated. For example, for the 1% AEP plan, contingencies range from 26 to 35 percent, which results in a contingency of around $2.3 billion as shown in figure 10-2.

The risk analysis already includes a conservative view of the trends for fuel and steel costs based on input from suppliers, research of the market and professional judgment.

Construction modifications are very common and often inevitable on a project of this magnitude. No cost for modifications was included in the base cost estimate and the amount of additional cost due to modifications can be mitigated by ensuring QA/QC guidance is followed and quality products are advertised for construction.

Borrow pit locations for some levee reaches and future levee lifts are unknown at this time. The geology of southeast Louisiana is sufficiently known, however, to reasonability assume that sufficient clay borrow sources would exist within an average distance of 20 to 25 miles to project sites. Encouraging the non-Federal sponsor to find suitable borrow at closest possible distance can mitigate for this risk.

The highest risk schedule item is funding availability. The project schedule was developed based on a reasonable timeframe to establish a level of risk reduction for the area. The USACE has little to no control over the congressional appropriation process other than to fully and timely execute the monies provided in an attempt to show the urgency and importance of the project.

10.5   Site-Adapting HSDRRS Criteria for the Morganza to the Gulf Project

This PAC report reflects the current estimated costs based on a project designed using the HSDRRS guidelines, which are the best available and compliant with current standards. In addition to the PAC analysis, the USACE Risk Management Center and New Orleans District jointly evaluated the proposed Morganza to the Gulf levee system to assess whether HSDRRS criteria could be "site-adapted" to reduce project costs without significantly increasing risk. Based on the preliminary results from the evaluation, the RMC and MVN recommend that the following site-adaptations of the HSDRRS criteria for the Morganza to the Gulf project be considered by the USACE's Chief of Engineering and Construction.

1. **Reduce the Factor of Safety (FoS) for end of construction global stability from 1.5 to 1.3.** The risk assessment team concluded that there is inconsequential change in post-project residual risk for a levee 800 ft wide (associated with global stability FoS = 1.5) versus a 600 ft wide (associated with a global stability FoS = 1.3). This reduction in end of construction factor of safety does increase the likelihood of slope stability failures during construction, which is often unacceptable in an urban environment. However, for the non-urban setting of

this project, slope stability failures during construction can be mitigated during construction at relatively low costs and are unlikely to cause loss of life or significant property damage.

2. **Change the Design Overtopping Rate for well-maintained grass covered levee slopes from 0.1 to 0.5 cfs/ft.** This change could result in reduction of levee and structure elevations by several feet. Based on tests conducted to assess USACE HSDRRs designs, the grass cover on clay levee slopes are generally not expected to fail at average overtopping rates of less than 1 cfs/ft.

3. **Elimination of the structural superiority requirement.** Reducing top elevations of structures to match adjacent levee heights would lead to significantly shorter structures, i.e. reducing structure elevations by 2 ft in addition to the reductions in elevation resulting in the change in design overtopping rate. Desired structure resiliency would be attained through less expensive means.

Additional risk assessment findings specific to the Morganza levee system are as follows:

- **Wave and/or surge overtopping is the predominant source of risk.** Other failure modes such as slope instability, internal erosion through the foundation, front-side erosion on levee slopes, closure structure failure, floodwall failure do not significantly contribute to the amount of risk.

- **Consequences on the western side of the project area are higher than on the eastern side.** Even if levees were overtopped and/or breached, reduced flood depths and delay of flood arrival could result in lower loss of life and property damage, generally in proportion to the distance from the overtopping/breach locations. For a reasonable worst case scenario, the population at risk on the west side is 70,000. On the east side, where the population at risk is further from the proposed alignment, an extreme event with overtopping and breach would likely impact fewer than 1,000 people.

- **From a life safety perspective, risks for both HSDRRS and site-adapted HSDRRS levees exceed tolerable risk guidelines.** The proposed Morganza system would likely reduce life safety risk from existing conditions, however, the post project residual risk for the system using either design approach is likely to remain above the tolerable risk guidelines suggested in ER 1110-2-1156.

Risk-based modifications to current design criteria have the potential to reduce the total project cost estimates reflected in the PAC report. Such modifications would be made to designs and costs during the next phase of implementation, Pre-construction Engineering and Design (PED). The USACE is also conducting a national risk assessment to ensure risk is addressed consistently across the country.

10.6 Post-Authorization Project Changes in Costs and Benefits

Both project costs and benefits have increased significantly since authorization. As shown in table 10-1, project first costs have increased by an order of magnitude. As described in the previous section, the primary reasons for the cost increases are changes in predicted surge elevations and more robust post-Katrina HSDRRS guidelines. As a result, levee lengths, levee and structure heights, and levee widths have increased significantly.

As shown in table 10-2, benefits have increased proportionately to costs, because the same, updated storm surge modeling indicates that more structures have a higher probability of getting

flooded. The 1% AEP surge elevations have increased from 4 to 6 ft, to 12 to 14 ft, so the 1% AEP floodplain is now larger and incorporates more structures. The 2009/2010 PAC inventory included approximately 53,000 structures, which is over twice the number of structures in the original 1997/1998 feasibility study inventory, which included approximately 26,000 structures. Emergency cost reductions for the authorized project were based on pre-Hurricane Katrina/Rita information, and the emergency cost reductions for the post-authorization project were based on post-Hurricane Katrina/Rita information. Also, the emergency cost reductions for the post-authorization project include damages to transportation infrastructure, while these damages were not included in the emergency cost reductions for the authorized project.

The project benefit-to-cost ratio in the 2002 feasibility report was 1.43 based on 2000 price levels and an interest rate of 6.625 percent. The benefit-to-cost ratio of the post-authorization 1% AEP plan is 1.43 based on 2012 price levels and a 3.75 percent interest rate. The benefit-to-cost ratio based on 2012 price levels and a 7.0 percent interest rate is 0.69 (required by ER 1105-2-100, Appendix H, Exhibit H-11).

Table 10-1. Changes in Project First Costs ($ Millions) by Project Feature

Work Breakdown Structure No. & Civil Works Feature Description	Project as Authorized by Congress (WRDA 2007)	Authorized Project (Updated)	3% AEP PAC Alternative	1% AEP PAC Alternative
Effective Price Level	**Oct 2006**	**Oct 2012**	**Oct 2012**	**Oct 2012**
02 Relocations	43	51	274	291
05 Locks	169	228	530	622
06 Fish & Wildlife Facilities	55	66	619	941
11 Levees & Floodwalls	253	314	2,467	5,351
15 Floodway Control & Diversion Structures	219	262	763	1,068
Construction Totals:	$739	$921	$4,652	$8,273
01 Lands and Damages	10	11	339	355
30 Planning, Engineering & Design	87	115	574	1,006
31 Construction Management	50	67	385	631
Project Cost Totals:	**$887**	**$1,113**	**$5,950**	**$10,265**

Note: Authorized project costs in 2006 and 2012 price levels developed using Form 17 (project cost estimates used to support budget submissions).

Table 10-2. Changes in Annual Costs and Benefits ($ Millions)

(All costs and benefits in $millions)	Project as Authorized by Congress in WRDA 2007	Authorized Project (Updated)	3% AEP PAC Alternative	1% AEP PAC Alternative
Effective Price Level, Interest Rate:	Oct 2006, 5.125%	Oct 2012, 3.75%	Oct 2012, 3.75%	Oct 2012, 3.75%
Structures, Contents and Vehicles	91	98	609	959
Emergency Costs	9.6	10	37	54
Boats	1.6	2	<1	<1
Agricultural	2	2	N/A	N/A
Water Supply	0.3	0.3	<1	<1
Avoided Structure Raising Costs	N/A	N/A	10	10
Total Equivalent Annual Benefits	104	113	657	1,023
Annual Costs	49	36	442	716
Benefit-Cost Ratio	2.12	3.16	1.48	1.43
Net Benefits	55	77	214	307

Note: Benefits for boats in 2006 price levels updated to 2012 price levels using the Energy Information Administration Diesel Fuel Price on Gulf Coast; Agricultural benefits updated using the National Agricultural Statistical Service ratio of prices received and paid; all other benefit categories updated using the Composite Index Civil Works Construction Cost Index System. Agricultural benefits were calculated for the 2002 authorized plan, but not for the PAC because a certified model was not available to incorporate risk analysis, and the agricultural benefits were a small percentage of the total PAC benefits.

No changes in cost allocation have occurred since authorization; all costs are for hurricane and storm damage reduction. The post-authorization project does not include any changes in the local cooperation requirements or changes in Federal/non-Federal cost share percentages. The cost apportionment would be 65 percent Federal and 35 percent non-Federal for construction.

The non-Federal sponsor requests that the entire non-Federal share be provided as work-in-kind rather than cash. The non-Federal sponsor would focus their effort on earthen levee construction (multiple lifts) concentrated between Reach E-2 and Reach L. The non-Federal sponsor would also construct floodgates on some bayous within the same geographical area, such as Bush Canal, Placid Canal, Bayou Pointe aux Chenes, Bayou Terrebonne, Humble Canal and Bayou Petit Caillou. Details regarding specific features and schedules for work-in-kind would continue to be coordinated between the USACE and the non-Federal sponsor throughout the design and construction phase of the project. Table 10-3 compares the Federal and non-Federal cost-share of the authorized project and post-authorization project.

Table 10-3. Changes in Cost Apportionment (Costs in $1000s)

(All costs in $1000s)	Authorized Project		Post Authorization 1% AEP Plan
	Authorized in WRDA 2007	Updated Price Levels	
Effective Price Level:	Oct 2006	Oct 2012	Oct 2012
Total Project Cost	$886,700	$1,113,000	$10,265,100
Federal Share (65%)	576,355	723,450	6,672,315
Non-Federal Share (35%)	310,345	389,550	3,592,785
Value of Proposed Work-in-Kind	139,655	177,934	2,947,262
LERRDs	51,970	60,357	645,523
Additional Cash Required	118,720	151,259	0

Note: Authorized project costs (including LERRDs) in 2006 and 2012 price levels developed using Form 17 (project cost estimates used to support budget submissions). Authorized project work-in-kind and cash breakdown in 2006 and 2012 price levels estimated based on percentage from 2002 feasibility report (work-in-kind was 45% of non-Federal share).

10.7 Project Benefits, Costs, and BCR Risk and Uncertainty

Although the costs, benefits, and project benefit-to-cost ratio are presented as "most likely" values in this report, there are large uncertainties surrounding single values. Uncertainty and variability are intrinsic in water resources planning, modeling, and design. Risk and uncertainty arise from measurement errors and from the underlying variability of complex natural, social, and economic situations. Major sources of uncertainty associated with the Morganza to the Gulf project that could impact the benefit-cost ratio include the following:

- **Uncertainty surrounding engineering models and inputs.** The uncertainty surrounding three key engineering parameters (ground elevations, stage-probability curves, and performance of existing and proposed levees) was quantified in the HEC-FDA model. Uncertainty in these engineering inputs arises from imperfect data and coastal storm damage models that represent complex meteorological and hydrologic conditions across a highly variable and changing coastal landscape. In general, the standard deviation associated with estimated still water elevations is greater than a foot. In reviewing the HEC-FDA risk analysis, the USACE Risk Management Center concluded that the lack of data on existing levees resulted in underestimating the without-project damages. The analysis performed on the with-project conditions understated the performance of the Federal levees as well as overstated the residual damages due to the interior/exterior relationships used in the economic model. The net effect of understating the without-project damages and overstating the with-project residual damages is that the benefit-cost ratio is understated.

- **Uncertainty surrounding economic inputs.** The uncertainty surrounding four key economic variables (structure values, contents-to-structure value ratios, first floor elevations, and depth-damage relationships) was quantified in the HEC-FDA model. Uncertainties arise from measurement errors associated with collection of this economic input data and extrapolation of a small set of samples to a large, highly variable study area. The samples used to develop the contents-to-structure value ratios (CSVRs) for the various residential and non-residential damage categories could not be randomly selected due to the difficulty in obtaining volunteers to participate in the surveys. Consequently, there is the risk that the CSVRs used in the economic analysis may not accurately reflect the true mean CSVRs of the population of residential and non-residential properties in the evaluation area. The uncertainty surrounding the first floor elevations assigned to structures is less than a third of a foot; however, adjustments were made to the structure

inventory assuming that structures that are repetitively damaged (i.e. in the 10-yr damage floodplain) will be rebuilt over time to higher elevations such that they are outside of the 100-yr floodplain. If more structures than expected are removed from the floodplain, the benefits would be lower than estimated. If fewer structures than expected are removed from the floodplain, the benefits would be higher than estimated.

- **Changes in the discount rate.** The 2013 discount rate of 3.75 percent is at a record low. All other variables being equal, the breakpoint for project justification is around a discount rate of 5.25 percent, meaning that if the discount rate rises above 5.25 percent and all other variables remain unchanged, the project would no longer have a benefit-cost ratio greater than unity.

- **RSLR uncertainty**. The National Research Council predicts that eustatic sea level (not accounting for subsidence) could increase 1 to 4 feet by 2085. Assuming that historic, local subsidence rates continue into the future, the amount of RSLR (sea level rise plus subsidence) for the Morganza to the Gulf area could range from 1.7 to 4.8 ft. The RSLR scenario selected for calculating the benefit-cost ratio assumes an intermediate level of approximately 2.4 ft RSLR by 2085. If the combined effect of RSLR is lower than expected, project benefits and costs would both be lower than estimated. If actual RSLR is higher than 2.4 ft, project benefits and costs would be higher than estimated. Based on a sensitivity analysis, the net effect on the benefit-cost ratio is minimal (a few tenths of a point) and the project would still be justified under any RSLR scenario (holding all other variables the same).

- **Total project cost uncertainties.** For the Morganza to the Gulf project, the high risk cost items are the structural and geotechnical uncertainty, steel cost, fuel cost, unidentified borrow pit for hauled in material, and construction modifications. A detailed cost risk analysis has been conducted and resulting project feature contingencies range from 26 to 35 percent, which results in a total project contingency of around $2.3 billion. Although there is a risk that costs could be underestimated, there is also the risk that costs could be overestimated.

- **Mitigation cost uncertainties for indirect impacts.** Compensatory mitigation costs for the 1% AEP plan are based on direct construction impacts assuming that tidal ebb and flow would be maintained by the environmental control structures designed throughout the levee alignment such that there would be minimal indirect impacts. However, if future sea level rise predictions come to fruition, these environmental control structures would have to be closed more frequently resulting in indirect impacts. Based on this assumption, the Habitat Evaluation Team estimates the constructible features could impact an additional 721 to 1,442 acres (216 to 418 AAHUs), which would require additional compensatory mitigation at a cost of $60 million to $120 million. Mitigation costs resulting from additional indirect impacts from the programmatic features would be covered in future supplemental NEPA documents.

- **Site-adapting the HSDRRS standards.** Estimated project costs in the PAC report are the best available and compliant with current HSDRRS standards. Based on a risk assessment of the Morganza to the Gulf project conducted after the PAC analysis was complete, risk-based modifications to current design criteria have the potential to reduce the total project cost estimates reflected in the PAC report. The USACE is also conducting a risk assessment to ensure risk is addressed consistently across the country.

Once this assessment is complete, the results may be applied to the Morganza to Gulf project area. Such modifications would be made to designs and costs during the next phase of implementation, Pre-construction Engineering and Design (PED). Any decrease in costs would increase the benefit-cost ratio assuming the benefit remain unchanged.

11 Findings

The District Commander has updated the Morganza to the Gulf of Mexico, Louisiana authorized project and finds that the updated 1% AEP plan is economically justified, environmentally acceptable and engineeringly sound. Post-Katrina engineering design criteria and standards for gulf coast communities were applied to reduce the potential of loss of life and property from coastal storms. These engineering practices were developed using the findings of the *Interagency Performance Evaluation Task Force* including key lessons learned from Hurricane Katrina and their implications for future hurricane preparedness and planning for south Louisiana. Project modifications were also found necessary to address developments after the project was authorized, including community resettlement patterns after Katrina, to incorporate improved water control elements and floodgates, and to update other outmoded aspects of the authorized project to more effectively provide the utility of function originally intended by Congress.

The District Commander has considered all the significant aspects of this study including the environmental, social, and economic effects, the engineering feasibility, and the comments received from other resource agencies, the non-Federal sponsors, and the public and has determined that the updated authorized project presented in this report remains in the overall public interest and is a justified expenditure of Federal funds. In coordination with other Federal, State of Louisiana, and parish agencies, USACE planned and would design a project that serves the needs of the nation.

The updated total cost for the project is $10,265,100,000 (October 2012 dollars) inclusive of associated investigation, environmental, engineering and design, construction, supervision and administration, and contingency costs (and exclusive of OMRR&R costs). The fully funded total project cost (includes inflation) is approximately $12,872,846,000. The project would be funded 65 percent by the Federal Government and 35 percent by the non-Federal sponsors, and subject to the implementation requirements specified in section 8 of this report.

The OMRR&R costs of this project are estimated to be approximately $7,400,000 annually. Approximately $1,700,000 annually would be a Federal responsibility, including OMRR&R for the GIWW floodgates and the HNC lock complex and a portion of cost-shared mitigation OMRR&R. The remaining annual OMRR&R costs of approximately $5,700,000 would be the responsibility of the non-Federal sponsor, including OMRR&R for all project features (except the GIWW floodgates and HNC lock complex), the sponsor funded additional work item, a portion of cost-shared mitigation OMRR&R, and the remainder of mitigation OMRR&R once USACE determines that the mitigation has achieved initial success criteria.

The benefit-to-cost ratio for the post-authorization project is 1.43 based on October 2012 price levels and a 3.75 percent interest rate.

The findings contained herein reflect the information available at the time and current Department of the Army policy and guidance and specific guidance provided for the Morganza to the Gulf of Mexico, Louisiana project. They do not reflect programming and budgeting priorities inherent in the formulation of national civil works construction program nor the perspective of higher review levels within the executive branch. Consequently, the findings may be modified before they are transmitted to Congress. However, prior to the transmission to Congress, the sponsor, State, interested Federal agencies, and other parties will be advised of any modifications and will be afforded the opportunity to comment further.

Edward R. Fleming
Colonel, US Army
District Commander

12 References

This section includes the list of references cited in the report, a glossary of terms, and list of acronyms. Many of the report documents are available online at www.mvn.usace.army.mil/prj/mtog/.

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12.2 Glossary

Alternative or Alternative Plan – A set of one of more management measures functioning together to address one or more objectives.

Annual Chance Exceedance (ACE) – A measure of the likelihood (expressed as a probability) of a flood event reaching or exceeding a particular magnitude. A one percent (ACE) flood event has a one percent (or 1 in 100) chance of occurring or being exceeded at a location in any year.

Annual Exceedance Probability (AEP) – The probability that flooding will occur in any given year considering the full range of possible annual floods (ER 1105-2-101), or the chance of an event of a given size (or larger) occurring in any one year, usually expressed as a percentage (EC 1110-2-6066).

Anthropogenic – Caused by human activity.

Assurance – The probability that a target stage will not be exceeded during the occurrence of a specified flood. Assurance is also interchangeable with the term **conditional non-exceedance probability** (EC 1110-2-6066).

Benefits – Valuation of positive performance measures.

Bottomland Hardwood Forest – Low-lying forested wetlands found along streams and rivers.

Brackish Marsh – Intertidal plant community typically found in the area of the estuary where salinity ranges between 4 and 15 ppt.

Coastal – Characterization of a structure or system that is situated in an environment subject to water level fluctuations and wave action. Coastal structures or systems must typically be designed to take into account storm surge and wave loadings (EC 1110-2-6066).

Conditional Non-exceedance Probability (CNP) – The probability that a target stage will not be exceeded during the occurrence of a specified flood. Conditional non-exceedance probability is also interchangeable with the term **assurance** (EC 1110-2-6066).

Control Structure – A gate, lock, or weir that controls the flow of water.

Cumulative Impacts – The combined effect of all direct and indirect impacts to a resource over time.

Datum – A point, line, or surface used as a reference, as in surveying, mapping, or geology.

Direct Impacts – Those effects that result from the initial construction of a measure. Contrast with "Indirect Impacts."

Economic – Of or relating to the production, development, and management of material wealth, as of a country, household, or business enterprise.

Economic Analysis Period – The period of time over which the flood risk management system would have significant beneficial or adverse effects. Usually this period does not exceed 50 years except for major multi purpose reservoir projects (EC 1110-2-6066).

Ecosystem – An organic community of plants and animals viewed within its physical environment (habitat); the ecosystem results from the interaction between soil, climate, vegetation and animal life.

Ecosystem Restoration – Activities that seek to return an organic community of plants and animals and their habitat to a previously existing or improved natural condition or function.

Endangered Species – Animals and plants that are threatened with extinction.

Environmental Impact Statement (EIS) – A document that describes the positive and negative environmental effects of a proposed action and the possible alternatives to that action. The EIS is used by the federal government and addresses social issues as well as environmental ones.

Estuary – A semi-enclosed body of water with freshwater input and a connection to the sea where fresh water and salt water mix.

Estuarine – Related to an estuary.

Feasibility Report – A description of a proposed action previously outlined in a general fashion in a Reconnaissance Report that will satisfy the Federal interest and address the problems and needs identified for an area. It must include an assessment of impacts to the environment (either in an Environmental Assessment, or the more robust Environmental Impact Statement), an analysis of alternative methods of completion, and the selection of a Recommended Plan through the use of a cost-effectiveness analysis.

Feature – A constructible increment of an alternative plan.

Final Array – The final grouping of alternative plans from which a final recommendation can be made.

Floodgate – The term "floodgate" is applied to a flood risk reduction structure that opens and closes. During non-flood conditions, a floodgate remains in an "open" position and allows for vehicle passage. Floodgates are erected across roads, railroads and navigable waterways.

Forced drainage levee system – A levee and pump system that provides controlled drainage for an area, normally to allow the area to remain dry at or below sea level under normal tidal conditions.

Fresh Marsh – Intertidal herbaceous plant community typically found in that area of the estuary with salinity ranging from 0 to 3 ppt.

Goals – Statements on what to accomplish and/or what is needed to address a problem without specific detail.

Gradient – A slope; a series of progressively increasing or decreasing differences in a system or organism.

Habitat – The place where an organism lives; part of physical environment in which a plant or animal lives.

Habitat Loss – The disappearance of places where target groups of organisms live. In coastal areas, usually refers to the conversion of marsh or swamp to open water.

Hazardous, Toxic, and Radioactive Wastes (HTRW) – Projects features must be examined to ensure that their implementation will not result in excessive exposure to pollutants possibly located in the study area.

Herbaceous – A plant with no persistent woody stem above ground.

Hydraulics - The scientific study of water and other liquids, in particular their behavior under the influence of mechanical forces and their related uses in engineering (from The American Heritage® Science Dictionary.)

Hydrodynamics – The continuous change or movement of water.

Hydrology – The pattern of water movement on the earth's surface, in the soil and underlying rocks, and in the atmosphere.

Indirect Impacts – Those effects that are not as a direct result of project construction, but occur as secondary impacts due to changes in the environment brought about by the construction. Contrast with "Direct Impacts."

Infrastructure – The basic facilities, services, and installations needed for the functioning of a community or society, such as transportation and communications systems, water and power lines, and public institutions including schools, post offices, and prisons.

Intermediate Marsh – Intertidal herbaceous plant community typically found in that area of the estuary with salinity ranging from 2 to 5 ppt.

Intertidal – Alternately flooded and exposed by tides.

I-Wall – A slender cantilever wall, deeply embedded in the ground or in an embankment that rotates when loaded and is thereby stabilized by reactive lateral earth pressures (EC 1110-2-6066).

Levee – A linear mound of earth or concrete floodwall built to stop or slow down storm surge.

Levee System – A levee system comprises one or more levee segments which collectively provide flood damage reduction to a defined area. The levee system is inclusive of all features that are interconnected and necessary to ensure protection of the associated separable floodplain (EC 1110-2-6066).

Level-of-Protection – The recurrence interval of the flood event that, with a high level of assurance, will be safely contained within the capacity of the protection system (EC 1110-2-6066). For example, a system could be deemed a 100-year Level-of-Protection system if it can contain the 1 percent annual chance exceedance flood with a high degree of assurance, e.g. 90 percent.

Sponsor Funded Additional Work Item – Alternative feature preferred by non-Federal sponsor for which the non-Federal sponsor agrees to pay the full cost increment.

Management Measure (or Measure) – A feature (a structural element that requires construction or assembly on-site) or an activity (a nonstructural action) that can be implemented at a specific geographic site that is to address one or more planning objectives. Management measures are the building blocks of alternative plans (Planning Guidance Notebook, 2-4).

Methodology – A set of practices, procedures, and rules.

Modes of Failure – The mechanism by which a system or structure ceases to perform its intended function (EC 1110-2-6066).

National Environmental Policy Act (NEPA) – Ensures that Federal agencies consider the environmental impacts of their actions and decisions. NEPA requires all Federal agencies to consider the values of environmental preservation for all significant actions and prescribes procedural measures to ensure that those values are fully respected.

No Action Alternative – An alternative that describes the without-project condition if no action is taken.

Objectives – More specific statements than "Goals," describing how to achieve the desired targets.

Organic – Composed of or derived from living things.

Overtopping – A static and/or dynamic event that occurs when the height of the stillwater level and/or associated waves exceed the top of a structure or levee embankment.

Prime Farmland – Land that has the best combination of physical and chemical characteristics for producing food, feed, fiber, forage, oilseed, and other agricultural crops with minimum inputs of fuel, fertilizer, pesticides, and labor, and without intolerable soil erosion. One of the categories of concern in the EIS.

Probability – A measure of the likelihood, chance, or degree of belief that a particular outcome or consequence will occur. A probability provides a quantitative description of the likelihood of occurrence of a particular event. This is expressed as a value between 0 and 1 (EC 1110-2-6066).

Probability Function – A discharge-exceedance or stage-exceedance probability relationship for a reach developed by traditional, site-specific, hydrologic engineering analysis procedures (EC 1110-2-6066).

Programmatic Environmental Impact Statement (PEIS) – An Environmental Impact Statement that supports a broad authorization for action, contingent on more specific detailing of impacts from specific measures.

Project – A constructible increment of an alternative plan.

Quantitative – Able to assign a specific number; susceptible to measurement.

Reach – The term Reach is used to describe two distinct features in the PAC analysis. (1) Levee Reach refers to a segment of the proposed Morganza to the Gulf levee alignment that has similar levee elevations, ground surface elevations, soil types, and foundation strengths. The ten primary Levee Reaches are Barrier, A, B, E, F, G, H, I, J, K, and L. (2) Hydraulic and/or Economic Reach refers to an area that has similar water surface elevations and is used to aggregate economic damages and benefits. The Morganza study area has been divided into 276 Hydraulic/Economic Reaches, which are also sometimes referred to as Storage Areas.

Reconnaissance Report – A document prepared as part of a major authorization that examines a problem or need and determines if sufficient methods and Federal interest exists to address the problem/need. If so, then a "Feasibility Report" is prepared, which details the solution and its impacts further.

Redundancy – The duplication of critical components of a system with the intention of increasing reliability of the system, usually in the case of a backup or fail-safe (EC 1110-2-6066).

Relative Sea Level Rise (RSLR) – The sum of the sinking of the land (subsidence) and eustatic sea level change; the change in average water level with respect to the surface.

Reliability – The probability of a component, unit or system adequately performing its intended purpose, for a specified period of time, under given operating conditions (EC 1110-2-6066).

Residual Risk – The flood risk that remains in the floodplain after a proposed flood risk reduction project is implemented. Residual risk includes the consequence of capacity exceedance as well as consideration of project performance.

Resilience – The ability to avoid, minimize, withstand, and recover from the effects of adversity, whether natural or manmade, under all circumstances of use (EC 1110-2-6066).

Risk – The probability an area will be flooded, resulting in undesirable consequences (ER 1105-2-101).

Risk Analysis – An approach to evaluation and decision making that explicitly, and to the extent practical, analytically, incorporates considerations of risk and uncertainty in a flood damage reduction study (ER 1105-2-101).

Robustness – The ability of a system to continue to operate correctly across a wide range of operational conditions (the wider the range of conditions, the more robust the system), with minimal damage, alteration or loss of functionality, and to fail gracefully outside of that range (EC 1110-2-6066).

Safety – Commonly thought of as the condition of being free from danger, risk, or injury; however, safety is not something that can be absolutely achieved or guaranteed. Instead safety is the condition to which risks are managed to tolerable levels. Therefore, safety is a subjective concept based upon individual perceptions of risks and their tolerability.

Saline Marsh – Intertidal herbaceous plant community typically found in that area of the estuary with salinity ranging from 12 to 32 ppt.

Salinity – The concentration of dissolved salts in a body of water, commonly expressed as parts per thousand (ppt). The salinity of ocean water is approximately 35 ppt.

Scoping – Soliciting and receiving public input to determine issues, resources, impacts, and alternatives to be addressed in the draft EIS.

Social – Relating to human society and its modes of organization.

Socioeconomic – Involving both social and economic factors.

Still Water Level or Elevation – The height of the water surface measured above a datum without waves.

Storm Surge – An abnormal and sudden rise of the sea along a shore as a result of the winds of a storm.

Structural Superiority – All new structures that are difficult to construct due to their nature, such as railroad and highway gate monoliths that require detours causing disruptions to traffic, pumping station fronting protection that require cofferdams within their discharge basins causing reductions to pumping capacity, sector gated structures causing disruptions to navigation, large utility crossings, etc., shall be designed with a minimum of 2 ft. of additional wall height.

Subsidence – The gradual downward settling or sinking of the Earth's surface with little or no horizontal motion.

Tainter Gate - Radial gate most commonly used on navigational projects. In its simplest form, a tainter gate is a segment of a cylinder mounted on radial arms that rotate on trunnions anchored to the piers. Because of its simple design, relatively light weight, and low hoist-capacity requirements, the tainter gate is considered one of the most economical and most suitable gates for controlled spillways.

T-wall – A cantilever T-type reinforced concrete floodwall consists of a concrete stem and base slab which form an inverted T. The structural members are fully reinforced to resist applied moments and shears. As necessary to resist under seepage during a flood event, a steel sheet pile cutoff wall is cast into the base slab of the T-Wall. The T-wall may need to be supported on a pile foundation if soft soils exist. The piles can be made of either steel or concrete and derive their support to the T-wall from friction of the soil surrounding the pile or through end bearing at a deeper stronger soil strata.

Uncertainty – A measure of imprecision of knowledge of parameters and functions used to describe the hydraulic, hydrologic, geotechnical, and economic aspects of a project plan (ER 1105-2-101).

Unique Farmland – Land other than Prime Farmland (see "Prime Farmland") that is used for the production of specific high-value food and fiber crops, such as citrus, tree nuts, olives, cranberries, fruits, and vegetables.

Upland – A general term for non-wetland elevated land above low areas along streams or between hills.

Water Resources Development Act (WRDA) – A bill passed by Congress that provides authorization and/or appropriation for projects related to the conservation and development of water and related resources.

12.3 Measurement Abbreviations

AAHU	Average Annual Habitat Unit
cfs	cubic feet per second
cfs/ft	cubic feet per second per linear foot
ft	foot or feet
ppm	parts per million
ppt	parts per thousand

12.4 Acronyms

Acronyms used in the RPEIS are not listed here because the RPEIS has its own list of acronyms.

ACE	Annual Chance Exceedance
AEP	Annual Exceedance Probability
BFE	Base Flood Elevation
BLH	Bottomland Hardwood
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CFR	Code of Federal Regulations
CPRA	Coastal Protection and Restoration Authority (formerly the Office of Coastal Protection and Restoration)
CPRAB	Coastal Protection and Restoration Authority Board (formerly the CPRA)
CSVR	Content-to-Structure Value Ratio (see Economic Appendix)
CWCCIS	Composite Index Civil Works Construction Cost Index System
CWPPRA	Coastal Wetlands Planning, Protection, and Restoration Act
DNR	Department of Natural Resources
DOTD	Department of Transportation and Development
EC	Engineering Circular
EIS	Environmental Impact Statement
ER	Engineering Regulation
FEMA	Federal Emergency Management Agency (see Economic Appendix)
FIRM	Flood Insurance Rate Map
GIS	Geographic Information System

GIWW	Gulf Intracoastal Waterway
HEC-FDA	Hydrologic Engineering Center - Flood Damage Analysis
HNC	Houma Navigation Canal
HSDRRS	Hurricane and Storm Damage Risk Reduction System
LACPR	Louisiana Coastal Protection and Restoration
LCA	Louisiana Coastal Area
LERRD	Lands, Easements, Rights-Of-Way, Relocation, and Disposal Areas
LiDAR	Light Detection and Ranging
MLODS	Multiple Lines of Defense Strategy
MOU	Memorandum of Understanding
NAVD	North American Vertical Datum
NEPA	National Environmental Policy Act
NGO	Non-Governmental Organization
NGVD	National Geodetic Vertical Datum
NMFS	National Marine Fisheries Service
OMRR&R	Operation and Maintenance, Repair, Replacement and Rehabilitation
OSE	Other Social Effects (see Economic Appendix)
PAC	Post Authorization Change
PED	Preconstruction Engineering and Design
PEIS	Programmatic Environmental Impact Statement
PL	Public Law
PPA	Project Partnership Agreement
RED	Regional Economic Development (see Economic Appendix)
RPEIS	Revised Programmatic Environmental Impact Statement
RSLR	Relative Sea Level Rise
SLLD	South Lafourche Levee District
TLCD	Terrebonne Levee Conservation District
TSP	Tentatively Selected Plan
USACE	U.S. Army Corps of Engineers
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service
WMA	Wildlife Management Area
WRDA	Water Resources Development Act
WVA	Wetland Value Assessment

Attachment 1 – Project Map

Map legend:

- **MODIFIED ALIGNMENT**
- **EXTENSIONS to the MODIFIED ALIGNMENT**
- **AUTHORIZED ALIGNMENT**
- ○ **ENVIRONMENTAL CONTROL STRUCTURE**
- ▭ **NAVIGABLE STRUCTURE**
- ▬ **ROADWAY GATE**
- ▣ **PUMP STATION**
- ⚓ **LOCK**

ASSUMPTION PARISH
GIBSON
ST. CHARLES PARISH

Proposed Bayou Black Floodgate
Proposed Shell Canal West Floodgate
Proposed Shell Canal East Floodgate
Proposed Elliot Jones Floodgate
Proposed Humphreys Canal Floodgate
Barrier Reach
Bayou Black Pump Station
Proposed NAFTA Roadway Gate
Hanson Canal Pump Station
Proposed G.I.W.W. Floodgate West
Proposed Minors Canal Floodgate
Proposed Marmande Canal Floodgate
Proposed Falgout Canal Floodgate
Proposed Bayou DuLarge Floodgate and Highway 315 Roadway Gate
Proposed Bayou Grand Caillou Floodgate
Proposed Houma Navigation Canal Lock
Proposed Four Point Bayou Floodgate and Roadway Gate
Proposed Bayou Petite Caillou Floodgate and Highway 56 Roadway Gate

LAFOURCHE PARISH
TERREBONNE PARISH
HOUMA
BOURG
Lake Long
Lake Theriot
Lake Decade
Lake Boudreaux
DULAC
BOUDREAUX
CHAUVIN
MONTEGUT

Union Pacific RR Roadway Gate
LOCKPORT
Bayou Lafourche
Proposed G.I.W.W. Floodgate East
Larose Floodgate
Hwy 24 & 3235 Roadway Gates
LAROSE
Lockport to Larose Reach
Larose C. North Reach
Proposed Grand Bayou Floodgate
Grand Bayou Canal
Proposed Pointe Aux Chenes Floodgate and Highway 665 Roadway Gate
Pointe Aux Chenes Pump Station
Bayou Pointe Au Chien

Humble Canal Floodgate
Madison (Nettleton) Pump Station
Humble Canal
Bayou Terrebonne Floodgate and Highway 55 Roadway Gate
Bush Canal Floodgate
Placid Canal Floodgate

G.I.W.W.
Houma Navigation Canal
Terrebonne Bayou
Marmande Canal
Falgout Cnl.
Bayou du Large
Bayou Grand Caillou
Bayou Petite Caillou

SCALE: 1" = 6000'
6000 0 6000 12000

EGIS Map ID No. 09-IWA Other
MTOG Alternate Alignments

Attachment 2 – Economic Reach Maps

MTOG Economic Reaches

MTOG Economic Reaches Behind Local Levees

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

- MTOG Construction Reaches
- Lock
- Federal Water Control Structure
- Interim Control Structure
- Sector Gate
- Channel Floodgate
- Weir
- Control Structure
- Diversion Structure
- Drainage Structure
- Navigable Structure
- Cities
- Parishes

Last Modified : 2/24/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1:000000

Morganza to The Gulf
MTOG Economic Reaches Behind Local Levees

EGIS Map ID No. 12_027-002

Attachment 3 – Inundation Depth Maps

MTOG Inundation Depth Grid 2010 50-Year
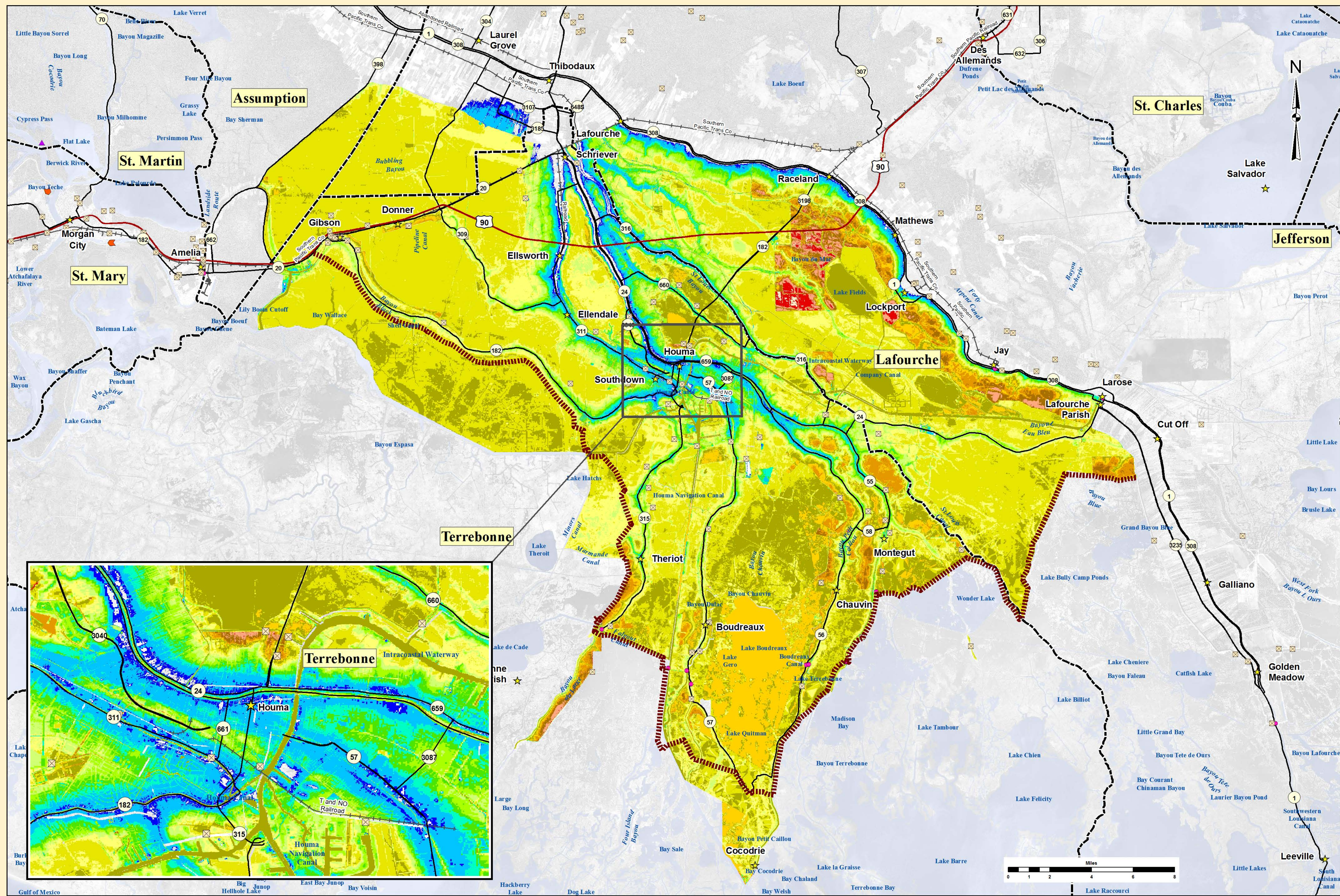
U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

Inundation Depth Map 2010 50-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
- 12 ft - 13 ft
- 13 ft - 14 ft
- 14 ft - 15 ft
- 15 ft - 60 ft

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- Control Structure
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- Navigable Structure
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- Parishes

Last Modified : 2/22/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2010 50-Year

EGIS Map ID No. 12_027-002
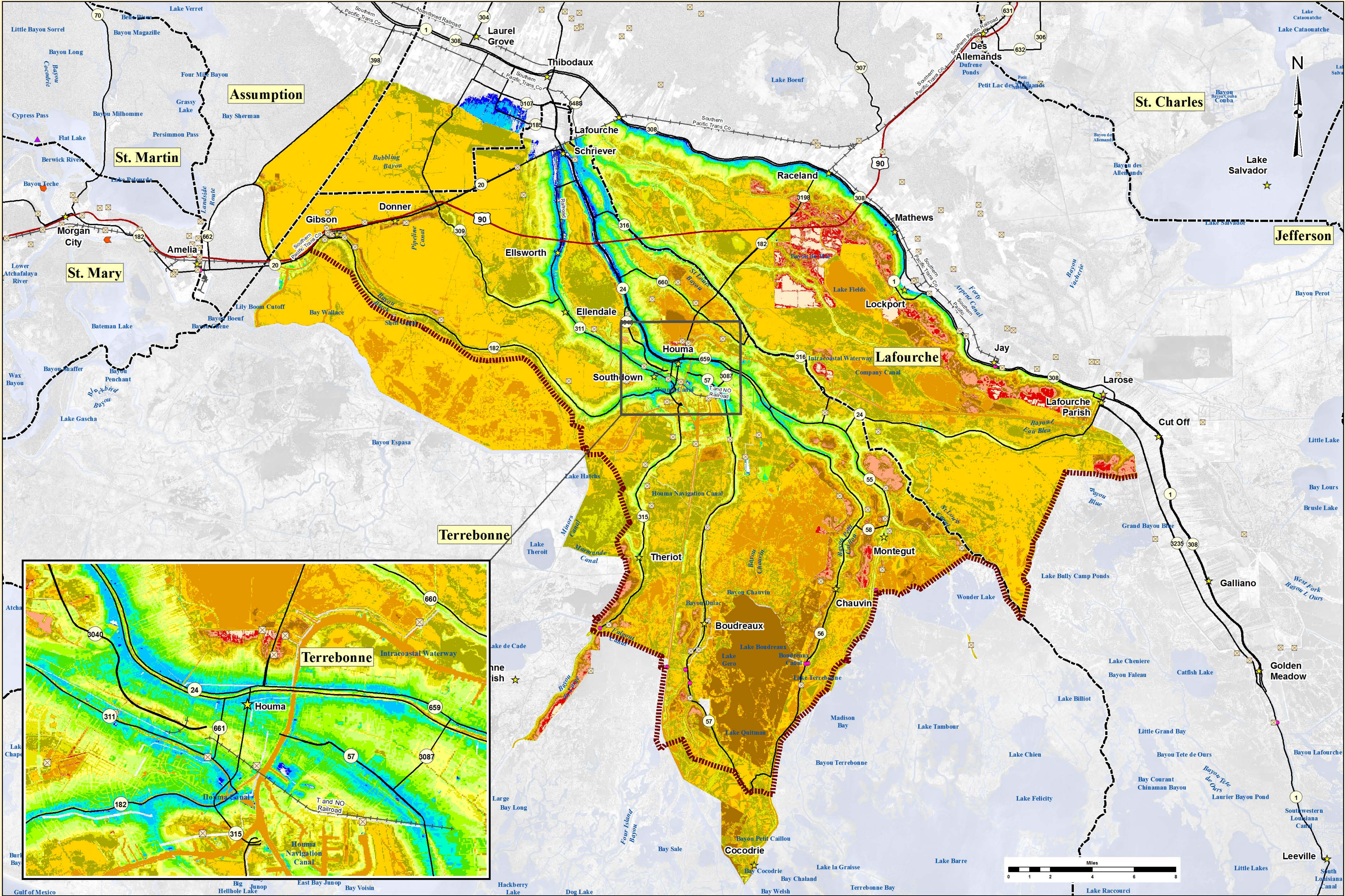
MTOG Inundation Depth Grid 2010 100-Year

Legend

Inundation Depth Map 2010 100-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
- 12 ft - 13 ft
- 13 ft - 14 ft
- 14 ft - 15 ft
- 15 ft - 60 ft

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U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office
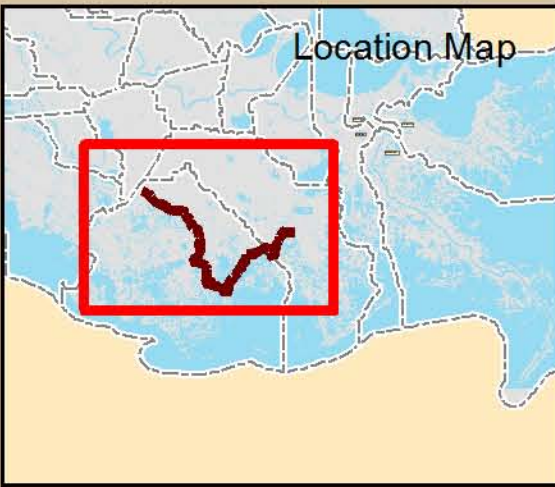
Last Modified : 2/22/2012

Location Map

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Projected coordinate system name:
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Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inundation Depth Grid
2010 100-Year

Miles
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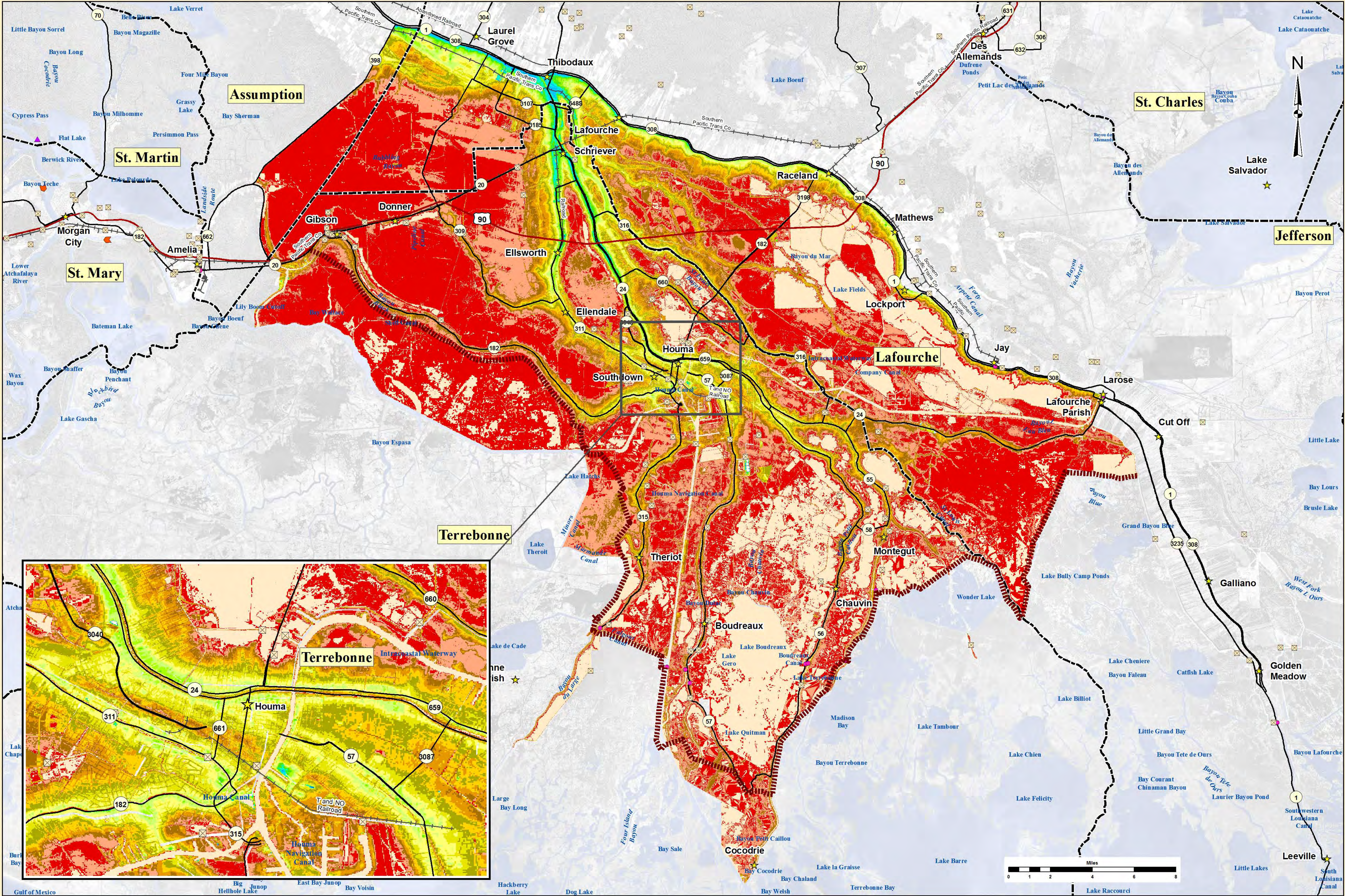
EGIS Map ID No. 12_027-002

MTOG Inundation Depth Grid 2010 500-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

EGIS

Legend

Inundation Depth Map 2010 500-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
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- 15 ft - 60 ft

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Last Modified : 2/22/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2010 500-Year

N

St. Martin
Assumption
St. Charles
Jefferson
St. Mary
Terrebonne
Lafourche

Morgan City
Amelia
Thibodaux
Laurel Grove
Lafourche
Schriever
Raceland
Mathews
Lockport
Des Allemands
Lake Salvador
Gibson
Donner
Ellsworth
Ellendale
Houma
Southdown
Jay
Larose
Lafourche Parish
Cut Off
Theriot
Montegut
Chauvin
Boudreaux
Galliano
Golden Meadow
Cocodrie
Leeville

Terrebonne (inset)

Houma
Intracoastal Waterway
Houma Canal
Houma Navigation Canal

Miles
0 1 2 4 8

EGIS Map ID No. 12_027-002

MTOG Inundation Depth Grid 2035 50-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

Inundation Depth Map 2035 50-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
- 12 ft - 13 ft
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- 15 ft - 60 ft
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Last Modified : 2/22/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2035 50-Year

EGIS Map ID No. 12_027-002

MTOG Inundation Depth Grid 2035 100-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

Inundation Depth Map 2035 100-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
- 12 ft - 13 ft
- 13 ft - 14 ft
- 14 ft - 15 ft
- 15 ft - 60 ft

MTOG Construction Reaches
Lock
Federal Water Control Structure
Interim Control Structure
Sector Gate
Channel Floodgate
Weir
Control Structure
Diversion Structure
Drainage Structure
Navigable Structure
Cities
Parishes

Last Modified : 2/22/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2035 100-Year

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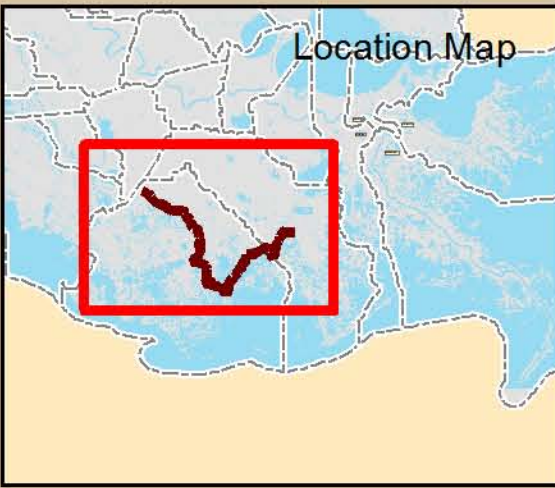
MTOG Inundation Depth Grid 2035 500-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

EGIS

N

Legend

Inundation Depth Map 2035 500-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
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Last Modified : 2/22/2012

Location Map

IMAGERY
Projected coordinate system name:
NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2035 500-Year

Miles

EGIS Map ID No. 12_027-002

MTOG Inundation Depth Grid 2085 50-Year

Legend

Inundation Depth Map 2085 50-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
- 9 ft - 10 ft
- 10 ft - 11 ft
- 11 ft - 12 ft
- 12 ft - 13 ft
- 13 ft - 14 ft
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- Control Structure
- Diversion Structure
- Drainage Structure
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- Parishes

Last Modified : 2/22/2012

Location Map

Morganza to The Gulf
Inudation Depth Grid
2085 50-Year

EGIS Map ID No. 12_027-002

Parish and place labels appearing on the map: Assumption, St. Martin, St. Mary, Morgan City, Amelia, Gibson, Donner, Ellsworth, Ellendale, Houma, Southdown, Theriot, Terrebonne, Boudreaux, Chauvin, Montegut, Cocodrie, Lafourche, Lafourche Parish, Thibodaux, Laurel Grove, Schriever, Raceland, Mathews, Lockport, Larose, Cut Off, Jay, Galliano, Golden Meadow, Leeville, Des Allemands, St. Charles, Jefferson, Lake Salvador

MTOG Inundation Depth Grid 2085 100-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

Inundation Depth Map 2085 100-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
- 8 ft - 9 ft
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- 11 ft - 12 ft
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Last Modified : 2/22/2012

Location Map

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Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

**Morganza to The Gulf
Inudation Depth Grid
2085 100-Year**

EGIS Map ID No. 12_027-002

MTOG Inundation Depth Grid 2085 500-Year

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
Engineering Office

Legend

Inundation Depth Map 2085 500-yr

- 0 ft - 1 ft
- 1 ft - 2 ft
- 2 ft - 3 ft
- 3 ft - 4 ft
- 4 ft - 5 ft
- 5 ft - 6 ft
- 6 ft - 7 ft
- 7 ft - 8 ft
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Last Modified : 2/22/2012

Location Map

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NAD_1983_UTM_Zone_15N
Geographic coordinate system name:
GCS_North_American_1983
Resolution: 1.000000

Morganza to The Gulf
Inudation Depth Grid
2085 500-Year

EGIS Map ID No. 12_027-002

POST AUTHORIZATION CHANGE REPORT APPENDICES

MORGANZA TO THE GULF OF MEXICO, LOUISIANA

MAY 2013

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SWL-A   Lockport to Larose Reach (a) 100 Yr LORR Base Yr Conditions with Overbuild Section
LWL-CN   LGM Reach C-North 100 Yr Base Yr Conditions with Overbuild
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SA-2   Levee Reach B Theoretical Settlement Analysis
SA-3   Levee Reach E Theoretical Settlement Analysis
SA-4   Levee Reach F Theoretical Settlement Analysis
SA-5   Levee Reach G Theoretical Settlement Analysis
SA-6   Levee Reach H Theoretical Settlement Analysis
SA-7   Levee Reach I Theoretical Settlement Analysis
SA-8   Levee Reach J Theoretical Settlement Analysis
SA-9   Levee Reach K Theoretical Settlement Analysis
SA-10   Levee Reach L Theoretical Settlement Analysis
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C-705   Black Bayou Canal 56' Sector Gate Cross Sections
C-709A   Shell Canal West 30' Stop Log Gate 100 Yr Final Plan
C-711   Shell Canal West 35' Sector Gate Cross Sections
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C-748   GIWW West 125' Sector Gate 100 Yr Cross Sections
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S-084A   Environmental Control Structure Section
S-085A   Environmental Control Structure Foundation Plan
S-094A   T-Wall Type 2A-2B
S-095A   T-Wall Type 2C-2E

1   INTRODUCTION

1.1   LOCATION AND DESCRIPTION OF PROJECT AREA

1.2   General

The project/study area is about 60 miles southwest of New Orleans, Louisiana and includes most of Terrebonne Parish and the portion of Lafourche Parish between the eastern boundary of Terrebonne Parish and Bayou Lafourche. The authorized Morganza to the Gulf of Mexico project (MTG) was intended to function as a 1% annual exceedance probability (100-year) coastal storm damage risk reduction system. In addition to flood risk reduction, the structural features of the authorized project were designed to provide tidal exchange, environmental benefits, and navigational passage. See Figure 1 for a map showing the post authorization change Morganza to the Gulf levee alignment. The levee reaches on the western extent (Barrier Reach) and eastern extent (Larose C North and Lockport to Larose Reaches) of the current PAC alignment were not part of the authorized alignment. Note that the Morganza alignment has evolved throughout the PAC analysis and these western and easternmost levee reaches do not appear on every map in this appendix. Refer back to Figure 1 as necessary.
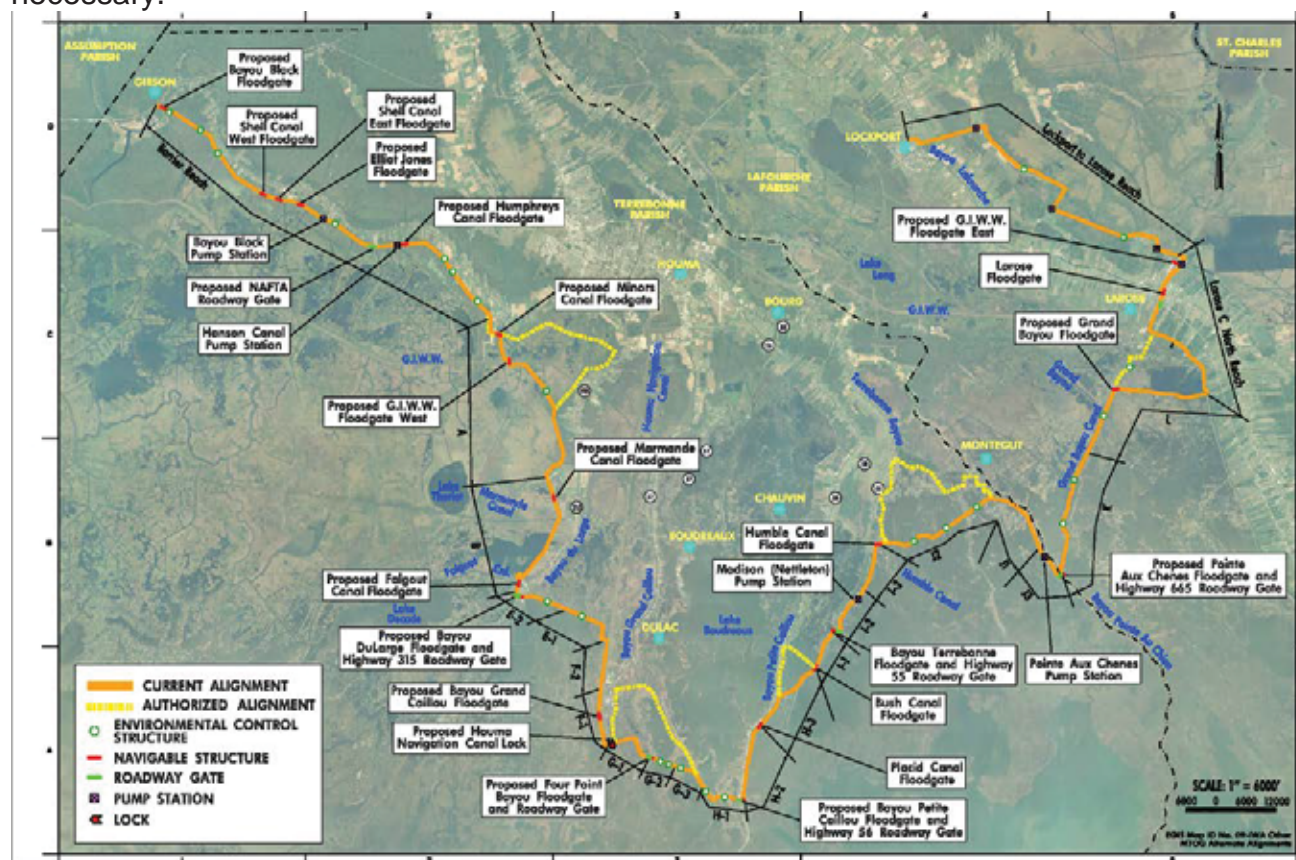
Figure 1 - Morganza to the Gulf Alignment

The PAC report is not a reformulation of the project; it includes an analysis of pre-and post-Katrina 1% Annual Exceedance Probability (AEP) design alternatives, the project levee alignment, and other features integral to the project. The two alternatives considered for this report are shown below-

- 3% AEP Storm Surge Risk Reduction System (3% AEP alternative)
- 1% AEP Storm Surge Risk Reduction System (1% AEP alternative)

1.3   Description of the 1% AEP Alternative

The 1% AEP alternative is a hurricane levee system that provides risk reduction for water levels that have a 1 percent chance of occurring each year. Levee elevations range from 14.0 to 24.0 feet for base year (2035) conditions and from 19.5 to 26.5 feet NAVD88 for future year (2085) conditions assuming relative sea level rise of approximately 2.4 feet by 2085 (intermediate scenario).

1.4   Description of the 3% AEP Alternative

The 3% AEP alternative is a hurricane levee system that provides risk reduction for water levels that have a 3 percent chance of occurring each year. Levee elevations range from 9.0 to 18.0 feet for base year (2035) conditions and from 13.0 to 20.0 feet NAVD88 for future year (2085) conditions assuming relative sea level rise of approximately 2.4 feet by 2085 (intermediate scenario).

1.5   Purpose

This Engineering Appendix outlines the engineering and design work done to support the preparation of the Morganza to the Gulf Post Authorization Change Report. The appendix summarizes modeling, hydraulic design, geotechnical investigations, structural design, levee design and cost estimates. Additional technical details including modeling reports, plates, calculations, soil boring logs, etc. are available at MVN and can be provided upon request. See list of references at the end of this report.

All elevations are referenced to North American Vertical Datum, NAVD 88 unless otherwise noted.

2 CLIMATOLOGY, HYDROLOGY, HYDRAULICS, AND WATER QUALITY

2.1 CLIMATOLOGY

2.1.1 Climate

The climate in the area is humid subtropical and is subject to significant polar influences during the winter as a cold air masses periodically move southward over the area displacing warm moist air. Prevailing southerly winds create a strong maritime character. This movement from the Gulf of Mexico helps to decrease the range between hot and cold temperatures and provides a source of abundant moisture and rainfall.

2.1.2 Temperature

Temperature records are available for Louisiana, published by the National Oceanic and Atmospheric Administration's (NOAA) National Climatic Data Center (NCDC). The records can be downloaded on the agency's website at http-//www1.ncdc.noaa.gov/pub/orders/IPS-E94999DE-130D-4437-8BB1-312509AAD725.pdf. The website has detailed records available for three locations in each parish within the study area. The monthly and annual mean normals over a 100 year timeframe, 1910-2010, are shown in Table 1. The average annual mean normal temperature is 66.6 degrees Fahrenheit (^{o}F), with monthly mean temperature normal varying from 39.5 ^{o}F in February to 86.1 ^{o}F in August. A maximum extreme temperature of 102 (^{o}F) was recorded at Morgan City during July of 1980 and a minimum extreme of 10 (^{o}F) was recorded during the month of December at Morgan City.

Table 1- Normal Temperatures (Source- NCDC)

Station	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Franklin - 3NW	51.9	55.1	61.3	67.2	74.6	79.7	81.4	81.1	77.4	68.6	60.3	54.1	67.7
Galliano	53.0	55.7	62.3	67.9	75.1	80.1	81.9	81.8	78.5	69.9	62.2	55.3	68.6
Morgan City	51.8	54.8	61.2	67.4	74.5	79.6	81.5	81.2	78.2	70.0	61.3	54.6	68.0
AVERAGE	52.2	55.2	61.6	67.5	74.7	79.8	81.6	81.4	78.0	69.5	61.3	54.7	68.1

2.1.3 Tides

The normal tidal range at Grand Isle, LA is diurnal and is from 1.5 to 2.0 feet from low to high tide. Further inland the extent of tidal range and area of influence are determined

by the rainfall discharge exiting the drainage areas into the Gulf of Mexico, and by discharges in the Gulf Intracoastal Waterway originating in the Atchafalaya River. The tidal ranges become smaller when moving further inland. During a spring tidal cycle these ranges may be larger; during neap tidal cycle these ranges may be less.

2.1.4   Precipitation

Precipitation records are also available for Louisiana and published on the National Oceanic and Atmospheric Administration's (NOAA) National Climatic Data Center website (http-//www1.ncdc.noaa.gov/pub/orders/IPS-E94999DE-130D-4437-8BB1-312509AAD725.pdf).

Table 2 lists the stations with their period of record and available extremes for the same 40 year period that was used for the temperature data. The stations have 30-year monthly and annual normals. The average annual normal rainfall for these stations is 65.36 inches.

Table 3 lists the monthly and annual normals. The wettest month is July with an average monthly normal of 7.55 inches. February is the driest month averaging 4.30 inches.

Table 2 - Precipitation Extremes

PRECIPITATION EXTREMES

Station	Period of Record (to 2010)	Maximum Monthly (in.)	Date	Minimum Monthly (in.)	Date	Greatest 1-Day	Date
Franklin 3NW	1955-Date	19.87	Dec-09	0	Nov-61	8.51	12/30/2010
Galliano	1968-Date	21.35	Sep-98	0.12	Oct-78	9.9	5/30/1975
Morgan City	1905-Date	23.04	Jun-01	0	Oct-11	15.55	4/16/1927

SOURCE: NCDC

Table 3 – Monthly and Annual Normal Precipitation   (Source- NCDC)

Station	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Franklin - 3NW	5.43	3.92	4.73	5.09	4.92	7.06	7.37	7.76	5.85	3.72	4.46	4.82	65.13
Galliano	5.85	4.59	5.53	4.43	5.72	5.82	7.69	7.13	6.34	3.65	4.67	4.03	65.48
Morgan City	5.81	4.39	4.7	4.22	5.38	5.81	7.6	7.4	6.49	3.66	5.07	4.95	65.48
AVERAGE	5.70	4.30	4.99	4.58	5.34	6.23	7.55	7.43	6.23	3.68	4.73	4.60	65.36

Wind data taken at New Orleans Louis Armstrong Airport are typically used to describe the study area. This wind data is used because the locations experience similar kinds

of events due to the close proximity of their geographic location on the Gulf Coast. Table 4 shows the average monthly and annual wind speeds over the period 1971-2000. The average velocity of the winds is 8.1 miles per hour over this period. Southeast winds predominate in the spring and summer. The prevailing winds of the fall and winter are from the northeast. Winter storms in the area have produced wind speeds of up to 47 miles per hour. The summer is often disturbed by tropical storms and hurricanes that produce the highest winds in the area. Maximum wind speeds observed (highest one-minute speed) since 1963 was 69 mph and was a result of Hurricane Betsy in September 1965.

Table 4 - Average Monthly and Annual Wind Speeds

NEW ORLEANS LOUIS ARMSTRONG AIRPORT 1971 - 2000												
JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)	(mph)
9.2	9.5	9.4	9.2	8.2	6.8	5.9	6.0	7.6	8.0	8.5	8.8	8.1

2.1.5   Stream Gaging Data

Stream gaging data was used from the 31 stations in the study area. Some stations are maintained through a cooperative agreement between the US Army Corps of Engineers (USACE) and the United States Geological Survey. The stations' period of record and available data types are shown in Table 5. The station locations are shown in Figure 2.
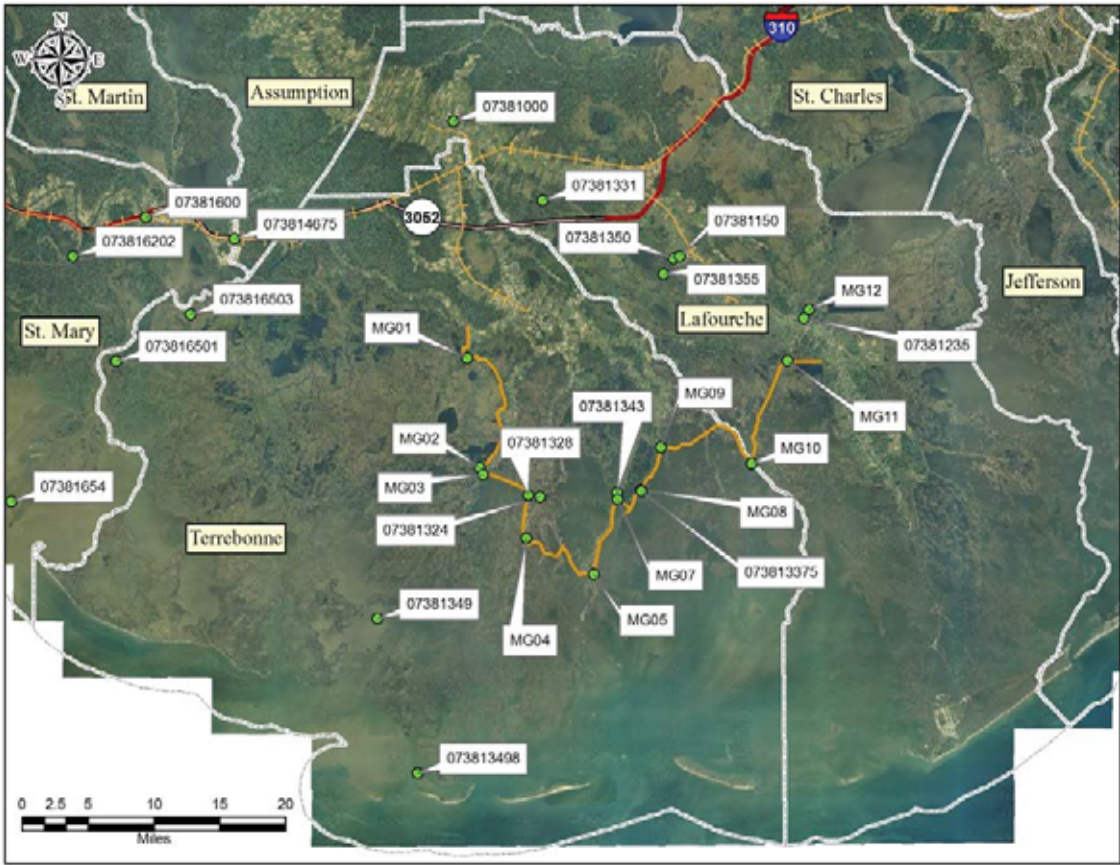
Figure 2 - Gage Station Locations

Table 5 - Gage Station Information

Station ID	Station Name	Period	Type	Conversion of Stage to Elevation (NAVD88 2004.65,ft)	Source
MG01	West Minor's Canal	07/2004 - 04/2005 * 02/2004 - 04/2006 05/2004 - 04/2006	Discharge Stage Salinity	-0.26	USACE MVN
MG02	Falgout Canal	04/2005 - 10/2005 09/2004 - 10/2005 *	Stage Salinity	-0.68	USACE MVN
MG03	Bayou Dularge	09/2004 - 04/2005 07/2003 - 10/2005 09/2004 - 04/2005	Velocity Stage Salinity	-0.68	USACE MVN
MG04	Bayou Grand Caillou	10/2004 - 11/2004 10/2004 - 11/2004	Stage Salinity	-0.54	USACE MVN
MG05	Bayou Petit Caillou	03/2004 - 06/2005 04/2004 - 06/2005	Stage Salinity	-0.4	USACE MVN
MG07	Bush Canal	11/2004 - 09/2005	Stage	-0.16	USACE

Station ID	Station Name	Period	Type	Conversion of Stage to Elevation (NAVD88 2004.65,ft)	Source
					MVN
MG08	Bayou Terrebonne	10/2003 - 05/2005 04/2004 - 06/2005 *	Stage Salinity	-0.16	USACE MVN
MG09	Humble Canal	05/2003 - 10/2005 04/2004 - 10/2005	Stage Salinity	-0.2	USACE MVN
MG10	Bayou Pointe-aux-Chenes	10/2003 - 02/2005 * 04/2004 - 03/2005	Stage Salinity	0	USACE MVN
MG11	Grand Bayou Canal	05/2004 - 10/2005 05/2004 - 10/2005	Stage Salinity	0	USACE MVN
MG12	GIWW Larose	12/2004 - 04/2005 06/2004 -05/2005 06/2004 - 05/2005	Discharge Stage Salinity	0	USACE MVN
-	Houma	1971 to 2008	Precipitation	-	NCDC NOAA
-	Morgan City	1996 to 2006	Precipitation	-	NCDC NOAA
7381000	Bayou Lafourche at Thibodaux, LA	01/2004 - 10/2009	Discharge/Stage	0	USGS
7381150	Bayou Lafourche at Lockport, LA	10/2006 - 10/2009	Stage	-3.793	USGS
7381235	GIWW West of Bayou Lafourche at Larose, LA	01/2004 - 10/2009	Discharge/Stage	0	USGS
7381324	Bayou Grand Caillou at Dulac, LA	01/2004 - 10/2009	Discharge/Stage	0.357	USGS
7381328	Houma Navigation Canal at Dulac, LA	01/2004 - 10/2009	Discharge/Stage	0.02	USGS
7381331	GIWW at Houma, LA	01/2004 - 10/2009	Discharge/Stage	-0.712[1]	USGS
73813375	Bayou Terrebonne at Ctrl Str near Lapeyrouse, LA	12/2001 - 11/2005	Precipitation/Stage	0	USGS
7381343	B. Petit Caillou at Ctrl Str near Lapeyrouse, LA	01/2004 - 10/2009	Precipitation/Stage	0	USGS
7381349	Caillou Lake (Sister Lake) SW of Dulac, LA	08/2008 - 10/2009	Precipitation/Stage	0	USGS
73813498	Caillou Bay SW of Cocodrie, LA	01/2004 - 09/2006	Precipitation/Stage	N.C.F.[2]	USGS
7381350	Company Canal at Hwy 1 at Lockport, LA	01/2004 - 10/2009	Stage	-0.566[1]	USGS

Station ID	Station Name	Period	Type	Conversion of Stage to Elevation (NAVD88 2004.65,ft)	Source
7381355	Company Canal at Salt Barrier near Lockport, LA	01/2004 - 10/2009	Stage	-1.090[1]	USGS
7381600	Lower Atchafalaya River at Morgan City, LA	01/2004 - 10/2009	Discharge/Stage	-0.45	USGS
73816501	Avoca Island Cutoff south of Morgan City, La.	10/2008 - 10/2009	Stage	N.C.F.[2]	USGS
73816503	Bayou Penchant S of Morgan City, LA	01/2004 - 10/2009	Discharge/Stage	0	USGS
7381654	Atchafalaya Bay at Eugene Island	08/2006 - 10/2009	Stage	N.C.F.[2]	USGS
73816202	GIWW at mile 103 S of Morgan City, LA	01/2004 - 10/2009	Stage	0	USGS
73814675	Bayou Boeuf at Railroad Bridge at Amelia, LA	01/2004 - 03/2009	Discharge/Stage	0	USGS

* Three or more consecutive months missing from data.
** Data is sporadic.
Notes: 1. Datum based on OPUS measurements and it is not tied to any tidal epoch.
        2. "N.C.F." in the above table corresponds to "No Conversion Factor" for converting the stage to an elevation.

2.1.6   Floods of Record

2.1.6.1   Non-Tropical Events

The study area floods from tidal surges associated with hurricanes and tropical storms. Lower Atchafalaya River water enters the study area from the Avoca Island Cutoff Channel and GIWW. Heavy rainfall also affects the lower reach in the areas that are highly developed.

Some of the major historical floods caused by heavy rainfall or tides occurred in 1973, 1980, 1983, 1991, 2005 and 2011. Descriptions of significant events are described below.

1973 Flood- Flooding occurred throughout the eastern portion of the study area during the spring of 1973. Tidal flooding inundated the area below Highway 90 with the exception of the alluvial ridges of the Mississippi River, Bayou Lafourche, and many of the smaller streams that drain into the Gulf of Mexico. Peak stages recorded on May 27 include 11.16 feet NGVD at Wax Lake Outlet at Calumet and 6.27 feet NGVD at Lower

Atchafalaya River below Sweet Bay Lake. On May 28, flooding caused a high stage of 10.53 feet NGVD on the Lower Atchafalaya River at Morgan City.

1980 Flood- Heavy rains at the end of March and early April setup flooding which occurred over the entire study area during mid April. A maximum extreme was set at Bayou Black at Greenwood, 4.82 feet NGVD. One day rainfall totals on April 13 exceeded 9 inches at Morgan City (9.1) and 11 inches at Thibodaux (11.8)**.**

1983 Flood- Heavy rains north of the study area produced this flood of record. In the Atchafalaya Basin Floodway System, peak stages from this event include 8.11 ft NGVD at Wax Lake Outlet at Calumet and 7.32 ft NGVD at Lower Atchafalaya River at Morgan City on June 6.

1991 Flood- Flooding occurred throughout the study area due to above normal rainfall during most of year. In the Houma Thibodaux Area during May 8-10, the three day totals for the two sites were 12.94 and 14.33 inches NGVD, respectively. The rainfall event set a maximum extreme event of 8.76 feet NGVD on Bayou Lafourche at Thibodaux gage on May 9. In addition to heavy rainfall, high tides in the Gulf of Mexico affected runoff.

2.1.6.2 Tropical Events

Some of the major historical hurricanes that produced significant flooding within the study area are listed below in Table 6.

Table 6 - Historical Hurricanes in Study Area

Year	Name	Dates	Category at Landfall
1909	Unnamed storm	September 18-21	Category 3
1915	Unnamed storm	September 25-October 1	Category 4
1956	Flossy	September 23-30	Category 1
1957	Audrey	June 26-27	Category 4
1961	Carla	September 10-12	Category 5
1964	Hilda	October 3-4	Category 4
1965	Betsy	September 9-10	Category 4
1971	Edith	September 16	Category 1
1974	Carmen	September 8	Category 4
1985	Danny	August 15	Category 1
1985	Juan	October 28-30	Category 1
1992	Andrew	August 24-27	Category 5
2002	Lili	October 3	Category 1
2005	Katrina	August 23-30	Category 5

Year	Name	Dates	Category at Landfall
2005	Rita	September 23-24	Category 5
2008	Gustav	September 2	Category 2
2008	Ike	September 12-13	Category 2

1909 Flood- Wind speeds of 80 mph were reported for Thibodaux and near the mouth of Bayou Terrebonne, 40 miles south of Thibodaux as a tropical cyclone passed through the study area from September 19-20. The highest tides were experienced at the mouth of Bayou Terrebonne in Lafourche Parish, where an elevation of 15 feet above sea level was attained at Sea Breeze.

1915 Flood- Heavy rainfall, high winds, and extremely low barometric pressures from this hurricane from September 29 to October 2 caused headwater flooding along Bayou Lafourche where stages of 9 and 5 feet above sea level, respectively, were reported at Leeville and Golden Meadow. The United States Weather Bureau 5 minute sustained and extreme wind velocities for the 29th of September were 66 and 75 miles per hour at New Orleans. In Leeville, approximately 13 miles west of Grand Isle, only 1 of 100 houses remained standing as a result of this storm.

1956 Flood- Hurricane Flossy, during the period of September 21-30, was the cause of this flood. Tides reached 5 to 8 feet above normal along most of the southeastern coast. Rainfall during the storm was quite heavy. The heaviest occurred at Golden Meadow where 16.7 inches of rain was recorded.

1957 Flood-Heavy rainfall and Heavy winds associated with Hurricane Audrey, June 25-28, caused headwater flooding along the Louisiana coast. The storm set peak stages of 8.05 feet NGVD at Lower Atchafalaya River below Sweet Bay Lake, 6.81 feet NGVD at Atchafalaya Bay at Eugene Island, 8.52 feet and 7.35 feet NGVD respectively at Intracoastal Waterway at Wax Lake East and West on June 27. Maximum stages were also set along the coastline on this date and included 6.00 feet NGVD at Schooner Bayou and 8.12 feet NGVD at Leland Bowman Lock.

1961 Flood- Hurricane Carla raised tides 3 to 4 feet above normal along the entire coastline of Louisiana during the period of September 4-14. Rainfall associated with the hurricane amounted to 6.2 inches at Morgan City and 3.4 inches at Houma.

1964 Flood- Hurricane Hilda, during the period of October 3-5, caused extensive tidal and headwater flooding in the study area. Heavy rainfall north of the study area associated with hurricane ranged from 10.1 inches at New Roads to 8.9 inches at Baton Rouge.

1965 Flood- Hurricane Betsy damaged most of southeast Louisiana, specifically Lafourche and Terrebonne Parish. In Thibodaux, winds of 130 mph to 140 mph were reported.

1971 Flood- Hurricane Edith damaged made landfall in the western parishes of Louisiana as a Category 2. The storm impacted St. Mary Parish as a Category 1.

1974 Flood- Hurricane Carmen was responsible for this flood during September 5-9. The highest known storm tide, 11.64 feet NGVD occurred at Cocodrie in Terrebonne Parish. This stage was reportedly more than 10 feet above normal.

1985 Flood- Hurricane Danny, Category I, hurricane made landfall between Grand Chenier in Cameron Parish and Pecan Island in Vermillion. Storm surge values ranged from 5 to 8 feet across Iberia, St. Mary, and Vermillion parishes.

Hurricane Juan, during the period of October 28-30, caused massive flooding throughout the study area due to its prolonged 5-day stay along the Louisiana Coast. Tides were generally 3 to 6 feet above normal, and storm surges of 5 to 8 feet were reported in several coastal parishes. Rainfall amounts in the study area ranged from 5 to nearly 17 inches for this period.

1992 Flood- Hurricane Andrew, during the period August 24-27, set a new maximum extreme of 7.65 feet NGVD at Round Bayou at Deer Island and recorded 6.8 ft NGVD for the Lower Atchafalaya River at Morgan City in St. Mary parish. The Gulf Intracoastal Waterway at Wax Lake East Control Structure recorded a stage of 6.15 feet NGVD.

2002 Flood- Hurricane Lili major flooding event for Terrebonne Parish had over 1,000 structures flooded. Lili made landfall on the morning of October 3 near Intracoastal City, as a weakening category one hurricane. Wind gusts reaching 120 mph, coupled with over 6 inches of rainfall and a storm surge of 12 feet caused over $790 million in damage to Louisiana. A total of 237,000 people lost power, and oil rigs offshore were shut down for up to a week.

2005 Flood- The year, 2005, will be recorded in the annals of history as the "Year of the Storms." Never in history had Terrebonne Parish been declared a disaster area twice within a period of thirty days. But, on August 29, 2005, for Hurricane Katrina and again on September 24, 2005, for Hurricane Rita, the Parish was declared a federal disaster area by the President of the United States. Maximum stage experienced in the area ranged from about 2.8 to 3.2 feet for Katrina. Katrina pushed the water out to the Gulf in Terrebonne Parish. During the period of September 20-26, Rita flooded over 4,000 structures in Terrebonne Parish. Maximum stage experienced in the area for Rita ranged from about 3.6 to 8.6 feet.

Lafourche Parish was affected by three storms in 2005, including Hurricane Cindy in July, Hurricane Katrina in August and Hurricane Rita in September. By far the most dangerous of the three, Hurricane Katrina carried maximum sustained winds of 140 mph, with hurricane-force winds extending over 100 miles from the center of the storm. Storm surges reached 20 to 30 feet above normal tide levels and brought large

battering waves. While these conditions did not occur at the location of the Morganza project, they represent the potential impacts a storm could impose on the project as the storm extended from New Orleans to the east along the Mississippi Gulf coast.

St. Mary Parish was one of the fortunate parishes that was not directly hit by Hurricane Katrina, which struck to the east, of Hurricane Rita, which passed west of the parish. Destructive storm surges spared St. Mary during Hurricane Katrina but hit the parish coastline as well as some inland areas through man-made canals during Hurricane Rita. The violent storm surged up to 15 feet as it moved towards the southern half of the parish, causing flooding in a localized area of the City of Franklin and in a widespread area of the parish south of U.S. Highway 90. Hurricane force winds of at least 75 mph from both Katrina and Rita also swept through St. Mary, causing wind damages.

2008 Flood- During the final days of August 2008, Hurricane Gustav entered the Gulf of Mexico. With an uncertain path and dire predictions, a state of emergency was declared on August 28 that included a mandatory evacuation of Terrebonne Parish. The parish was evacuated over a four day period. It was the largest and most successful evacuation in the region's history. Terrebonne Parish suffered a direct hit from the Category 2 hurricane at 10-30 a.m. on September 1st. Gustav was the largest hurricane to hit the parish since Hurricane Betsy in 1965. Maximum stage experienced in the area ranged from about 1.5 to 2.5 feet.

A little over a week after Hurricane Gustav made a direct hit, Hurricane Ike flooded the southern most areas of Terrebonne Parish. Hurricane Ike inundated 2,800 structures. High water marks were measured at 7.5 feet NAVD 29 near Highway 56 and 8.1 feet NAVD 29 just east of Highway 55. Terrebonne Parish constructed several non-federal levees that have performed well in prior storm events. However, the water from Hurricane Ike breached the Montegut Levee and overtopped the following levees- Madison Canal, 4-3B Pointe-aux-Chenes, 3-1A Susie Canal, 4-3C Isle De Jean Charles, 3-1B, 4-8 Montegut, 3-1B Extension Orange Street South, 5-1A Lower Little Caillou, 3-1C Shrimpers Row, 5-1B Upper Little Caillou, 3-2 Mayfield, 5-2 Boudreaux Canal, 4-1 Upper Pointe-aux-Chenes, 8-1 Lower Dularge, 4-2B Sara Road to Bush Canal, 8-2C Marmande North, 4-3A Middle Pointe-aux-Chenes, and 8-2D Falgout Canal North.

St. Mary parish was impacted by both Gustav and Ike. Surge values varied from 8-12 for Gustav and 4-5 feet for Ike. Lafourche Parish was hit with 100 mph winds and flooding was extensive south of Golden Meadow during Gustav.

2011 Flood- By noon on September 3, 2011, tropical storm Lee's center appeared to be headed ashore near Intracoastal City, about 100 miles west of Houma. Sustained winds were recorded at 60 mph, with gusts as high as 75 mph. The area was currently under a moderate drought until Lee dropped as much as 14 inches of rain in some areas. There were unconfirmed reports of flooded homes in the extreme low portions of Terrebonne parish, including Cocodrie, Isle de Jean Charles and Pointe-aux-Chenes.

Tidal surge peaked at about 6.5 feet above sea level at the Bayou Terrebonne floodgate in Montegut. This was the highest water level recorded in the parish. A normal tide is about 1.5 feet above sea level so the storm produced as much as 5 feet of increased tide into some communities.

2.1.7   Sea Level Change

2.1.7.1   Background

The Army Corps of Engineers has developed guidelines on how future sea level rise is to be incorporated into project engineering efforts. The guidance, detailed in Engineering Circular (EC) 1165-2-212, *Sea-Level Change Considerations for Civil Works Programs*, follows findings from the National Research Council's 1987 report.

Sea level change can cause a number of impacts in coastal and estuarine zones, including changes in shoreline erosion, inundation or exposure of low-lying coastal areas, changes in storm and flood damages, shifts in extent and distribution of wetlands and other coastal habitats, changes to groundwater levels, and alterations to salinity intrusion into estuaries and groundwater systems.

EC 1165-2-212

EC 1165-2-212 discusses three potential future sea level scenarios based on historic, intermediate, or high sea level rise rates. Relative sea level rise is a combination of eustatic sea level rise (i.e., global sea level rise due to polar ice cap melting) and subsidence (i.e., sinking of land). It is well known that relative sea level rise can vary considerably with eustatic sea level rise in southern Louisiana. The relative sea level rise scenarios for the Morganza project are based on both EC 1165-2-212 and EC 1165-2-211 (which was issued in July 2009 and replaced by EC 1165-2-212 in October 2011).

2.1.7.1.1 Historical Gage Data

The lowest rate is calculated through use of historical stage gage data. In case of our analysis, the gage data was based on the Leeville, LA gage (Figure 3). Gage data was available for a total of 43 years starting in 1957 and ending in 1999. A Leeville gage data scatter plot is given in Figure 4 on a monthly time scale. The width of scatter at any month is roughly 1.5 feet with a few outliers spread throughout the plot. Applying a linear trend to the data gives a yearly sea level rate of increase of approximately 7 millimeters per year. This is a relative sea level rise rate since subsidence is included in the gage readings.

Figure 3 - Location of Leeville Stage Gage in reference to the MTG alignment
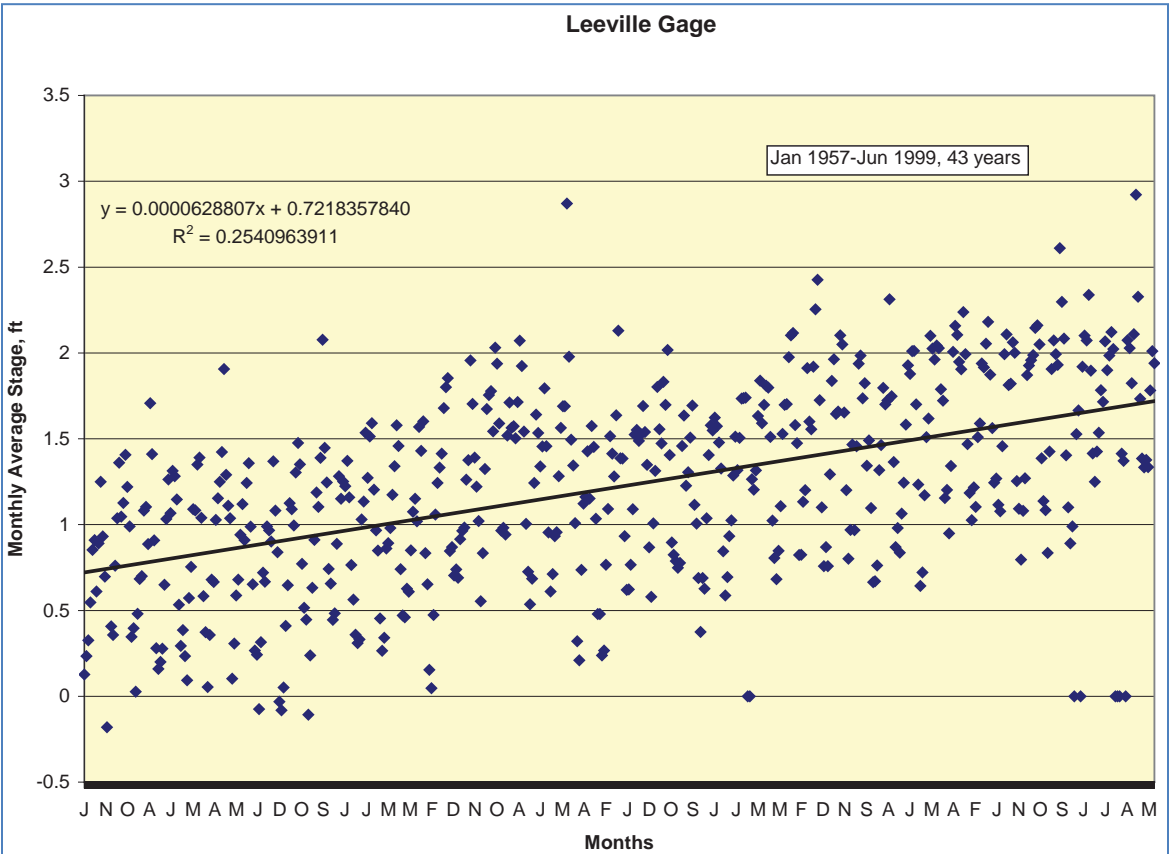
Figure 4 - Monthly Average Leeville gage data along with linear trendline

2.1.7.1.2 Estimating Future Change in Local Mean Sea Level

The EC discusses incorporation of accelerated global sea level rise scenarios. The two accelerated rates are given by NRC Curve I and NRC Curve III. A single equation is given in the EC to compute each of the projected rates of sea level rise. The equation units are meters.

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2),$$

where E corresponds to eustatic sea level rise, t_1 corresponds to the time between the project's construction date and year 1986, t_2 is the time between a future date at which one wants to estimate sea level rise and year 1986. The variable *b* is a coefficient dependent on the NRC Curve being followed. For Curve I and Curve III the values of *b* are 2.36E-5 and 1.005E-4, respectively. The value of 0.0017 in the equation corresponds to the historic global mean sea level change rate of 1.7 millimeters per year. Using the historic relative sea level rise of approximately 7 mm/year along with the equation given above, Figure 5 through Figure 7 give expected sea level rise values from year 2010 to 2085. The base year for completion of the MTG levee system is 2035. The future condition is year 2085.

Leeville gage 43 year period of record

model year 2010		Historic trend	Historic trend	Subsidence rate Leeville historic rate mm/yr historic - 1.7 =	Subsidence rate	0.5m @ 2100 NRC curve I acceleration factor: 0.00	1.5m @ 2100 NRC curve III acceleration factor: 0.00
		mm/yr 6.995		5.295		new eustatic + subsidenc	
	year	mm	feet	mm	feet	feet	feet
MODEL	2010	0	0.00	0	0.00	0.00	0.00
	2011	6.995	0.02	5.295	0.02	0.03	0.04
	2012	13.99	0.05	10.59	0.03	0.05	0.08
	2013	20.985	0.07	15.885	0.05	0.08	0.12
	2014	27.98	0.09	21.18	0.07	0.11	0.16
	2015	34.975	0.11	26.475	0.09	0.14	0.20
	2016	41.97	0.14	31.77	0.10	0.16	0.24
	2017	48.965	0.16	37.065	0.12	0.19	0.29
	2018	55.96	0.18	42.36	0.14	0.22	0.33
	2019	62.955	0.21	47.655	0.16	0.25	0.38
	2020	69.95	0.23	52.95	0.17	0.27	0.42
	2021	76.945	0.25	58.245	0.19	0.30	0.47
	2022	83.94	0.28	63.54	0.21	0.33	0.51
	2023	90.935	0.30	68.835	0.23	0.36	0.56
	2024	97.93	0.32	74.13	0.24	0.39	0.61
	2025	104.925	0.34	79.425	0.26	0.42	0.66
	2026	111.92	0.37	84.72	0.28	0.45	0.70
	2027	118.915	0.39	90.015	0.30	0.48	0.75
	2028	125.91	0.41	95.31	0.31	0.51	0.80
	2029	132.905	0.44	100.605	0.33	0.53	0.86
	2030	139.9	0.46	105.9	0.35	0.56	0.91
	2031	146.895	0.48	111.195	0.36	0.59	0.96
	2032	153.89	0.50	116.49	0.38	0.62	1.01

Figure 5- Part (1) of sea level rise computations by year. Historic (yellow), Intermediate (teal), and High (blue) rates are computed

	2033	160.885	0.53	121.785	0.40	0.65	1.07
	2034	167.88	0.55	127.08	0.42	0.68	1.12
BASE	2035	174.875	0.57	132.375	0.43	0.72	1.18
	2036	181.87	0.60	137.67	0.45	0.75	1.23
	2037	188.865	0.62	142.965	0.47	0.78	1.29
	2038	195.86	0.64	148.26	0.49	0.81	1.34
	2039	202.855	0.67	153.555	0.50	0.84	1.40
	2040	209.85	0.69	158.85	0.52	0.87	1.46
	2041	216.845	0.71	164.145	0.54	0.90	1.52
	2042	223.84	0.73	169.44	0.56	0.93	1.58
	2043	230.835	0.76	174.735	0.57	0.96	1.64
	2044	237.83	0.78	180.03	0.59	1.00	1.70
	2045	244.825	0.80	185.325	0.61	1.03	1.76
	2046	251.82	0.83	190.62	0.63	1.06	1.82
	2047	258.815	0.85	195.915	0.64	1.09	1.89
	2048	265.81	0.87	201.21	0.66	1.13	1.95
	2049	272.805	0.90	206.505	0.68	1.16	2.01
	2050	279.8	0.92	211.8	0.69	1.19	2.08
	2051	286.795	0.94	217.095	0.71	1.22	2.14
	2053	293.79	0.96	222.39	0.73	1.27	2.26
	2054	300.785	0.99	227.685	0.75	1.31	2.33
	2055	307.78	1.01	232.98	0.76	1.34	2.40
	2056	314.775	1.03	238.275	0.78	1.37	2.46
	2057	321.77	1.06	243.57	0.80	1.41	2.53
	2058	328.765	1.08	248.865	0.82	1.44	2.60
	2059	335.76	1.10	254.16	0.83	1.48	2.67
MODEL	2060	342.755	1.12	259.455	0.85	1.51	2.75
	2061	349.75	1.15	264.75	0.87	1.54	2.82
	2062	356.745	1.17	270.045	0.89	1.58	2.89
	2063	363.74	1.19	275.34	0.90	1.61	2.96
	2064	370.735	1.22	280.635	0.92	1.65	3.04
	2065	377.73	1.24	285.93	0.94	1.68	3.11
	2066	384.725	1.26	291.225	0.96	1.72	3.19
	2067	391.72	1.29	296.52	0.97	1.75	3.26
	2068	398.715	1.31	301.815	0.99	1.79	3.34
	2069	405.71	1.33	307.11	1.01	1.83	3.42
	2070	412.705	1.35	312.405	1.02	1.86	3.50
	2071	419.7	1.38	317.7	1.04	1.90	3.57

Figure 6 - Part (2) of sea level rise computations by year. Historic (yellow), Intermediate (teal), and High (blue) rates are computed

	2072	426.695	1.40	322.995	1.06	1.93	3.65
	2073	433.69	1.42	328.29	1.08	1.97	3.73
	2074	440.685	1.45	333.585	1.09	2.01	3.81
	2075	447.68	1.47	338.88	1.11	2.04	3.90
	2076	454.675	1.49	344.175	1.13	2.08	3.98
	2077	461.67	1.51	349.47	1.15	2.12	4.06
	2078	468.665	1.54	354.765	1.16	2.15	4.14
	2079	475.66	1.56	360.06	1.18	2.19	4.23
	2080	482.655	1.58	365.355	1.20	2.23	4.31
	2081	489.65	1.61	370.65	1.22	2.27	4.40
	2082	496.645	1.63	375.945	1.23	2.30	4.48
	2083	503.64	1.65	381.24	1.25	2.34	4.57
	2084	510.635	1.68	386.535	1.27	2.38	4.66
FUTURE	2085	517.63	1.70	391.83	1.29	2.42	4.75

Figure 7 - Part (3) of sea level rise computations by year. Historic (yellow), Intermediate (teal), and High (blue) rates are computed

From the above information, the base and future condition associated sea level rise values are 0.72 feet and 2.42 feet, respectively, for the intermediate rate of rise and 1.18 feet and 4.75 feet, respectively, for the high rate of rise.

2.1.7.2 Project Model Rates

Before incorporating the findings of EC-1165-2-211 and EC-1165-2-212 into the project, a few sea level rise scenarios had already been modeled according to the work of Kevin Knuuti used in Louisiana Coastal Protection and Restoration (LACPR). Knuuti's work is based on Intergovernmental Panel on Climate Change Fourth Assessment Report published in 2007. Table 7 provides modeled rates of sea level rise

in the last column which were done prior to the EC publication. The high rate of sea level rise (3.2 ft) was for a condition 50 years from year 2010 which would have been year 2060.

Table 8 summarizes all modeled rates as well as the latest rates established through the EC. Note inclusion of the old future condition of 2060 along with the updated new base and future condition. Rate 1 and rate 2 correspond to intermediate and high rates, respectively.

Table 7 - Breakdown of eustatic and subsidence rates used to compute historic and high rates of sea level rise modeled

	Eustatic Sea Level Rise - ft per 50 years	Source	Subsidence - ft per 50 years	Source	Total - ft per 50 years
MTG Scenario 1 (historic)	0.66	ERDC Knuuti	0.5	MVN Geology for project area	1.15
MTG Scenario 2 (high rate)	2.0	ERDC Knuuti	1.2	ERDC Knuuti	3.2

Table 8 - Summary of all sea level rise rates established for the project.

Morganza to Gulf SLR Rates (ft) Relative to Year 2010 (existing)						
				Modeled		
Year	Historic	Rate 1	Rate 2	Low	High 1	High 2
2035	0.57	0.72	1.18			
2060	1.12	1.51	2.75		1.15	3.2
2085	1.70	2.42	4.75			5.0

Note: The old future condition year (2060) is also included.

2.2 WATERSHED CHARACTERISTICS

2.2.1 Basin Delineation

The project area is located south of Houma, LA near the southern coast of Louisiana. The study area includes portions of Terrebonne and Lafourche Parishes and consists primarily of waterways, lakes, and marsh areas. The primary inflows to the system are the Atchafalaya River and the Wax Lake Outlet with lesser inflows from Bayou Lafourche, the Gulf Intracoastal Waterway (GIWW), Bayou Boeuf, and additional drainage channels. The tidal signal for the area is from the Gulf of Mexico, which makes it a diurnal, micro-tidal system (~ 2 ft spring tide range).

The total drainage area within the study area is about 1891 square miles (mi^2). The

area east of Louisiana State Highway 311 drains predominately from north to south, while the area west of the highway drains predominately from east to west. There are several watercourses within the study area. Many of the watercourses are interconnected and have extremely low grade, resulting in a complex drainage system. The major watercourses in the study area include the Gulf Intercoastal Waterway (GIWW), Bayou Terrebonne, Bayou Lafourche, Bayou Black, Bayou Petit Caillou, Houma Navigation Canal (HNC), Bayou Grand Caillou, and Bayou Dularge. The overland elevation within the study area ranges from 0 to 14 ft North American Vertical Datum (NAVD) 1988 (epoch 2004.65). Specific subbasin information can be found in Section 2.6.2.5.2 of this report.

2.3   ADCIRC AND STWAVE MODELING

2.3.1   Background

Numerical models were used to simulate surge and wave response in the project area. This section will give a brief summary of the hydrodynamic models used for hurricane simulation and how the modeling effort relates to the project. Statistical probabilities will be discussed in the modeling section as well as several other sections of the report. Table 9 provides main Annual Exceedance Probabilities (AEP) discussed in this report as well as each corresponding return period.

Table 9 - Relation of common frequencies with return periods

	Frequency with Corresponding Return Period									
Frequency (%)	50	20	10	4	2.86	2	1.3	1	0.5	0.2
Return (yrs)	2	5	10	25	35	50	75	100	200	500

Hydrodynamic models were needed to define statistical probabilities for specified locations over the project area. Storm surge and wave statistics are normally developed through use of 304 synthetic hurricane storms where each storm is defined by a track and parameters (forward speed, size, orientation to coast, etc.). The need for such a large number of synthetic storms is in part due to a lack of historical hurricanes hitting the gulf coast region. A total of 304 storms are used to fully cover the range of statistical probabilities needed. In some cases a subset of the 304 storms can be used to define statistical probabilities for a project if the storms not used are determined to be of no statistical significance. In other words, if a given statistical surge probability value established for a given point in the project area is equal to the same value whether running a subset of storms or the full set of storms then storms outside of the subset are defined as statistically insignificant. Subsets are mainly selected through comparing storm tracks with project location. Storm tracks far away from the project area are less likely to have a significant effect on the model results. In regards to the MTG project, analyses indicate that only 115 storms of the 304 would be statistically significant to the project area. Only 115 storms were simulated for the project. The subset of storms was created by experts at the Army Corps Engineer Research and Development Center (ERDC).

Various models were used to simulate ocean circulation, offshore waves, near-shore waves, and inland surge. Ocean circulation was modeled with the planetary boundary layer model (PBL). The model generates a time series of wind and pressure fields for each specific storm and reads directly to ADCIRC. The surge model, ADCIRC, develops offshore surge based on these wind and pressure fields. The surge model ADCIRC is a two dimensional finite-element model that runs on a large unstructured grid domain (Luettich et al. 1992, Westerink et al. 1993, Luettich and Westerink 2004). The grid covers the entire Gulf of Mexico as well as much of the Atlantic Ocean. The advantage of an unstructured grid is that it allows for higher resolution at user defined locations instead of having to increase resolution throughout the entire grid. The grid resolution ranged from meters around the project area to tens of kilometers in the Atlantic Ocean. Running in parallel with ADCIRC is the offshore wave model (WAM). The wave model computes directional wave spectra that will be further used by the near shore wave model STWAVE. The near shore wave model computes wave fields and radiation stresses that are read back into ADCIRC. The wave model STWAVE is a steady state model able to compute wave heights and periods based on, among other forces, a given surge field. There are 5 rectangular grids associated with STWAVE where all grids combined cover the entire Louisiana Gulf of Mexico coastline. Sensitivity analysis indicated STWAVE half-plane mode was sufficient to solve for the wave components. Both ADCIRC and STWAVE were run in a "coupled" fashion to solve for still water elevation (SWE) and wave characteristics.

2.3.2   ERDC Studies

ERDC was tasked with performing all modeling efforts for the project. Each of the project alternatives were modeled on the 2007 ADCIRC mesh sl15v6f containing approximately 2.2 million nodes. Modifications were made to this main mesh depending on whether with or without project conditions were being modeled. For without project conditions, no project levees were included in the model except for high ground that already existed in the area. The main sl15v6f mesh already included raised features of significance. The Larose to Golden Meadow levee was not considered in place for MTG without project conditions, however existing high ground along the Larose to Golden Meadow alignment was in the mesh since it already existed (Figure 8). For with-project conditions, the MTG authorized alignment was applied to the grid. Full levee boundaries associated with the Larose to Golden Meadow project were also included. Both the height of MTG and Larose levees were set to non-overtop (i.e., 20 meters high) (Figure 9). Another with-project alignment termed the multiple lines of defense alignment was modeled but was not considered a viable alternative after a short review. All raised features in the without project mesh were also included in the with-project mesh. Both mesh figures shown below have been filtered to a smaller set of points so as to have increased clarity of elevation values. In regards to STWAVE, all five rectangular grids were used (Cialone et al., 2010). The ADCIRC model does contain the Barrier Alignment, but is modeled as non-overtopping.

For all 115 synthetic storms the resulting surge and wave simulation results are available at every node on the grid. A small set of points from the entire nodal mesh were used to represent the model results. The subset of points surrounded the project area. A total of 320 points defined as the MTG points were specified by the USACE New Orleans District (MVN). A map of the points is shown in Figure 10. Statistical frequencies ranging from 0.2% to 2.0% chance exceedance were developed at each of these MTG points. Each of the 115 storms produced a maximum SWE, significant wave height (Hs), and mean wave period (Tm). The statistical analysis used each of the maximum values at a given location to produce event frequencies. Given some points will be "submerged" more times than others over the suite of storms a threshold was set for the number of times (i.e., storms) a point must be submerged in order to calculate a frequency at that location. The methodology used to develop the probability is termed joint probability method with optimum sampling. Refer to Resio (2007) for a more in-depth look at the methodology used to develop statistical probabilities of synthetic storms.
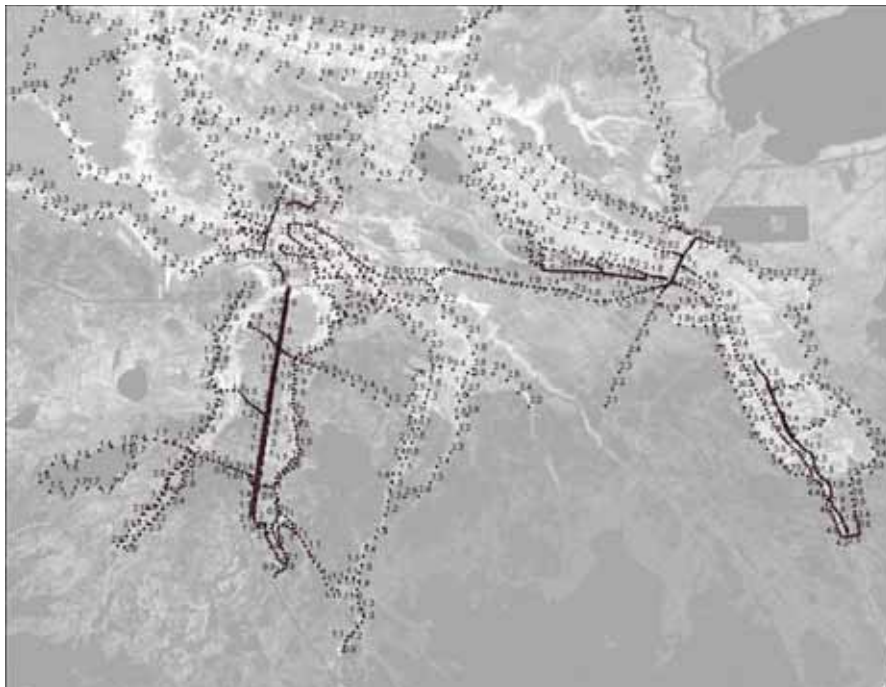
Figure 8 - Without Project raised features in mesh around project area. Elevations are in meters

Figure 9 - With Project non-overtop levee alignment. Without project raised features are included in the with-project mesh. Elevations are in meters
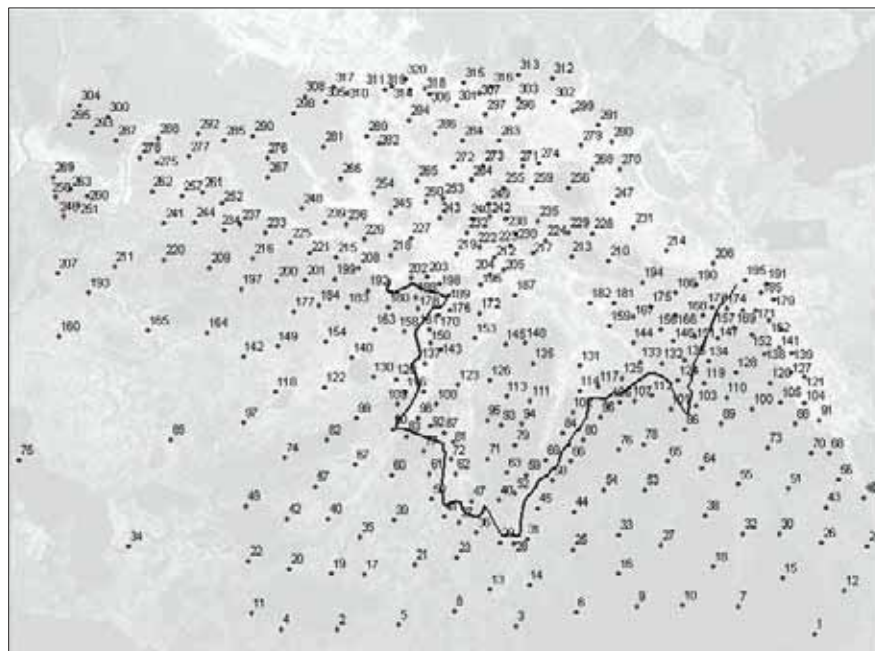
Figure 10 - Map of 320 Morganza nodal points used in the design effort along with the approximate alignment of the MTG Alignment

As of spring of 2009, ERDC had completed all modeling alternatives under the original base and future year conditions. The alternatives modeled along with corresponding

sea level rise (SLR) values are as follows-

Without Project
Base 2010 - No SLR (full suite 115 storms)
Future 2060 - 1.15 feet SLR (full suite 115 storms)
Future 2060 - 3.2 feet SLR (sensitivity analysis 12 storms)

With-Project - Authorized Alignment
Future 2060 - 1.15 feet SLR (full suite 115 storms)
Future 2060 - 3.2 feet SLR (sensitivity analysis 12 storms)

The 1.15 and 3.2 feet SLR values are intermediate and high approximations for year 2060. When the word "rate" is mentioned in this report it refers to the summation of rise from the current year to the given year. For example, 1.15 feet of rise would be the total rise from year 2010 to year 2060. The 3.2 feet SLR data are a result of a sensitivity analysis done ERDC. The analysis was completed to avoid having to run all 115 storms. Only the above mentioned sea level rise conditions were modeled since 2010 and 2060 were at the time considered base and future conditions, respectively. Also note the future condition rate of 3.2 ft will not be found in Figure 4. Discussion in section 2.1.8.2 notes the value was established through the work of Kevin Knuuti. This was completed before the SLR circular was published.

In July of 2009 SLR Engineering Circular 1165-2-211 was released to the New Orleans District. The SLR circular called for three rates of SLR to be incorporated into all future condition planning formulation and engineering designs. The three rates of SLR were defined as a historical rate, an intermediate rate, and a high rate. The intermediate and high rates of SLR were computed through equations defined in the circular. Since release of the circular and modification of base and future years to 2035 and 2085, three new rates of SLR have been established. The historic rates of SLR were established through a 43 year period of record from the Leesville gage in the MTG project area. All rates include eustatic SLR as well as localized subsidence.
Table 8 provides the new base and future conditions, the SLR rates established under the new conditions, as well as the SLR rates modeled under the old future condition year.

With the modeled SLR rates shown in the table above all historic, intermediate, and high SLR data could be interpolated. Any data corresponding to a SLR rate below 1.15 feet would use existing condition as well as 1.15 feet data to interpolate. The 4.75 feet SLR condition could not be interpolated since the 3.2 feet SLR condition was the highest scenario run. To establish the 4.75 feet statistical results ERDC ran one more set of 115 storms for with and without project conditions at a SLR of 5 feet. The model was run with a slightly higher SLR to account for any possible changes to project SLR rates in the future. This completed the modeling simulations for the project. ERDC's final report contains a more detailed description of the entire modeling efforts and can be furnished upon request. Also the future condition sensitivity analysis report used to

define the 3.2 feet SLR results can be furnished upon request.

2.4   RISK AND RELIABILITY ANALYSIS

The MTG Hurricane and Storm Damage Risk Reduction project is the first coastal project in Louisiana to incorporate a risk-based analysis for a double levee system containing both local levees and a future project levees. The purpose of this work is to analyze the performance of these two levee systems individually and together as a double levee system, against the influence of storm surge, wave overtopping and rainfall.

A risk based approach is required for all flood risk reduction studies by Engineering Regulation., ER 1105-2-101, Paragraph 7.a

> *"All flood risk management studies will adopt risk analysis as described herein. The risk analysis approach and results shall be documented in the principal decision document used for recommending authorization and/or construction."*

Uncertainty and variability are intrinsic and important in water resources planning and design. Risk and uncertainty arise from measurement errors and from the underlying variability of complex natural, social, and economic situations. If the analyst is uncertain because the data are imperfect or the analytical tools crude, the plan is subject to measurement errors.   The MTG project covers an extremely large area, with a wide variety of coastal and inland hydraulic and hydrologic data. Much of this data has to be adjusted to account large changes for sea level rise in this coastal region.

If the randomness of the data can be described by some probability distribution, based on a historical data base that is applicable to the future, then distributions can be described or approximated by objective techniques. If there is no such historical data base, the probability distribution of random future events can be described subjectively, based upon the best available insight and judgment. The degree of risk and uncertainty generally differs among various aspects of a project.

Reliability analysis is that part of the risk study that leads to an evaluation of the conditional probability of failure (i.e., reliability) of the levee systems and components when they are exposed to the loads of a hurricane. The reliability analysis for MTG had three steps-

1. Specify the components constituting the local levee system and the future project levee system.
2. Define the failure modes of each levee reach.
3. Assign conditional probabilities to the failure states for given water elevations caused by hurricane conditions.

2.4.1   Objective

The purpose of the hydraulic analysis for this project was to do a preliminary investigation to determine potential methods of protecting the study area from damaging storm surges resulting from tropical events. Objectives for this project were identified as follows-

- Evaluate the existing condition, which only considers the local levee performance against various storm surges, resulting from tropical events.

- Evaluate the various project conditions, which consider two project levee heights for three different future year conditions as a result of the estimate relative sea level rise.

The existing condition model simulations for this project were called the 2010 run. The three future year project conditions are the base year 2035 run, which is when construction of the project levee system will be complete to 2035 design elevations, the 2085 run, which is the year when the project is 50 years old, and finally the 2024 run, which is a year in the middle of project construction. At the year 2024 the partially built levee is considered to provide at least some risk reduction to the study area thus generating a benefit before project completion. The basic difference in these project conditions are the change in estimated future sea level rise, which becomes an important factor in coastal areas such as this.

The project levee alternatives, as stated above, have included the performance of the local levees in addition to the project levees. The combination of the two levee systems makes the stage-damage computations for this study very complex. The process of developing frequency curves from the modeling results and applying these frequency curves to the economic analysis became an iterative process with multiple flood damage model run results needing to be combined to provide a true assessment of the damages in the study area. The local and project levees were represented in the economic analysis by fragility curves.

2.4.2   Reliability Analysis

The purpose of this risk and reliability analysis was to determine the reliability of the existing local levees in the study area. A reliability analysis was to give credit to the local levees by determining the probability of failure of the existing levees as a function of the floodwater elevation. The USACE Engineer Technical Letter(ETL) 1110-2-556, *Risk-Based Analysis on Geotechnical Engineering for Support of Planning Studies*, dated 28 May 1999 (Reference 2), was used as guidance. This guidance expired on 30 June 2004 and was rescinded on 4 MAR 2009. The "credit to existing levees" analysis was after the ETL was rescinded.   No other guidance that has replaced this ETL; consequently, it is still the most appropriate document to use as guidance.

The reliability criteria that were considered for the analysis were elevation, geotechnical conditions, history of levee performance, and water elevations. The analysis was conducted using engineering judgment based on available data. Levee stability analysis was performed by district geotechnical engineers. Probabilities of failure were assigned to various water elevations, below the top of the levee, based on the factors of safety determined in the stability analysis. A levee failure methodology was devised by hydraulic engineers to determine when given water elevations would cause levees to fail by overtopping. This resulted in fragility curves for each reach by mode of failure.

2.4.2.1   Levee Breach Methodology

In order to assess the performance of the local levees, a worst case combination (highest stage, lowest levee elevation, worst material composition) was used to construct fragility curves for the non-uniform local levee system. This was not a localized worst case location but a representative worst case reach of levee (lowest and weakest as compared to other reaches) that will also experience the maximum loading on the system. This was developed for all local levees in the study area on a reach-by-reach basis. The reaches distinguish changes in the geometry and/or soil-makeup of the levees.

The levee failures were associated with two principal failure modes-

(1) Levee or levee foundation failure.
(2) Levee erosion caused by overtopping.

This approach represents a simplified analysis to yield generic conditional probability of failure vs. water surface elevation with respect to top of levee. The fragility curves reflect a qualitative evaluation of the major geotechnical aspects of levee integrity.

2.4.2.2   Local Levees

2.4.2.2.1 Local Levee Descriptions

The MTG hurricane and storm surge reduction project study area has existing local levees that are protecting areas of their communities from tidal influences from the Gulf of Mexico. These levees have been designed and constructed by the communities and are not a part of the Corps' federal or non-federal levee programs. An evaluation is necessary to determine the reliability of the existing levees as required by Policy Guidance Letter No. 26, *Benefit Determination Involving Existing Levees*, dated 23 December 1991 (Reference 1).

The local levees constructed by the communities in the study area vary in elevations,

compositions, top width, and side slopes and are scattered throughout the study area. There was little or no levee design documentation available for all the local levees, therefore geometric and engineering material properties of the local levees were determined from aerial photographs, GIS overlays, available geotechnical data, extensive interviews with levee district personnel, and best engineering judgment. A table of local levees and the elevations of the top of levee is shown in Table 12.

2.4.2.2.2 Stability Analysis

Stability analysis was performed on the local levees based on the information gather as described in the previous paragraph. This analysis was performed by Geotechnical Engineers and is described in detail in the Geotechnical section of this report. The results the stability analysis revealed that the local levee systems probability of failure due to stability or under seepage was relatively low for still water elevations reaching to the top of the levee.

2.4.2.2.3 Surface Erosion

The purpose of the performance mode "surface erosion" is to assess the potential for the local levee to be eroded by the wave action and potential free flow. The overtopping rate at which the local levees (1) begin to erode, and (2) will experience catastrophic failure will be calculated. The height of water that equates to the overtopping rate will then be determined. The ETL 1110-2-556 (Reference 2) outlines a procedure to estimate the probability of failure due to erosion. This procedure was followed for the "credit to existing levee" analysis.

2.4.2.2.3.1 **Erosion Analysis**

The minimum average system elevation is 3ft for Levees 1-5 and 11BW11, and a maximum of 10ft for Levee EAST RIDGES - Reach J1. Levee slopes ranged from 1-10 to 1-3.

The overtopping rate at which the levee begins to erode is estimated to be 0.10 cfs/ft. The basis for defining 0.1 cfs/ft as the start of levee erosion is per HSDRRS guidelines. Erosion is expected beyond the 0.1 cfs/ft 90% non-exceedance value associated with levee design. The value at which catastrophic failure of the levee will occur is estimated to be 2.0 cfs/ft.

The corresponding surge to the overtopping rates of 0.10 and 1.00 cfs/ft were calculated via the Van der Meer Equation (EQ 1 and EQ2). As a sensitivity analysis, the surge at overtopping rate 1.50 cfs/ft was also taken and can be found in Table 11. The difference in surge between overtopping rate of 1.00 cfs/ft and 1.50 cfs/ft was usually less than 0.5ft.

EQ 1- $$\frac{q}{\sqrt{gH_{mo}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_0\exp\left(-4.3\frac{h_k}{H_{mo}}\frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

With a maximum-

EQ 2- $$\frac{q}{\sqrt{gH_{mo}^3}} = 0.2\exp\left(-2.3\frac{h_k}{H_{mo}}\frac{1}{\gamma_f\gamma_\beta}\right)$$

where

q - wave overtopping discharge
g - gravitational acceleration
H_{m0} - significant wave height
ξ_0 - breaker parameter = $\tan\alpha / \sqrt{s_0}$
s_0 - wave steepness = $2\pi H_{m0} / (gT^2_{m-1,0})$
$\tan\alpha$ - levee slope
h_k - freeboard of levee above still water level
γ - coefficients for berm (b), roughness (f), angle of incidence (β), vertical wall (v)

In order to determine the reliability of the local levees, historical data was used to investigate the levee performance during past flooding events where the levees experienced significant loading. The historical surge data was used in conjunction with reported levee damage and compared to the calculated surge that represents overtopping failure. If the storm surge known to cause damage was lower than the calculated surge for overtopping, then the calculated value was adjusted down to the observed value to determine the final adjusted critical surge value. The critical surge values calculated using the Van der Meer overtopping equation were always greater than the surge observed during the storms. In the past four years these levees have experienced significant loading due to two hurricanes that both occurred in 2008. Hurricanes Gustav (August 2008) and Ike (September 2008) produced storm surge elevations that reached the local levee alignment in this study area. The hurricane modeling of these two storms which produced hindcast plots showing the stages produced against the local levees can be furnished upon request. The levees that failed during Ike were 5-1B, 8-1, and East Ridges North. In all other reaches the calculated critical surge level will be used.
Table **10** displays adjustments that were made on this principle.

Table 10 - Adjustments made to calculated surge due to known levee failure

System	Name	Levee Elev. (ft.)	Calculated Surge for Overtop 1.0 (cfs/ft)	Ike Observed Surge (ft)	Gustav Observed Surge (ft)	Adjusted Value (ft)
13	EAST RIDGES-North/Montegut	7	7.24	5.00	3.20	5.00
14	8-1S	4	4.39	5.70	4.10	4.39

15	5-1B		6	6.41	5.70	0.00	5.70

2.4.2.2.3.2 **Overtopping Rates**

The surge found with an overtopping rate 0.10 cfs/ft ranged between 3.87 and 9.27ft.
The surge found with an overtopping rate 1.00 cfs/ft ranged between 4.39 and 9.98ft.
The surge found with an overtopping rate 1.50 cfs/ft ranged between 4.57 and 10.61ft.
Results for all levees can be found in Table 11.

Table 11 - Erosion failure heights for local levees

System	Name	Min System Elv (ft)	Levee Slope	Crown Width (ft)	Material	Surge for 0.1 (cfs/ft)	Assoc. 0.1 Wave Height (ft)	Surge for 1.0 (cfs/ft)	Assoc. 1.0 Wave Height (ft)	Surge for 1.5 (cfs/ft)	Assoc. 1.5 Wave Height (ft)
						Surge (ft) for Overtopping of-					
0	BGC4 and HNC8	-	-	-	-	-	-	-	-	-	-
1	MARSH - CENTRAL	-	-	-	-	-	-	-	-	-	-
2	BT5	-	-	-	-	-	-	-	-	-	-
3	MARSH - EAST	-	-	-	-	-	-	-	-	-	-
4	HOUMA - SOUTHEAST	-	-	-	-	-	-	-	-	-	-
5	MARSH - SOUTHWEST	-	-	-	-	-	-	-	-	-	-
6	HNC0	-	-	-	-	-	-	-	-	-	-
7	NORTH - EAST	-	-	-	-	-	-	-	-	-	-
8	LBB5	-	-	-	-	-	-	-	-	-	-
9	BL89	5	0.33	8 - 10	sidecast	4.80	1.47	5.37	1.65	5.54	1.70
10	TRS PROJECT	-	-	-	-	-	-	-	-	-	-
11	BL1-BL7	-	-	-	-	-	-	-	-	-	-
12	UNDEVELOPED - NORTHWEST	-	-	-	-	-	-	-	-	-	-
13	EAST RIDGES - Reach J1	10	0.10	50	MVN P&S	9.27	2.85	9.98	3.06	10.61	3.26
13	EAST RIDGES - Reach J3	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05

System	Name	Min System Elv (ft)	Levee Slope	Crown Width (ft)	Material	Surge (ft) for Overtopping of-					
						Surge for 0.1 (cfs/ft)	Assoc. 0.1 Wave Height (ft)	Surge for 1.0 (cfs/ft)	Assoc. 1.0 Wave Height (ft)	Surge for 1.5 (cfs/ft)	Assoc. 1.5 Wave Height (ft)
13	EAST RIDGES-North/Montegut	7	0.33	10-12	sidecast	5.00*	2.05	5.00*	2.22	7.54	2.31
14	8-1S	4	0.20	8 - 10	sidecast	3.87	1.19	4.39	1.35	4.57	1.40
15	PETITE CALLOU	6	0.25	10 - 12	-	5.74	1.76	6.41	1.97	6.54	2.01
	5-1A	6	0.25	10 - 12	Spec for 5-1A/5-1B	5.74	1.76	6.41	1.97	6.54	2.01
	5-1B	6	0.25	10 - 12	Spec for 5-1A / 5-1B	5.70*	1.76	5.70*	1.97	6.54	2.01
16	3-1C	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
17	3-1B	9.5	0.33	10	MVN P&S	8.81	2.70	9.63	2.95	9.98	3.06
18	8-2C	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
18	8-2D	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
19	D-36	9.5	0.33	10	MVN P&S	8.81	2.70	9.63	2.95	9.98	3.06
20	RIDGE SOUTH (1)	-	-	-	-	-	-	-	-	-	-
21	RIDGE SOUTH (2)	-	-	-	-	-	-	-	-	-	-
22	SLB PROJECT	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
	11BE6-W	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
	11BW79	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
23	RIDGE WEST	-	-	-	-	-	-	-	-	-	-

System	Name	Min System Elv (ft)	Levee Slope	Crown Width (ft)	Material	Surge (ft) for Overtopping of-					
						Surge for 0.1 (cfs/ft)	Assoc. 0.1 Wave Height (ft)	Surge for 1.0 (cfs/ft)	Assoc. 1.0 Wave Height (ft)	Surge for 1.5 (cfs/ft)	Assoc. 1.5 Wave Height (ft)
	(2)										
24	RIDGE WEST (1)	4	0.33	8	sidecast	4.39	1.35	4.48	1.37	4.57	1.40
25	D-30 and D-48	4	0.33	6-8	sidecast	4.39	1.35	4.48	1.37	4.57	1.40
25	D-30	4	0.33	6-8	sidecast	4.39	1.35	4.48	1.37	4.57	1.40
25	D-48	4	0.33	6-8	sidecast	4.39	1.35	4.48	1.37	4.57	1.40
26	BPC5	-	-	-	-	-	-	-	-	-	-
27	NWM - NORTH	8	0.33	8	sidecast	7.54	2.31	8.18	2.51	8.44	2.59
28	D-62	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
29	D-10	6	0.33	8-10	sidecast	5.74	1.76	6.41	1.97	6.67	2.05
30	MARSH HNC 1	-	-	-	-	-	-	-	-	-	-

*Adjusted value due to observed failure during Hurricane Ike.

2.4.2.2.4 Fragility Curves

The performance of the local levee system is defined in the FDA model through a fragility curve. A fragility curve gives the probability of levee failure associated with a given rate of overtopping. The primary mode of levee failure is erosion where the magnitude of erosive forces is dependent upon exterior water elevations. Fragility curves were based on four overtopping rates.

The stability analysis results along with the results from the erosion analysis were used to develop the fragility curves for the all the local levees. Since the stability analysis revealed that a stability analysis had a very low probably of occurring the major failure mode of the local levees would be due to erosion from wave overtopping. The water elevations associated with overtopping rates of 1 cubic feet per second per linear foot of levee were given a probability of failure of 95%. The water elevations associated with overtopping rates of 0.1 cfs/ft were given a probability of failure of 5%. These two points along with the low probabilities associated with the stability analysis were used to

develop the fragility curves.

The reliability assessments that were performed for individual levee reaches resulted in fragility curves for each reach by mode of failure. The process of how the hydraulic results were used to develop the stage damage relationships, considering the reliability of the local levees is detailed in the next section of this report.

Table 12 - Local levee heights and probability of failure water elevations

Economic Reach	HEC-FDA Index	System	Probability of Failure				Top of Levee
			0	0.1	0.45	0.95	
D-16S	379	D-16S	2	3	3.52	3.72	4
BL89	298	BL89	2	3.8	4.4	4.65	5
D-01	367	EAST RIDGES - Reach J1	2	7.5	8.8	9.3	10
PAC1	709	EAST RIDGES - Reach J1	2	7.5	8.8	9.3	10
SL3	718	EAST RIDGES - Reach J1	2	7.5	8.8	9.3	10
D-61	487	EAST RIDGES - Reach J3	2	4.5	5.28	5.58	6
D-61-B	490	EAST RIDGES - Reach J3	2	4.5	5.28	5.58	6
BT4-SA	334	EAST RIDGES- North/Montegut	2	5.3	6.16	6.51	7
4-1S	133	EAST RIDGES- North/Montegut	2	5.3	6.16	6.51	7
D-25	406	EAST RIDGES- North/Montegut	2	5.3	6.16	6.51	7
8-1N	166	8-1S/RIDGE WEST (1)	2	3	3.52	3.72	4
8-1N-B	169	8-1S/RIDGE WEST (1)	2	3	3.52	3.72	4
8-1S-B	175	8-1S/RIDGE WEST (1)	2	3	3.52	3.72	4
BPC3	307	PETITE CALLOU	2	4.5	5.28	5.58	6
BPC4	310	PETITE CALLOU	2	4.5	5.28	5.58	6
4-2C	145	PETITE CALLOU	2	4.5	5.28	5.58	6
4-2B	142	PETITE CALLOU	2	4.5	5.28	5.58	6
4MGT	151	PETITE CALLOU	2	4.5	5.28	5.58	6
BT4	331	PETITE CALLOU	2	4.5	5.28	5.58	6
D-56	481	PETITE CALLOU	2	4.5	5.28	5.58	6
4-2A	139	PETITE CALLOU	2	4.5	5.28	5.58	6
LBC1	670	LBC1	2	4.5	5.28	5.58	6
LBC2	673	LBC2	2	4.5	5.28	5.58	6
5-1A	154	5-1A	2	4.5	5.28	5.58	6
5-1B	157	5-1B	2	4.5	5.28	5.58	6
3-1C	127	3-1C	2	4.5	5.28	5.58	6
3-1B	124	3-1B	2	7.1	8.36	8.835	9.5
8-2C	178	8-2C	2	4.5	5.28	5.58	6

Economic Reach	HEC-FDA Index	System	Probability of Failure				Top of Levee
			0	0.1	0.45	0.95	
8-2D	181	8-2D	2	4.5	5.28	5.58	6
D-36	436	D-36	2	7.1	8.36	8.835	9.5
11BE6-W	25	11BE6-W	2	4.5	5.28	5.58	6
11BW79	64	11BW79	2	4.5	5.28	5.58	6
D-30	421	D-30	2	3	3.52	3.72	4
D-48	466	D-48	2	3	3.52	3.72	4
9-1BMIDE	196	NWM - NORTH	2	6	7.04	7.44	8
9-1AMID	187	NWM - NORTH	2	6	7.04	7.44	8
9-1AE	184	NWM - NORTH	2	6	7.04	7.44	8
9-1AW	190	NWM - NORTH	2	6	7.04	7.44	8
9-1BMIDW	199	NWM - NORTH	2	6	7.04	7.44	8
9-1BW	202	NWM - NORTH	2	6	7.04	7.44	8
D-62-B	496	D-62-B	2	4.5	5.28	5.58	6
D10	373	D-10	2	4.5	5.28	5.58	6
BL2	280	BL2	2	4.5	5.28	5.58	6
BL3	283	BL3	2	4.5	5.28	5.58	6
BL4	286	BL4	2	3.8	4.4	4.65	5
BL5	289	BL5	2	3.8	4.4	4.65	5
BL6	292	BL6	2	3.8	4.4	4.65	5
1-7_N3-4	85	1-7_N3-4	2	4.1	4.84	5.115	5.5
1-7_N4-7	88	1-7_N4-7	2	4.1	4.84	5.115	5.5
1-7_N7-10	91	1-7_N7-10	2	4.1	4.84	5.115	5.5
1-7-N10-13	94	1-7-N10-13	2	4.1	4.84	5.115	5.5
1-7N13-16	97	1-7N13-16	2	4.1	4.84	5.115	5.5
1-7N16-17	100	1-7N16-17	2	4.1	4.84	5.115	5.5
1-7N17-24	103	1-7N17-24	2	4.1	4.84	5.115	5.5
1-7N24-28	106	1-7N24-28	2	4.1	4.84	5.115	5.5
D-29	418	D-29	2	4.9	5.72	6.045	6.5
11BW79-W7	67	11BW79-W7	2	4.1	4.84	5.115	5.5
11BW5	58	11BW5	2	4.1	4.84	5.115	5.5
11BW6	61	11BW6	2	4.1	4.84	5.115	5.5
1-1AB	1	1-1AB	2	3.8	4.4	4.65	5
1-1AN	4	1-1AN	2	3.8	4.4	4.65	5
1-3	79	1-3	2	4.9	5.72	6.045	6.5
1-2S	76	1-2S	2	3	3.52	3.72	4

Economic Reach	HEC-FDA Index	System	Probability of Failure				Top of Levee
			0	0.1	0.45	0.95	
1-5	82	1-5	2	2.3	2.64	2.79	3
4-2	136	4-2	2	3	3.52	3.72	4
4-1N	130	4-1N	2	3	3.52	3.72	4
BL7	295	BL7	2	4.5	5.28	5.58	6
11BW11	40	11BW11	2	2.3	2.64	2.79	3
11BE4	16	11BE4	2	4.5	5.28	5.58	6
4-7	148	4-7	2	4.5	5.28	5.58	6
11BE5	19	11BE5	2	3	3.52	3.72	4
6-1B1	160	6-1B1	2	4.5	5.28	5.58	6
6-1B1-B	163	6-1B1	2	4.5	5.28	5.58	6
D-53	478	D-53	2	3.8	4.4	4.65	5
D-60	484	D-60	2	4.5	5.28	5.58	6
D-64	499	D-64	2	3.8	4.4	4.65	5
E2-LF	517	E2-LF	2	4	4.7	5	5.4
E2-LF-B	520	E2-LF	2	4	4.7	5	5.4

2.4.2.3 Project Levees

The MTG project study project levees will be designed to reduce risk in the study area from storm surge and tidal influences from the Gulf of Mexico. Even though the project levees are designed and engineered to withstand the conditions chosen by the project, a risk and reliability analysis is required to be performed on these levees. The project levees will endure atypical conditions and have to perform differently than other levees in a normal river system. This is primarily due to their continuous exposure to water on both sides of the levees. Additional issues associated with levees include tidal fluctuation, wave run-up; poor foundation conditions (organic soils).

The project levees vary in elevations, based on the alignment and the required risk reduction needed at various locations, based on the hurricane modeling results.

2.4.2.3.1 Stability Analysis

Stability analysis was performed on the project levees as part of the levee design. This analysis was performed by Geotechnical Engineers and is described in detail in the Geotechnical section of this report.

The results the stability analysis revealed that the local levee systems probability of failure due to stability or under seepage was relatively low for still water elevations reaching to the top of the levee and that the major factor contributing to any failure

would be due to erosion by wave overtopping flows.

2.4.2.3.2 Surface Erosion

The erosion analysis performed on the project levees, was done so as part of the levee design. The process is similar to what was done for the local levees with the exception that the designers had much more information about the project levees. The erosion failure heights for the project levees were determined by allowable rated of wave overtopping flows on the levees. The allowable overtopping rates were developed as actual weir flow heights. These weir flow heights had to be converted to actual water surface elevations. The failure probabilities associated with the failure heights were used to construct the fragility cures for the project levees.

2.4.2.3.3 Fragility Curves

The stability analysis results and the results from the erosion analysis were used to develop the fragility curves for the all the project levees. The stability analysis revealed a geotechnical stability failure had a very low probably of occurring, the major failure mode for the project levees would be due to erosion from wave overtopping and/or weir flow. The water elevations associated with various overtopping rates were associated with a probability of failure to construct fragility curves for both the local and federal levees.

This methodology involved combining the stability analysis of the levees with a wave overtopping assessment that identified the stage where a critical overtopping rate fails the levee. The percentage of probability on the fragility curve at the critical stage and above was adjusted to 100% failure if exceeded. This would then combine the probability of failure for other modes of failure below the critical stage with 100% erosion failure above critical stage.

The without-project HEC-FDA model includes fragility curves that describe the reliability of the local levees. These fragility curves were developed using limited available data, stability analyses, and erosion evaluation. Wave overtopping was not considered; if the local levee does not fail, no damages are computed by the HEC-FDA model, even though wave overtopping could result in without-project damages. These assumptions resulted in overstating the performance of the local levee system and thus reduced the without-project damages that were used to compute project benefits.

The economic analysis performed for the with-project conditions represent the Federal levee performance by a single point fragility curve that has a zero percent probability of failure until a wave overtopping rate of 2 cfs/ft is reached, at which time, failure is considered likely. For all events where the wave overtopping is less than 2 cfs/ft, no damages are computed by the HEC-FDA model. The failure point chosen on the Federal levee would overstate residual damages and therefore understate benefits.

Recent full scale wave overtopping simulation research at Colorado State University (CSU) and simulation research projects on levees were performed to determine the need for armoring. This specific analysis forms the basis for the assumed performance of the proposed Federal grass-covered earthen levees. The upper estimate in the CSU tests was 2.0 cfs/ft. Levees could fail if armoring is not present and overtopping is greater than 2.0 cfs/ft. With the likelihood more certain, the failure probability for 2.0 cfs/ft was set at 95 percent.

A sensitivity analysis was performed on an economic reach for the with-project Federal levee with three test cases. Test Case A has a typical fragility curve developed from geotechnical and hydraulic analyses, Test Case B is the single point failure with the 100 percent probability at a surge elevation that results in 2 cfs/ft wave overtopping, and Test Case C is a single point failure with the 100 percent probability surge elevation at the top of levee. The sensitivity analysis demonstrated that the single point failure is a proxy for a typical fragility curve that meets the intent of the ER 1105-2-101 to address risk and obtain economic consequences. The with-project economic analysis using a single point fragility curve with a failure point at 2 cfs/ft wave overtopping understates the performance of the levee, calculating higher residual damages and therefore understating the benefit-cost ratio.

In the HEC-FDA model, once the wave overtopping reaches 2 cfs/ft, the Federal levees fail and interior stage becomes the without-project stage and residual damages are computed. A breach analysis for the with-project Federal levee demonstrated the interior stage with a breach is likely to be lower than the exterior stage and the without-project stage; the interior/exterior relationships used in the Federal levee analysis therefore understates the performance of the levee again understating the benefit-cost ratio.

In reviewing each of these assumptions, the USACE Risk Management Center concluded that the methodology used in this study resulted in underestimating the without-project damages. The analysis performed on the with-project conditions understated the performance of the Federal levees as well as overstated the residual damages due to the interior/exterior relationships used in the economic model.

2.4.3   Approach to Compute Project Benefits

The stage frequency curves developed from external surge stages are used in the Flood Damage Analysis (FDA) Model to test the performance of the local levees for the existing condition. The existing and local levee systems are represented in FDA by constructing fragility curves. A portion of the external/surge volume would become the overtopping wave volume in the with-project condition. A representative stage frequency for wave overtopping will be constructed and used to evaluate the without local levee condition as well. The wave overtopping stage frequency would be

constructed assuming no failure of the project surge levee (i.e. assume wave overtopping only through the full range of frequencies so as not to double count damages).

In order to assess the 3% AEP levee project conditions in FDA two flood damage simulations needed to be performed. The first run used same without project external/surge stages for stage frequency curve, and then evaluated the proposed 3% AEP project levee by constructing fragility curves in FDA. Second run used overtopping of 3% AEP levee to generate stage frequency curves to capture the overtopping wave volume between the project and local levees, then evaluated local levees by using the fragility curves in FDA. These two runs were combined outside of FDA to determine the damages reduced by the 3% AEP alternative including the existing local levees. Assuming that a portion of the external surge volume has been transformed into internal overtopping wave volume (and results in the internal stage frequency loading) then the local levee assessment essentially becomes an interior drainage type of analysis. The combined benefits of both project and local levees would be:

- Without Project EAD – With Project Levee EAD = Project Levee Benefit using surge stage frequency and project design fragility, plus
- Without Local Levee EAD – With Local Levee EAD = Local Levee Benefit using wave overtopping stage frequency and local levee fragility.

The 1% AEP levee project conditions were assessed by using the external/surge stages for stage frequency curve. Since 1% AEP levee meets HSDRRS Criteria, it is logical to assume levee holds back water all the way to top for this case. Given the height/resiliency of 1% AEP levee, the relatively low volume of overtopping and the tremendous storage areas inside the levee, it's reasonable to assume that damages reduced by local levees are not significant. Therefore it was decided not to do second HEC-FDA run for local levees inside the 1% AEP proposed levee.

2.4.4   Benefits During Construction Analysis

The benefits during construction (BDC) analysis will determine the maximum AEP of corresponding surge and wave parameters that a given levee will be able to withstand in year 2024. The 2024 benefit analysis has been conducted with the single point failure elevation in lieu of the fragility curve in the same manner as the 2035 and 2085 with project economic analyses. The use of a single-point of failure as a proxy for the Federal fragility curve meets the intent of ER 1105-2-101 to address risk and obtain economic consequences. This section describes how the single point failure elevation is determined for the BDC analysis. The BDC analysis is performed on the Morganza to the Gulf authorized alignment. Consistent with the 2035 and 2085 with project analyses, the maximum overtopping rate that can be withstood by a levee with grass is approximately 2.0 cfs/ft. To determine the single point failure elevation, the AEP associated with failure needs to be determined; this is the AEP that slightly exceeds the

2.0 cfs/ft overtopping rate. This single point approach is valid because consequences associated with the fragility curve prior to 2.0 cfs/lf overtopping are negligible. A single point elevation was selected because it provided a timely method in representing the levee performance using a conservative approach. Levee performance associated with this method will not overestimate project benefits.

The analysis uses appropriate 2024 levee parameters to determine level of overtopping. Only one reach for the 1% and 3% AEP levee elevations were used. Reaches I3 and H3 were used for 3% AEP and 1% AEP conditions, respectively. Associated levee elevation and geometries are given in Table 13.

Table 13 - Summary of Levee data used to analyze benefits during construction

Return (yr)	Reach	Levee Slope	Berm Toe (ft)	Berm Slope	Berm Toe (ft)	Berm Slope	Berm Top (ft)	Levee Slope	Levee Crest at year 2024 (ft)
35	I3	--	--	1 on 6	3.2	1 on 12	8.1	1 on 6	14.1
100	H3	1 on 3	4.5	1 on 30	6.7	1 on 15	8.9	1 on 6	15.7

Both H3 and I3 sub reaches fall into the same main reach used in other levee design efforts. Reaches H3, I1, I2, I3, J1, J2, and J3 will be used to do the entire analysis. See Figure 11 to view the map of where these reaches are located in the system. The modeled alignment follows the northern most yellow line on the west and the yellow segment alignment on the east.
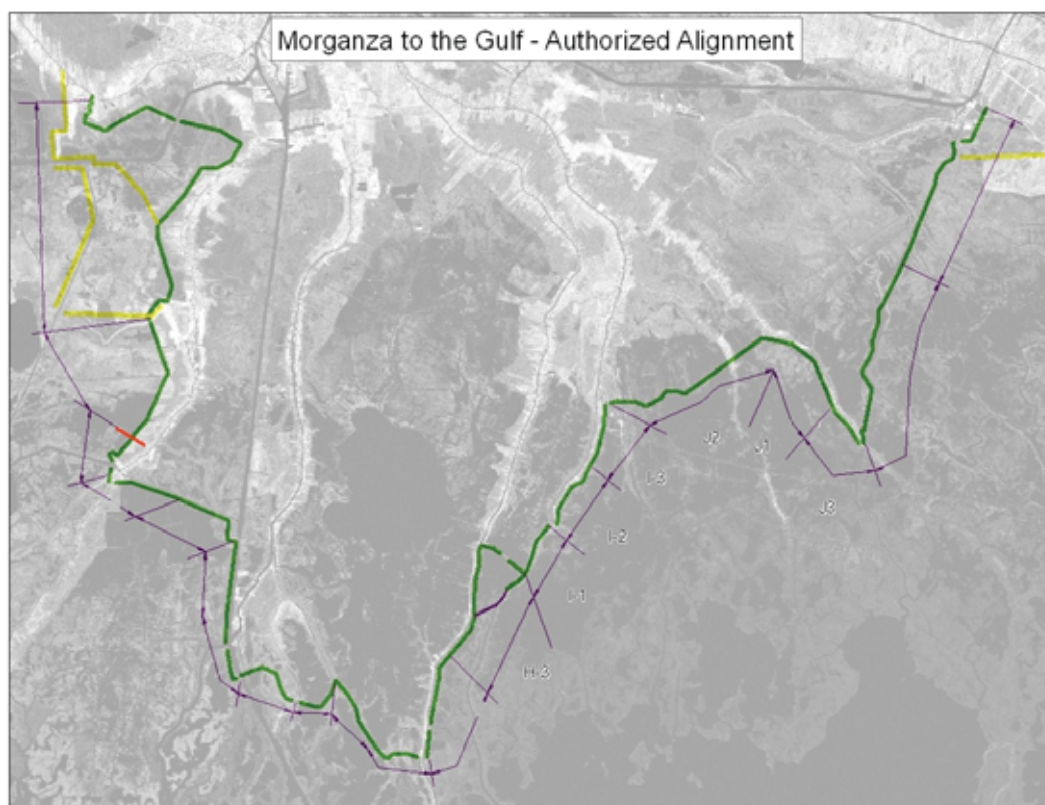
Figure 11 - Map of Morganza to Gulf Authorized Alignment showing Reach H3, I1, I2, I3, J1, J2, J3

The overall task was to determine a single probability. A range of probabilities was used to identify the specific probability associated with failure. In each case, the probability calculated for the partially constructed 1% AEP and 3% AEP levees will be lower than a 1% AEP and 3% AEP, respectively. If the given 1% AEP and 3% AEP design parameters were applied to the partially constructed 2024 levee the overtopping would most likely exceed the 2.0 cfs/ft limit and would definitely exceed the design overtopping limits of 0.01 and 0.1 cfs/ft at the 50% and 90% confidence limits, respectively. Overtopping was calculated for a range of probabilities below the 1% AEP and 3% AEP, respectively. For year 2024 the low SLR rate was 0.39 ft. This SLR rate was used to interpolate for 2024 condition data using existing (0 ft. SLR) and 0.57 ft. SLR data already available. All corresponding 2024 design data are given in Table 14. With the data established for a range of 2024 probabilities the next step was to determine berm factors for each of the scenarios. PC-Overslag as well as an excel sheet was used to develop all berm factors. An example output is given in Figure 12 through Figure 14 for calculating the overtopping for the 1.8% AEP corresponding to the partial 1% AEP levee elevation. The q50 overtopping rate was used to determine the probability event at which failure of the partial levee would occur. From the calculations, at approximately a 1.8% AEP event, the partially constructed 1% AEP levee will fail catastrophically whereas at approximately a 2.3% AEP event, the partially constructed 3% AEP levee will fail catastrophically. See Figure 15 for an example an overtopping scenario using 1.8% AEP boundary conditions.

The failure probabilities developed above have been rounded down to the nearest AEP used in the FDA model; all FDA model runs performed for the study use the same eight AEPs. To be conservative, the partial 1% AEP levee elevation will fail at a 2% AEP event, while the partial 3% AEP levee elevation will fail at a 4% AEP event. Refer to Table 14 through Table 17 for a summary of the results. The step-wise procedure for levee design is described in section 2.6.1.
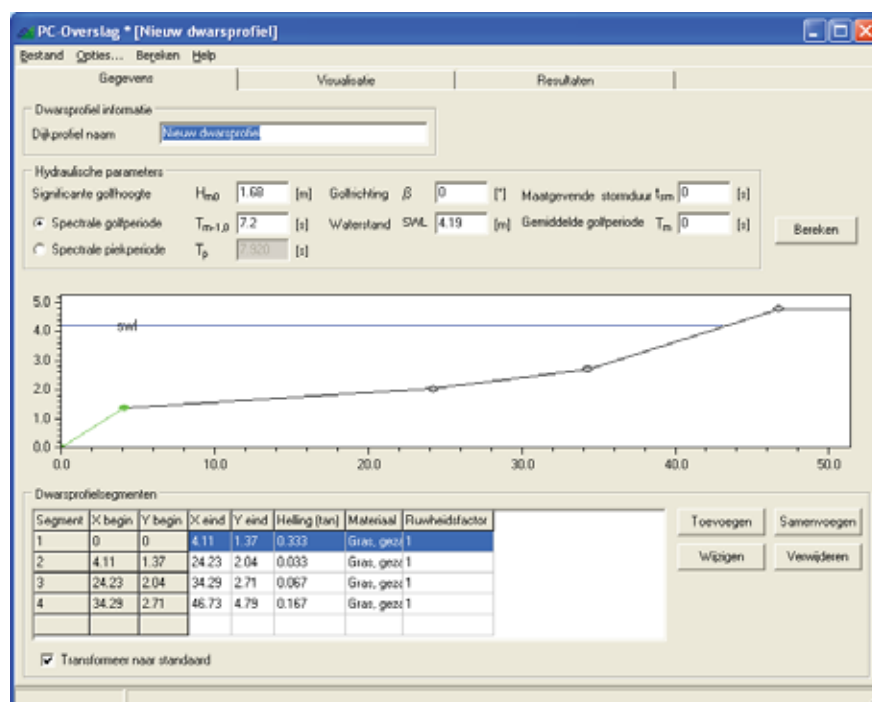
Figure 12 - PC-Overslag geometry section based on 1% AEP levee elevation and 1.8% AEP boundary conditions
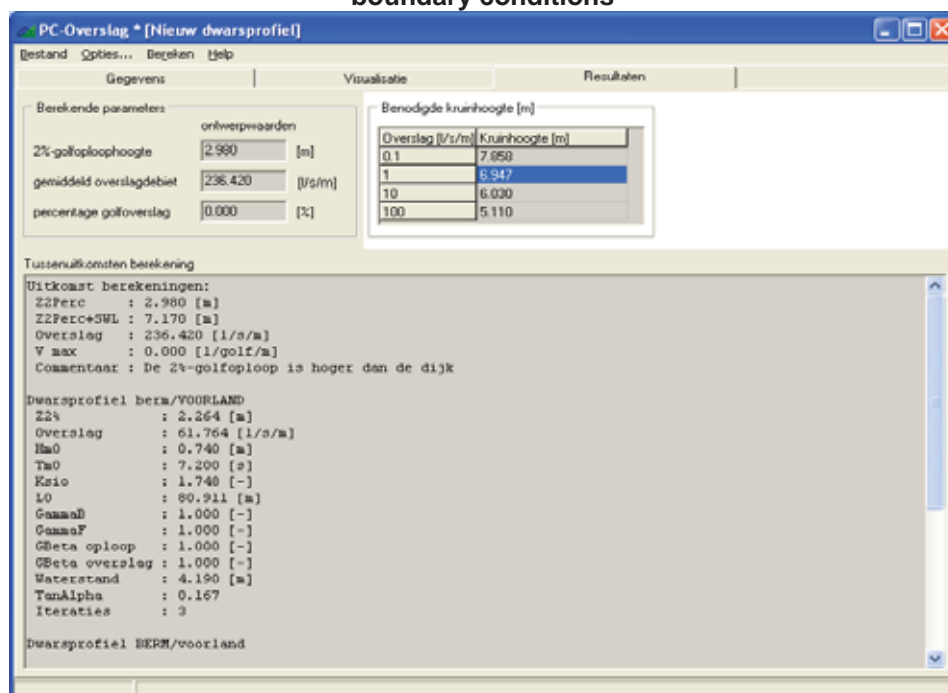
Figure 13 - PC-Overslag output corresponding to 1% AEP levee elevation and 1.8% AEP boundary conditions
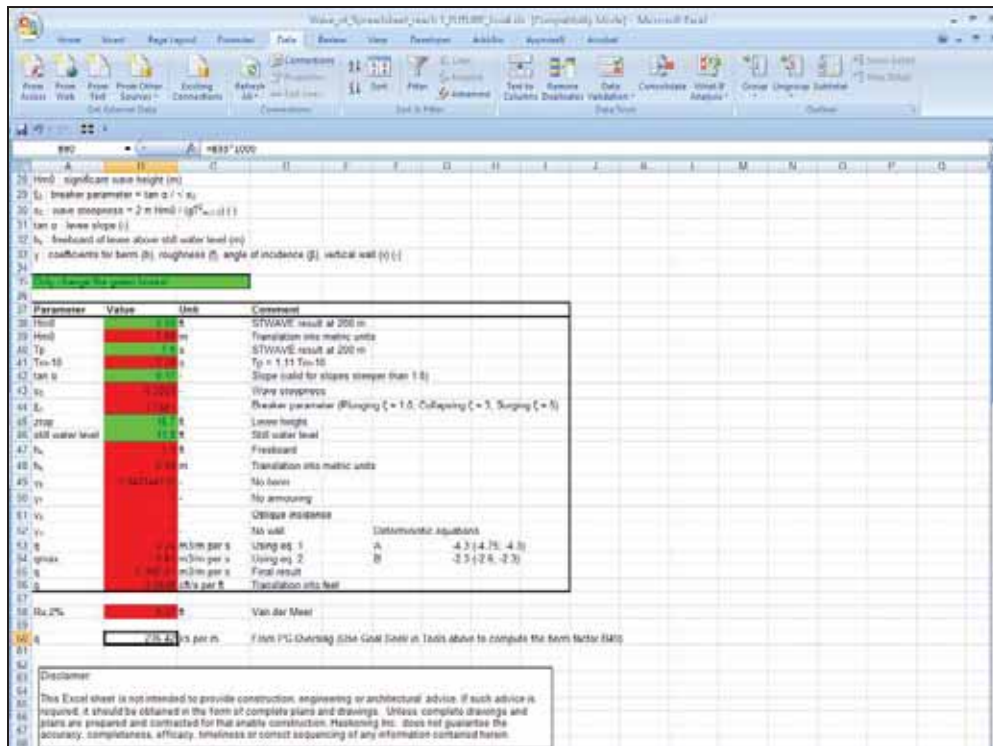
Figure 14 - Excel sheet to find berm factor (gamma b) using 1.8% AEP surge and wave parameters with 1% AEP
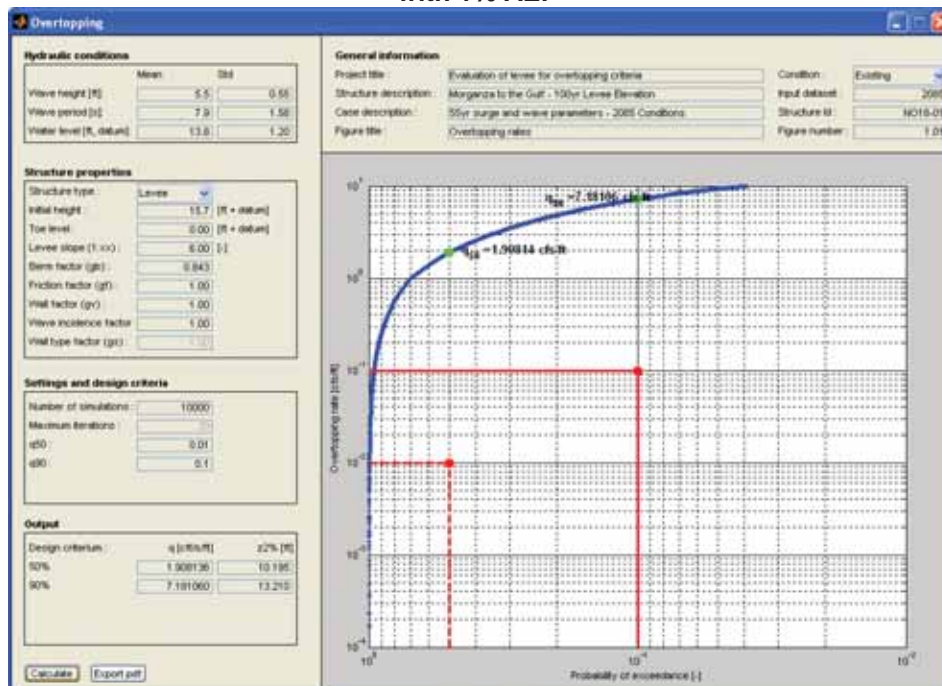
Figure 15 - Matlab overtopping output using 1.8% AEP surge and wave parameters with corresponding 1% AEP. 50 percent confidence overtopping (q50) is approximately 1.90 cfs/ft

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Table 14 - First part of excel sheet showing all "trial and error" probabilities corresponding to 1% AEP

Reach	RETURN	Surge (ft)	Surge (m)	Surge Sdev (ft)	Surge Sdev (m)	Hs (ft)	Hs (m)	Hs StDev (ft)	Hs StDev (m)	Tm (s)	Tm StDev (s)	Tp (s)	Toe of Berm (ft)	Toe of Berm (m)	Top of Berm (ft.)	Top of Berm (m)	Top of Berm (ft.)	Top of Berm (m)
	10	8.0	2.44	0.95	0.29	2.3	0.70	0.40	0.12	3.2	0.3	3.5	4.50	1.37	6.70	2.04	8.90	2.71
	15	9.4	2.87	1.00	0.30	3.1	0.93	0.45	0.14	3.7	0.3	4.1	4.50	1.37	6.70	2.04	8.90	2.71
	20	10.4	3.15	1.04	0.32	3.6	1.10	0.45	0.14	4.0	0.3	4.4	4.50	1.37	6.70	2.04	8.90	2.71
	25	11.1	3.37	1.07	0.33	4.0	1.22	0.45	0.14	4.3	0.3	4.7	4.50	1.37	6.70	2.04	8.90	2.71
H3, I1, I2, I3,	30	11.7	3.57	1.09	0.33	4.3	1.32	0.45	0.14	4.5	0.3	5.0	4.50	1.37	6.70	2.04	8.90	2.71
J2, J1, J3	35	12.2	3.71	1.11	0.34	4.6	1.41	0.45	0.14	4.8	0.3	5.3	4.50	1.37	6.70	2.04	8.90	2.71
	40	12.6	3.84	1.13	0.34	4.9	1.49	0.55	0.17	5.3	0.3	5.8	4.50	1.37	6.70	2.04	8.90	2.71
	45	13.0	3.96	1.15	0.35	5.1	1.56	0.60	0.18	6.2	0.3	6.8	4.50	1.37	6.70	2.04	8.90	2.71
	50	13.3	4.05	1.17	0.36	5.3	1.62	0.60	0.18	7.1	0.3	7.8	4.50	1.37	6.70	2.04	8.90	2.71
	55	13.8	4.19	1.20	0.37	5.5	1.68	0.65	0.20	7.2	0.3	7.9	4.50	1.37	6.70	2.04	8.90	2.71

Table 15 - Second part of excel sheet showing all "trial and error" probabilities corresponding to 1% AEP

Top of Levee (ft)	Top of Levee (m)	Slope 1 (Slope Levee)	Slope 2 (Berm)	Slope 3 (Berm)	Slope 4 (Slope Levee)	x1	x2	x3	x4	q (l/s per m)	γb (Gamma)	FINAL LEVEE HT.	q50 - Overtopping	RETURN CORRESPONDING TO 2.0 CFS/FT OVERTOPPING (q50)	Final return labeled for Benefits during Construction
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7			
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7			
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7			
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7			
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7			
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73			15.7		56	50
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73	12.346	0.765	15.7	0.078		
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73	38.538	0.739	15.7	0.291		
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73	70.815	0.680	15.7	0.545		
15.7	4.79	3	30	15	6	4.11	24.23	34.29	46.73	236.420	0.843	15.7	1.904		

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Table 16 - First part of excel sheet showing all "trial and error" probabilities corresponding to 3% AEP

Reach	RETURN	Surge (ft)	Surge (m)	Surge Sdev (ft)	Surge Sdev (m)	Hs (ft)	Hs (m)	Hs StDev (ft)	Hs StDev (m)	Tm (s)	Tm StDev (s)	Tp (s)	Toe of Berm (ft)	Toe of Berm (m)	Top of Berm (ft.)	Top of Berm (m)	Top of Levee (ft)	Top of Levee (m)
	10	8.0	2.44	0.95	0.29	2.3	0.70	0.40	0.12	3.2	0.3	3.5	3.20	0.98	8.10	2.47	14.1	4.30
	15	9.4	2.87	1.00	0.30	3.1	0.93	0.45	0.14	3.7	0.3	4.1	3.20	0.98	8.10	2.47	14.1	4.30
	20	10.4	3.15	1.04	0.32	3.6	1.10	0.45	0.14	4.0	0.3	4.4	3.20	0.98	8.10	2.47	14.1	4.30
H3, I1, I2, I3,	25	11.1	3.37	1.07	0.33	4.0	1.22	0.45	0.14	4.3	0.3	4.7	3.20	0.98	8.10	2.47	14.1	4.30
J2, J1, J3	30	11.7	3.57	1.09	0.33	4.3	1.32	0.45	0.14	4.5	0.3	5.0	3.20	0.98	8.10	2.47	14.1	4.30
	35	12.2	3.71	1.11	0.34	4.6	1.41	0.45	0.14	4.8	0.3	5.3	3.20	0.98	8.10	2.47	14.1	4.30
	40	12.6	3.84	1.13	0.34	4.9	1.49	0.55	0.17	5.3	0.3	5.8	3.20	0.98	8.10	2.47	14.1	4.30
	45	13.0	3.96	1.15	0.35	5.1	1.56	0.60	0.18	6.2	0.3	6.8	3.20	0.98	8.10	2.47	14.1	4.30

Table 17 - Second part of excel sheet showing all "trial and error" probabilities corresponding to 3% AEP

Slope 1 (Slope Levee)	Slope 2 (Berm)	Slope 3 (Slope Levee)	x1	x2	x3	q (l/s per m)	γb (Gamma)	FINAL LEVEE HT.	q50 - Overtopping	CORRESPONDING TO 2.0 CFS/FT OVERTOPPING (q50)	Final return labeled for Benefits during Construction
6	12	6	5.85	23.77	34.75	0.000	N/A	14.1	N/A		
6	12	6	5.85	23.77	34.75	0.034	N/A	14.1	N/A		
6	12	6	5.85	23.77	34.75	0.697	0.869	14.1	0.003		
6	12	6	5.85	23.77	34.75	4.926	0.870	14.1	0.028	43	25
6	12	6	5.85	23.77	34.75	19.941	0.872	14.1	0.146		
6	12	6	5.85	23.77	34.75	54.472	0.867	14.1	0.384		
6	12	6	5.85	23.77	34.75	134.489	0.866	14.1	1.109		
6	12	6	5.85	23.77	34.75	356.715	0.920	14.1	2.488		

2.4.5　Results and Conclusions

The performance of the existing MTG levees and the lack of reported distress for the past flood events indicates that they have provided some reliable means against flooding. In general, the results of the reliability of local levees analysis tend to show this, as expected. However, the existing levees, for the most part, have not been tested with flood waters in the range on those being used for the design. Therefore, the analyses performed show a wide range of reliabilities from high to low due to uncertainties associated with performance at maximum floodwater and variations in geometry and levee height.

The results of the "credit to existing levees" analysis indicate a wide range of probability of failure of the levees. With the flood water at the top of the levee, the probability of failure is greater than 95%. In general, analysis indicates that the performance of the local levees, in most instances, is controlled by the erodability of the levee. This is what is generally thought to be the case when clay levees are constructed on clay foundations. The stability of the levee is controlled by the strength of the levee material and also the foundation material.

Due to the "discontinuities" of many of these levees, flood water could flank these levees through adjacent lower spots. In addition, some portions of the levees themselves are lower. These "discontinuities" were incorporated into the relationship of the probability of failure to flood elevation. If the flood water elevation was higher than the "discontinuity", then the probability of failure of the levee reach was taken to be 100%.

2.5　FREQUENCY

2.5.1　Introduction

The New Orleans District's Economics Branch uses stage-frequency curves to compute surge-related damages throughout the study area for both "with" and "without" the project in place. Project benefits are based on the difference between the with- and without-project damages and are factored into the benefit cost analysis. To satisfy this need, both with- and without-project stage-frequency curves have been constructed by the Hydraulics & Hydrology (H&H) Branch. For more details, see the Economics Appendix, specifically "Engineering Inputs to the HEC-FDA Model."

Economics Branch calculated stage-damage relationships based on storage areas (SA) established through the interior modeling. The Morganza study area has been divided into 276 sub-areas or reaches (Figure 16). Of the 276 reaches, 264 were determined to be at risk from storm surge. Of the 264 reaches at risk, structures were present in 234 reaches. Hydraulic connectivity information for each storage area will be provided in

Section 2.6.3 and Section 2.6.4. Rainfall was not included in the overall stage frequency curves. The curve development relied on data provided through surge modeling for the upper portion of the curve and gage data for the lower portion of the curve. Composite stage frequency curves were developed for each of these storage areas. All elevations are given in vertical datum NAVD88 epoch 2004.65 unless otherwise stated. All elevations associate with results or input to analyses are given in vertical datum NAVD88 epoch 2004.65 unless otherwise stated. The frequency development only used the intermediate sea level rise case to calculate the BCR.

Figure 16 - Map of storage areas (HEC-FDA reaches) delineated for the interior hydrologic model

2.5.2 Established Risk reduction and Data Sources

Two levels of risk reduction were established for the MTG project:

- 3% Annual Exceedance Probability Storm Surge Risk Reduction System (3% AEP Alternative)
- 1% Annual Exceedance Probability Storm Surge Risk Reduction System (1% AEP Alternative)

The two levels are to be compared through economic analysis to determine which is more cost effective. The originally authorized level of risk reduction was deemed 1% risk reduction prior to Hurricane Katrina. After Hurricane Katrina new design criteria was established. The design elevations associated with the old level of risk reduction were compared against new 1% elevations developed from the new criteria. Results indicated that the old elevations were closer to a 2.86% (35 year) level of risk reduction based on current criteria. Thus, the old 1% level is represented as the 3% AEP Alternative for short throughout the report. In comparison, a new actual 1% AEP post

Hurricane Katrina level of risk reduction will also be included in the analysis.

The economic frequency analysis was performed over existing, base and future conditions. Base and Future conditions correspond to years 2035 and 2085, respectively. All exterior surge and wave modeling for the project was performed by ERDC. Both ADCIRC and STWAVE were used to develop surge and near shore waves, respectively (Cialone et. al., 2010). Detailed information on the modeling effort is given in ERDC's Final Modeling Report. All STWAVE modeling was done in half plane mode.

Table 8 shows all conditions established for the project along with the conditions modeled. A total of 115 synthetic storms were run for existing, 1.15 ft sea level rise (SLR), and 5.0 ft SLR project conditions. A 12 storm sensitivity analysis was done for the 3.2 ft SLR. ERDC computed all statistical values for frequencies of 2% and lower. Historic, rate 1 (intermediate), and rate 2 (high) SLR rates were based on sea level rise formulations derived from Army Corps Engineering Circular 1165-2-211. SLR rates 1 and 2 will be referred to as intermediate and high SLR rates, respectively. Initially, designs were developed for each of the three sea level rise rates for base and future conditions in accordance with the engineering circular. However, the economic cost-benefit analysis was based on only the results associated with the intermediate rate. The main MTG Post Authorization Change Report includes a narrative explanation on how costs and benefits will increase or decrease if the historic or high sea level rise rates became reality. With the modeled SLR scenarios the intermediate rates were interpolated for base and future years.

Two main data sources were used to develop the frequency curves. The sources are as follows.

- ADCIRC Data
    - grid nodes over the project area (surface)
    - 320 MTG point set

- Stage Gage Data

The MTG point data is a subset of 320 nodes from the full ADCIRC nodal data set. Both of these data sources are available for with and without project conditions. The 320 points were selected so that they represented the entire project area (Figure 10). Gage data will be discussed under the without project section of the report.

2.5.3   Without Project

2.5.3.1   Existing Conditions

For without project conditions surge response was much greater on the interior than for with-project conditions. Response was available in storage areas as far inland as surge would propagate. As the frequency of event decreased water propagated further inland.

The same was true in going from existing to future conditions since water elevation increases at a minimum of proportional to the SLR increase. The frequencies established in this and future condition analyses discussed in this report correspond with the specified frequencies chosen for inclusion in the HEC-FDA economic program. HEC-FDA is the main software used by Economic Branch to calculate stage damage relationships. Eight frequencies were selected for the economic analysis including 0.2%, 0.5%, 1%, 2%, 4%, 10%, 20%, and 99.99%. Without project conditions was established first. Still water elevation (SWE) values were available in each storage area over the 0.2% to 2% frequency range. All ADCIRC grid node values lying within each storage area were used to develop the top end of the without project storage area frequency curves. Using all nodal points in each storage area provided a more accurate final representation of each storage area surge elevation. The process for selecting a single representative SWE for each storage area is given below using a 1% frequency event.

1. Map all SWE data available for 1% frequency over project storage areas
2. Determine which storage areas have at least 1 SWE value within
3. Calculate the mean and standard deviation of all values within each SA
4. Take the maximum value in each SA if the coefficient of variation of standard deviation/mean is less than 0.15.
5. If ratio is greater than 0.15, use LIDAR data to manually select a representative SWE.
6. Represent all storage areas that are dry (no surge) with -777.

The limit of 0.15 in step 4 was used as a measure of data scatter. Storage areas with calculated coefficient of variation of less than 0.15 signified a rather minor difference between maximum and minimum values. If larger than 0.15 the difference was rather significant. The limit of 0.15 cannot be found in HSDRRS guidelines since the value was only created as a reference for this analysis. For in depth information regarding HSDRRS criteria, see the *Hurricane and Storm Damage Reduction System Design Guidelines (Interim), New Orleans District, Engineering Division, October 2007*.

Data was summarized for each storage area and respective frequency. Storage area connectivity was reviewed to ensure realistic frequency values were used. For example, frequency values between northern and southern storage areas should have a smooth transition. Also, a check was performed on individual frequency curves to ensure SWE decreased as frequency of event increased. Surge model results were heavily relied upon to complete this task. In some cases engineering judgment had to be used to define a representative value for a storage area. Most of these cases were for either storage areas in the northern reaches of the project where wetting and drying of ADCIRC model nodes occurred sporadically. In some cases, the highest SWE was in a channel within the storage area. Such values were in many instances not consistent with surge elevations over land in the same storage areas. Keeping the highest value would have over inflated structural damages. Instead, the maximum SWE value was manually set to a lower value based on elevations over the entire storage area.

Still water elevations (SWE) were manually set to a lower elevation based on knowledge of all SWE throughout the storage area. Selecting a lower SWE value was dependent upon the specific storage area being analyzed. One example included in the report was when the maximum SWE was found to be in a channel within the storage area. In this case the SWE was lowered to be more consistent with overland surge in that storage area. Wetting and drying of areas from storm to storm also caused some higher than expected statistical results. Such a case was also grounds for setting to a lower SWE which was again dependent upon overland surge in the storage area.

2.5.3.2 Develop Stage Frequency data

Frequency curve development given in the previous section was based on ADCIRC model results for 0.2% through 2% frequencies. To complete the lower part of the frequency curve (i.e., 4% through 50%) another data source had to be used. FEMA data were initially used to obtain 10% frequency values. However, there were known issues with the FEMA data including ground truthing concerns. Use of the FEMA data meant incorporation of a Dokka data adjustment (Cialone et al, 2010) as well as justification on why the data was used when there were notable reservations concerning the FEMA data. Upon further evaluation and discussion, gage data was found to better represent the entire range of high frequency events. Gage data did not need a Dokka data adjustment. Six gage sites were available within the area, but all were lacking decent records except for the gage near Houma, LA and the gage near Leeville, LA (Figure 18). Frequency analysis was done on the Houma and Leeville gages. Both gages resulted in very similar frequency values so only the gage at the Gulf Intracoastal Water Way Houma was used. A summarization of frequency values established through the gage analysis is given in Table 19.

Due to the hydraulic modeling limitations for events with smaller probability of exceedance (50%, 20%, 10%, and 5% AEP events), these events lower on the stage-frequency curves were developed using gage data and a statistical distribution. The Weibull distribution was selected based on the experience and expertise of senior coastal engineers in the MVN H&H branch. The historical gage data was obtained from the USACE Water Management Section and USGS websites. The gage data was converted to NAVD88 2004.65 using the best available conversions from the USACE MVN Surveys Section. The results of the lower stage frequency curves are presented in Table 18.

Figure 17 illustrates the results of the lower stage frequency curves.

The following gages were included in this stage-frequency exercise-
- Houma Canal at Dulac
- Bayou Lafourche at Golden Meadow Floodgate N
- Bayou Lafourche at Golden Meadow Floodgate S
- Intracoastal Water Way at Houma

- Bayou Petit Calliou North of Cocodrie
- Bayou Lafourche at Leeville

Table 18 - Lower Stage Frequency Curve Results

Frequency (%)	50	20	10	5	2
Houma Canal at Dulac	2.7	5.10	5.90	-	-
Bayou Lafourche at Golden Meadow Floodgate N	2.40	3.10	3.40	3.50	-
Bayou Lafourche at Golden Meadow Floodgate S	2.6	3.50	5.90	7.20	-
Intracoastal Water Way at Houma	1.80	2.70	3.00	3.80	4.20
Bayou Petit Calliou North of Cocodrie	1.20	3.20	6.80	8.40	-
Bayou Lafourche at Leeville	1.40	2.30	3.90	4.00	-
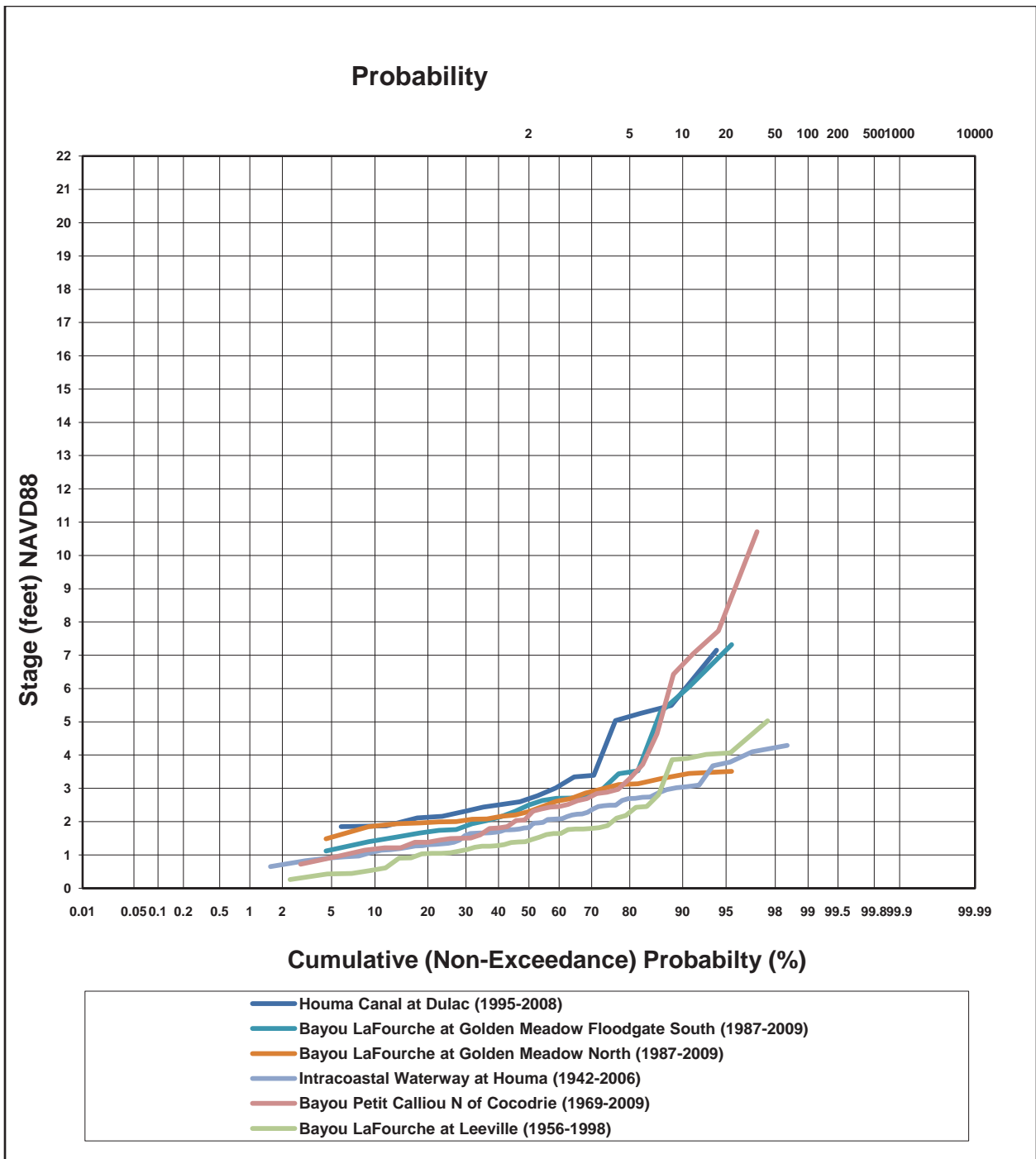
Figure 17 - Lower Stage Frequency Curve Results
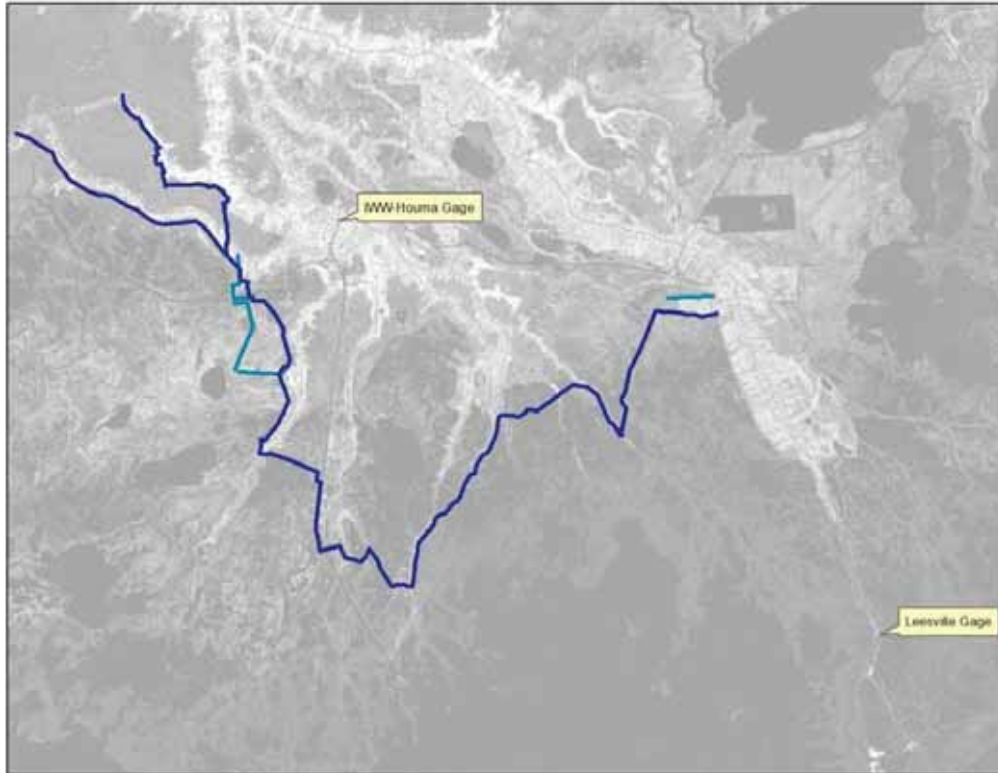
Figure 18 - Map of Leesville and GIWW Houma Stage Gages along with MTG Authorized Levee Alignment

Table 19 - Stage frequencies established through gage analysis

Frequency (%)	50	20	10
Return Period (yrs)	2	5	10
Intracoastal Water Way at Houma	1.9	2.3	3.1

After completion of the gage frequency analysis, a methodology had to be developed to translate these results onto each storage area frequency curve. The most straight forward method was to apply each frequency value directly to each storage area frequency curve. A basic curve was established for all storage areas based on this method. A check was performed on all storage area curves and modifications were made to the 10% through 50% values based on the following main guidelines-

- Frequency values will agree with hydraulic connectivity
- Any adjustment to gage frequency values will be lower than the initial gage value.
- The curve will be smoothed as much as possible with no significant shifts unless a physical boundary is present.
- Frequency values lower than the minimum ground elevation will be labeled as dry.

In case of a value on a given frequency curve being dry the higher frequency events were also labeled as dry. In addition to the frequency values already mentioned, two

additional frequencies needed inclusion on the curves based on the frequencies defined through the economic branch damage analysis. Logarithmic interpolation was performed to compute 0.5% and 4% frequency values. The 10% and 2% values were used to interpolate the 4% frequency while the 1% and 0.2% values were used to define the 0.5% frequency value. For scenarios where either the 10% or 1% value was dry the ground elevation as well as topography was reviewed to make a judgment on whether the 0.5% or 4% frequency value was dry or wet. The final frequency curves for existing, base, and future conditions for each storage area can be furnished upon request.

After adjusting each curve, final quality checks were made on a bigger scale by visualizing storage area frequency values on a map. Every frequency value on each curve had to make sense in light of the entire system. Economic Branch contributed a significant amount of time and effort to assist in the final development of the curves. High frequency surge values might be lowered in such cases where the economic damage analysis showed costs significantly higher than would be expected. In other cases, economic damages at a given frequency event were compared to the same frequency of adjacent storage areas for correlation. If values looked abnormal they were checked and lowered if the high value could not be justified.

2.5.3.3   Base & Future conditions

Base and future condition frequency curves were to be developed by the same method as used for existing without project conditions. Refer back to Table 8 for the sea level rise scenarios that were modeled. Only 0 ft SLR (2010), 1.15 ft SLR (2060), and 5.0 ft SLR (2085) data sets were available to develop base and future conditions. The equivalent intermediate SLR values associated with base and future conditions were 0.72 ft and 2.42 ft., respectively. As mentioned earlier, only the intermediate rate of SLR is important for the stage damage relationship. Refer to EC1165-2-211, dated July 2009, for more information on the three sea level rise rates established for base and future conditions.

Base and future conditions were established after first understanding the model runs. A simple logarithmic interpolation could not be done without first translating the model run data into appropriate intermediate rates of SLR. The 1.15 ft SLR model run corresponds to a year 2060 condition while the 5 ft SLR model run corresponds to a year 2085 condition. Table 20 gives the conversion of each model run to intermediate SLR rates so interpolation could be performed. The translation value shown in the table is equal to the difference between the modeled value and true intermediate value given in Table 8. Taking year 2035 for an example, the intermediate rate is equal to 0.72 ft. The existing condition data as well as modeled 2060 intermediate data needed to be established for interpolating to 2035 intermediate conditions. A translation value of 0.36 ft was added to 2060 SWE values to arrive at intermediate conditions.

Using the actual intermediate SLR values, interpolation for base condition was performed over all nodal points within the project area. Interpolation for the 2035 intermediate SLR value of 0.72 ft could be done using existing condition and 2060

intermediate data. The 2085 intermediate SLR value was established by simply subtracting the translation value in Table 20 from the values associated with the 5.0 ft SLR model results. Interpolation was done spatially over the entire project area. In case of 2035 conditions, the first step was to ensure points for existing and 1.15 ft. SLR data were matched to the same geographic location. This ensured interpolation would be done over the same location. With points geographically matched for base condition, interpolated values were computed at 0.2%, 1%, and 2% frequencies. The same frequencies were also established for intermediate future conditions. At this point all point values were developed for base and future condition analysis. The same step wise process used for development of without project existing condition curve data was used for without project base and future conditions.

Similar to the methodology used to represent the higher frequency events, gage data was used for future conditions after adding a given amount of elevation to account for base and future condition sea level rise. To establish base condition gage data the intermediate SLR rate of 0.72 ft. was added to each of the existing condition gage frequency values. For future conditions, the rate of 2.42 ft. was applied to the existing condition gage values. It's known that sometimes sea level rise does not correlate 1-1 with increase in surge. Without modeling of each specific SLR rate a good approximation was to follow a 1-1 relationship for 2035 and 2085 data development. Table 21 gives a summary of the approximated base and future condition gage values. All methods have been discussed for development of all based and future condition without project frequency curves.

Table 20 - Translation of modeled data to intermediate SLR data

Year	Modeled SLR (ft)	Translation	Final Intermediate SLR (ft)
2010	0	--	--
2035	--	--	0.72
2060	1.15	0.36	1.51
2085	5	-2.58	2.42

Table 21 - High frequency values developed for Base (2035) and Future (2085) conditions

Year	Sea Level Rise (ft)	Gage Frequency Values (ft)		
		10%	20%	50%
2010	0	3.1	2.3	1.9
2035	0.72	3.8	3.0	2.6
2085	2.42	5.5	4.7	4.3

2.5.4   With Project – Levee Overtopping

Stage damage relationships also needed to be established for with project conditions. However, a different approach was taken to build with-project frequency curves. Focusing on the propagation of surge, the intent of with-project conditions was to create a barrier to stop surge from inundating populated areas. Thus, surge would be factored

into the interior storage area frequency curves through levee failure as a direct result of overtopping. An exception to this statement would be the limited number of storage areas that lie outside of risk reduction. For with-project conditions the storage areas outside of the project would be developed in the same manner as the without project frequency curves. Only base and future condition curves were established in this analysis. The intermediate rate of SLR was used for base and future condition with project analysis.

Overtopping was calculated for base and future years based on the 1% and 3% AEP alternatives. A simplified approach was used to calculate overtopping since no modeling was done to simulate levee deterioration during an overtopping event. The approach assumed multiple SWE frequencies as boundary conditions over each levee level of risk reduction. Table 22 shows the boundary conditions applied for each level of risk reduction. All boundary conditions are based on frequencies above the given level of risk reduction in order to produce overtopping. The eight economic frequencies discussed in Section 3.1 were considered when determining frequencies expected to produce overtopping for each level of risk reduction.

Table 22 - Summary of boundary condition frequencies applied to each level of risk reduction for base and future conditions

Level of Protection	Exterior Boundary Conditions			
	2%	1%	0.5%	0.2%
2.86% (35 yr)	X	X	X	X
1% (100 yr)			X	X

2.5.4.1   Storm selection

SWE boundary frequencies have now been selected to be applied to each level of risk reduction. The points used to define those SWE frequencies were based on the MTG 320 point set. The points chosen were the same SWE points used in base and future condition design efforts. A list of the points used at each reach is given in Table 23. Overtopping was computed for each levee elevation and boundary frequency. Base 2035 condition with 1% levee elevation over reach K, L will be used in the example step-wise procedure. Applied as boundary condition to the 1% levee elevation will be 0.2% frequency data. The procedure is given as-

- Determine the 2035 SWE value corresponding to a 0.2% frequency at reach K, L point 174.
- Select a storm out of the 115 storms for with project conditions by searching for a storm that produced a peak surge at point 174 nearest to the 0.2% SWE value.
- Extract the storm surge hydrograph for the given storm at point 174.
- Use a Van der Meer overtopping spreadsheet to compute overtopping flow rate (TAW, 2002).

Before the above procedure could be performed, storm maximum values had to be

developed for 2035 and 2085 conditions. There were no model runs specifically performed for 2035 or 2085 intermediate SLR conditions so interpolation was necessary. Interpolation was done for base and future SLR cases at each of the 320 MTG points. As in previous interpolations the existing, 1.15ft SLR, and 5.0ft SLR storm data sets were used. After the interpolated maximum storm values were computed the above procedure was followed to select storms for all frequencies corresponding to 1% and 2.86% level of risk reductions. In most cases, maximum storm elevations selected had values within 0.50 ft. of the actual frequency SWE value. An example of this is given in Table 24.

Table 23 - Reaches with corresponding point from the MTG 320 point set

Reach	Overtopping Points	
	Surge	Wave
K, L	174	103
H3, I1, I2, I3, J1, J2, J3	96	96
H2	45	31
G1, G2, G3, H1	36	36
E2, E1, F2, F1	83	90
B	90	90
B2	90	90
A-South	90	90
A-North	208	208

Table 24 - Reach K, L data corresponding 0.2% SWE value for 2035 conditions to a given storm

2035 Conditions				
Reach	SWE Point	0.2% SWE (ft)	Closest Storm	Storm SWE (ft)
K, L	174	21.8	8	22.28

2.5.4.2 Overtopping – fragility curves

Using the extracted storm hydrographs overtopping was developed. Overtopping rates in cfs/ft were used as a key indication of expected levee performance. As mentioned previously, an overtopping rate of 2.0 cfs/ft was considered as the failure rate. Besides the catastrophic failure rate other probabilities of failure needed to be defined for the stage damage relationship. A fragility curve was the best way to define a levee based on its ability to withstand overtopping. A fragility curve plots overtopping in cfs/ft versus probability of levee failure for a given levee reach. A number of overtopping rates must be developed to provide a range of probabilities from near 0% to near 100% failure. Five points were to be established on the fragility curve. The rates included 0.1, 0.5, 1.0, 1.5, and 2.0 cfs/ft. The lowest overtopping rate of 0.1 cfs/ft corresponded to the allowable design rate while the 2.0 cfs/ft corresponded to catastrophic failure.

Overtopping rates were established through an overtopping spreadsheet that utilizes

the Van der Meer overtopping equation. Main inputs to the spreadsheet include Top of Levee, SWE, Hs, Tm, and a storm hydrograph. The statistical surge and wave data were based on the reach and frequency being analyzed. Storm hydrographs were extracted from existing surge runs in front of a given levee reach and frequency. This means a different storm hydrograph was used for each levee reach and frequency. The storm hydrograph was normalized to the input SWE so as to produce a peak of the storm hydrograph equal to the input SWE. The same was done for Hs and Tm where the input values corresponded to the same time on the hydrograph as the input SWE. The hydrograph had a time-step of 15 minutes. At each time-step the free flow and wave flow was calculated. An example of spreadsheet and overtopping hydrograph for a given levee reach are given in Figure 19 and Figure 20, respectively.

Plots of SWE versus overtopping rate were produced for each reach and frequency. For the 1% AEP, the 0.5% and 0.2% AEP boundary conditions were applied. For the 3% AEP, 2%, 1%, 0.5%, and 0.2% AEP boundary conditions were applied. SWE corresponding to the five main overtopping rates needed for the fragility curve were established through these plots. Figure 21 gives an example of one of the SWE versus overtopping plots along with an arrow approximating the appropriate SWE. SWE values were provided to the nearest tenth. If all plotted curves for a given reach did not fall near each other the minimum SWE needed to reach a specific overtopping rate was taken as the representative value. For example, if a SWE of 17.5 ft corresponded to a 2.0 cfs/ft limit for the 0.5% AEP curve and 17.2 ft SWE for the 0.2% AEP curve then the value that governs is the 17.2 ft SWE. Also, in cases where a 2.0 cfs/ft overtopping rate was not reached it was approximated. We expect little failure risk when a considerable height of free board exists on the levee. The risk is expected to increase at a much more dramatic rate as the water level rises and freeboard lessens. As SWE increases the risk will asymptotically approach catastrophic failure. All top of levee information was provided by the project management team (Table 25). These values are not the same as the levee design elevations provided by H&H. Subsidence and lift schedules were taken into account to develop the final top of levee elevations used in this analysis.

Table 25 - Top of levee values corresponding to each condition for 3% and 1% levels of risk reduction

3% AEP Alternative							
Reach	**2024 Actual**	**2035 Actual (before lift)**	**2024 to 2035 Average Actual**	**2024 Top of Levee for HEC-FDA**	**2035 Top of Levee for HEC-FDA**	**2085 Top of Levee for HEC-FDA**	**Basis for 2035**
Barrier/A	10.7	10.5	10.6	10.5	12.5	13.0	2070
B	13.8	12.8	13.3	13.0	12.5	13.5	2050
E	16.0	14.0	15.0	15.0	16.5	15.5	2035, 2085
F	15.0	14.5	14.8	14.5	15.0	15.5	2065
G	17.5	17.0	17.3	17.0	17.0	17.5	2050
H	19.0	18.3	18.6	18.5	20.5	20.0	2035, 2085
I	19.7	19.0	19.4	19.0	19.0	20.0	2045
J	20.5	18.5	19.5	19.5	19.5	20.0	2070
K	17.5	16.0	16.8	16.5	18.0	17.5	2035, 2085
L	17.5	18.3	17.9	17.5	18.0	17.5	2035, 2085

1% AEP Alternative							
Reach	**2024 Actual**	**2035 Actual (before lift)**	**2024 to 2035 Average Actual**	**2024 Top of Levee for HEC-FDA**	**2035 Top of Levee for HEC-FDA**	**2085 Top of Levee for HEC-FDA**	**Basis for 2035 Value**
Barrier/A	16.7	15.8	16.3	16.0	19.0	20.5	Around 2068
B	16.0	19.4	17.7	17.5	19.0	20.5	2055
E	16.0	14.0	15.0	15.0	22.0	23.5	2045
F	16.0	14.0	15.0	15.0	23.0	23.5	2060
G	19.2	18.0	18.6	18.5	24.0	24.0	2035, 2085
H	18.2	24.8	21.5	21.5	24.5	26.5	2045
I	19.6	24.8	22.2	22.0	24.5	26.5	2045
J	19.5	25.8	22.7	22.5	25.0	26.5	2055
K	18.5	24.4	21.5	21.5	24.0	25.5	2050
L	18.5	24.0	21.3	21.0	23.0	25.5	2045

Notes:

2024 value based on straight average of 2024 and 2035 predicted elevations.

2035 value based on "average" of predicted elevations between 2035 and 2085 using one of two methods:
1. If "Basis for 2035 Value" is a single year, it means elev. based on point immediately before last lift (completely settled levee) rounded to the nearest half foot.
2. If "Basis for 2035 Value" has multiple years, means elevation is based on average along settlement curve between those two years.

2085 value based on predicted elevation at 2085 (similar to final design elevation).

Table 25 shows the "top of levee" elevations used in the 2024, 2035, and 2085 HEC-FDA models for the risk-based economic benefits analysis. The "top of levee" elevations are based on predicted crown elevations and lift schedules from USACE, MVN, Geotechnical Branch. In order to maintain the levee crown at or above the base year (2035) and future year (2085) design elevations while accounting for levee settlement and relative sea level rise, levees would be constructed in multiple lifts over

the construction period of approximately 2015 to 2070. Both the design elevations and constructed "top of levee" elevations vary by levee reach. Design elevations vary by levee reach because of surge and wave differences due to storm path, wind speeds and direction, etc. Because of the long construction schedule, not all reaches will achieve the design level of risk reduction at the same time. By 2024, there will be a "closed system" with first-lift levees and structures in place but the "top of levee" elevations may be above or below the design elevations depending on the reach. By 2035, all "top of levee" elevations will be at or above the 2035 base design elevation, and by 2085 all "top of levee" elevations would be at the 2085 future design elevations.

The last step of the process was assigning each of the five overtopping rates to specific probabilities of failure. This final plot was created for base and future conditions and each levee reach. Figure 22 gives a plot of overtopping versus probability of failure. The rate of 2.0 cfs/ft was given a failure probability of 95%. Statistically a failure probability cannot reach 100% because there is always a small amount of uncertainty. Probabilities for the 0.5 and 1.5 cfs/ft rates were arrived at by smoothing out the curve knowing that the curve should be more S-shaped than linear with inflection at the 1.0 cfs/ft probability. The rationale for assigning probabilities is subjective to the engineer, their prior experience, and the conditions present for the levee under study. Geotechnical engineers and risk analysis experts were consulted during the development of these fragility curves. The inflection points based on overtopping rates were determined as a result of levee overtopping erosion tests performed at Colorado State University for other post-Katrina MVN levee designs. Reading from the curve we note the 0.5 and 1.5 overtopping rates are equal to 6% and 85% probability of failure, respectively. Final summary tables are given in Table 26 through Table 29

Figure 19 - Example of overtopping spreadsheet used to calculate overtopping rates. Neon green cells indicate changeable inputs.
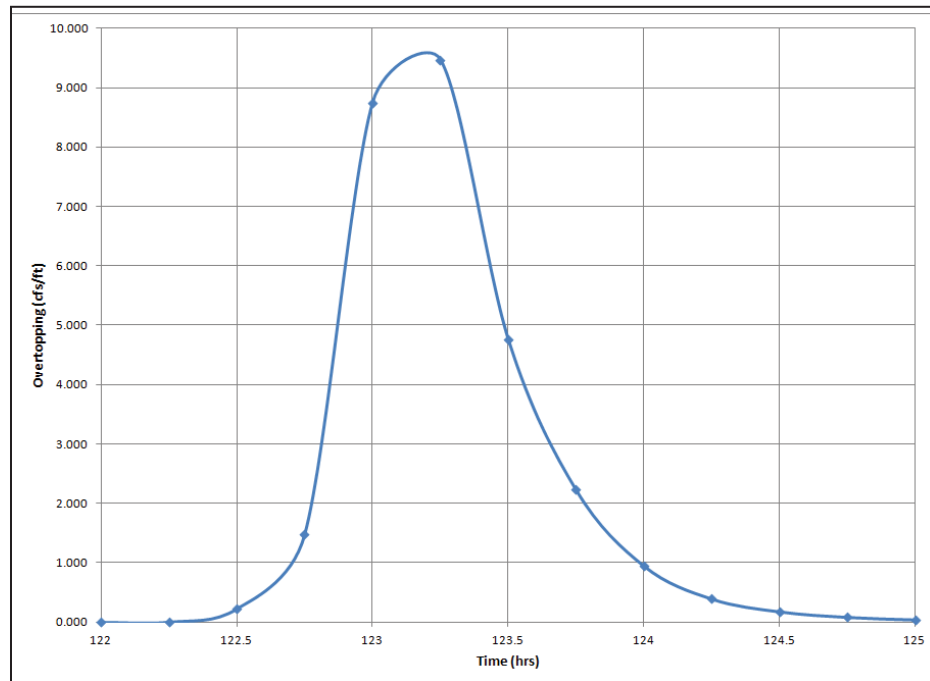
Figure 20 - Example hydrograph of time versus overtopping rate
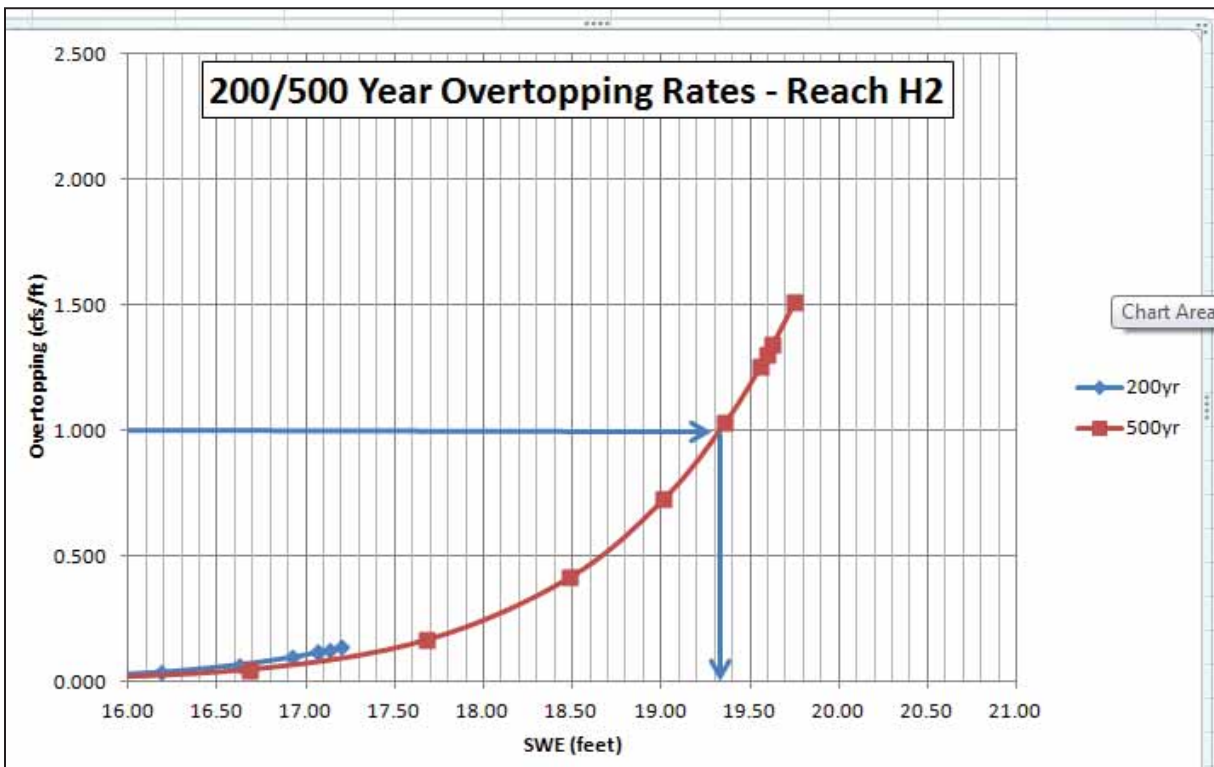
Figure 21 - Plot of SWE versus Overtopping for 0.5% and 0.2% AEP Boundary Conditions over Reach H2.
In this example, the 2.0 cfs/ft rate had to be approximated
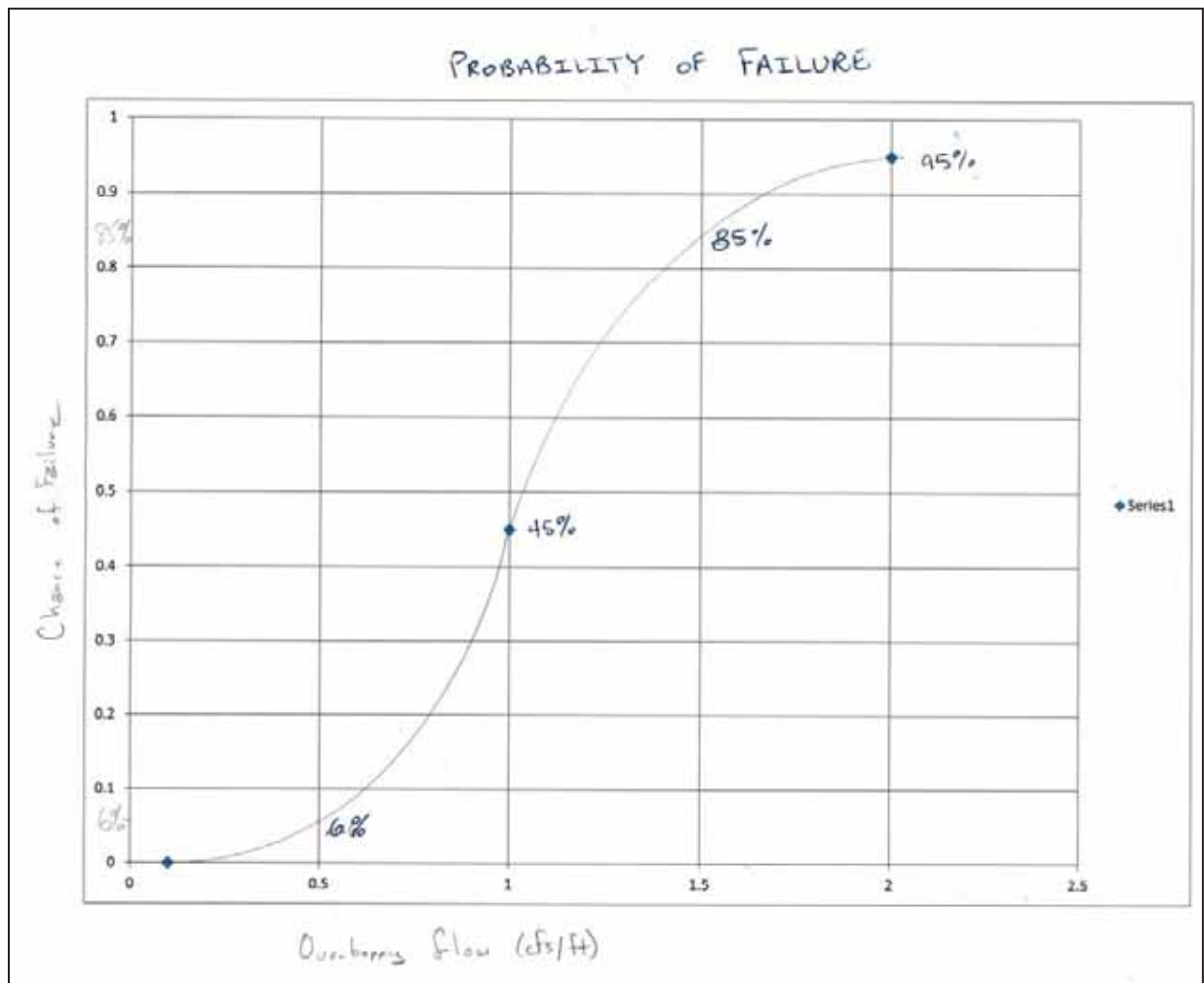
Figure 22 - Plot of overtopping versus probability of failure

Table 26 - Fragility curve data for year 2035, 3% AEP

2035	35 yr levee	Overtopping Rate (cfs/ft)				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
12.5	A	7.1	9.5	10.0	10.3	10.5
12.5	B	9.1	9.7	10.2	10.5	10.8
16.5	E	11.4	13.2	13.7	14.0	14.2
15.0	F	11.4	12.1	12.7	13.0	13.3
17.0	G	11.4	12.2	12.6	12.9	13.1
20.5	H	12.5	14.9	15.5	15.9	16.2
19.0	I	12.5	13.9	14.5	14.9	15.1
19.5	J	12.5	14.3	14.8	15.2	15.5
18.0	K	11.8	14.0	14.5	14.8	15.1
18.0	L	11.8	13.9	14.5	14.8	15.1

Table 27 - Fragility curve data for year 2035, 1% AEP

2035	100 yr levee	Overtopping Rate (cfs/ft)				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
19.0	A	10.4	14.7	15.3	15.7	15.9
19.0	B	12.4	15.6	16.1	16.4	16.5
22.0	E	15.2	18.8	19.3	19.6	19.8
23.0	F	15.2	19.6	20.1	20.4	20.6
24.0	G	14.8	18.1	19.0	19.3	19.5
24.5	H	16.3	19.1	19.7	20.2	20.5
24.5	I	16.3	19.1	19.7	20.2	20.5
25.0	J	16.3	19.4	20.2	20.6	20.9
24.0	K	16.1	20.1	20.5	20.8	21.0
23.0	L	16.1	19.4	20.0	20.2	20.3

Table 28 - Fragility curve data for year 2085, 3% AEP

2085	35 yr levee	Overtopping Rate (cfs/ft)				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
13.0	A	9.8	10.5	10.8	11.0	11.2
13.5	B	10.7	10.8	11.1	11.2	11.4
15.5	E	12.5	12.6	12.9	13.1	13.5
15.5	F	12.5	12.6	12.9	13.1	13.5
17.5	G	12.5	12.6	13.1	13.4	13.6
20.0	H	13.9	14.4	15.1	15.5	15.8
20.0	I	14.0	14.4	15.1	15.5	15.8
20.0	J	14.0	14.4	15.1	15.5	15.8
17.5	K	13.1	13.5	13.9	14.1	14.4
17.5	L	13.1	13.5	13.9	14.1	14.4

Table 29 - Fragility curve data for year 2085, 1% AEP

2085	100 yr levee	Overtopping Rate (cfs/ft)				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
20.5	A	13.7	17.3	17.8	18.1	18.3
20.5	B	14.2	16.6	17.1	17.4	17.7
23.5	E	16.6	19.7	20.2	20.6	20.8
23.5	F	16.6	19.7	20.2	20.6	20.8
24.0	G	16.2	18.0	18.9	19.3	19.6
26.5	H	17.8	20.2	21.2	21.6	21.8
26.5	I	17.8	20.1	21.2	21.6	21.8
26.5	J	17.8	20.2	21.2	21.6	21.8
25.5	K	17.7	20.3	20.9	21.4	21.8
25.5	L	17.7	20.3	20.9	21.4	21.8

After the above analysis was completed, a sensitivity analysis was performed on an economic reach for the with-project Federal levee with three test cases. Test Case A has a typical fragility curve developed from geotechnical and hydraulic analyses, Test Case B is the single point failure with the 100 percent probability at a surge elevation that results in 2 cfs/ft wave overtopping, and Test Case C is a single point failure with the 100 percent probability surge elevation at the top of levee. The sensitivity analysis demonstrated that the single point failure is a proxy for a typical fragility curve that meets the intent of the ER 1105-2-101 to address risk and obtain economic consequences. Based on this analysis, Case B, the single point failure elevation at the 2 cfs/ft wave overtopping was used for the project evaluation. See Section 2.4.2.3.3 for further discussion.

2.5.4.3   Benefits during construction - year 2024

In addition to the with project analysis for base and future years a separate analysis looked into possible benefits obtained by having the levee in place before reaching a final levee elevation in year 2035. By year 2024 the levee system would be in place and would offer a moderate amount of risk reduction however limited that may be. Both the 1% and 3% AEP were included in this investigation. As discussed previously, input data used for base and future condition levee design was used as boundary conditions for the with project overtopping analysis. For 2024 conditions the existing condition and 2035 year data were used to interpolate SWE, Hs, and Tm for 2024 conditions. The 2024 levee elevations used for 1% and 3% AEP are given in Table 25 also. Overtopping results were translated to fragility curves through the same process done for base and future years. Fragility curve information is summarized in Table 30 and Table 31.

Table 30 - Fragility curve data for year 2024, 3% AEP

2024	35 yr levee	Overtopping Rate				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
10.5	A	6.2	8.6	9.0	9.2	9.4
13.0	B	8.7	10.0	10.5	10.9	11.1
15.0	E	11.1	11.7	12.3	12.9	13.2
14.5	F	11.1	11.8	12.4	12.7	13.0
17.0	G	11.1	12.0	12.4	12.7	12.9
18.5	H	12.2	13.6	14.2	14.5	14.8
19.0	I	12.2	13.7	14.3	14.7	14.9
19.5	J	12.2	14.1	14.6	15.0	15.3
16.5	K	11.3	12.9	13.4	13.7	14.0
17.5	L	11.3	13.6	14.1	14.5	14.7

Table 31 - Fragility curve data for year 2024, 1% AEP

2024	100 yr levee	Overtopping Rate (cfs/ft)				
		0.01	0.5	1.0	1.5	2.0
		Probability of Failure				
TOL	Reach	0.004	0.06	0.45	0.85	0.95
16.0	A	9.3	12.3	13.1	13.4	13.6
17.5	B	11.9	14.4	15.0	15.2	15.5
15.0	E	14.9	11.8	12.5	13.0	13.3
15.0	F	14.9	11.9	12.5	12.9	13.2
18.5	G	14.6	13.0	13.8	14.2	14.5
21.5	H	16.0	15.8	16.4	16.8	17.2
22.0	I	16.0	16.9	17.5	17.9	18.2
22.5	J	16.0	17.2	17.8	18.2	18.5
21.5	K	15.6	16.5	17.0	17.5	17.8
21.0	L	15.6	16.2	16.6	17.0	17.3

2.5.4.4   Storage Areas Outside Risk Reduction Area

A limited number of storage areas were totally outside of the leveed area and others were hydraulically separated into two sections due to levee placement. Table 32 provides a list of storage areas lying at least partially outside the leveed area.

Table 32 - Storage areas outside leveed area

Storage Areas Outside Protection					
6-1B1	BB8	D-16N	D-35	E1-LF	GW18
8-1N	BD1	D1A	D-45	E2	
8-1S	BDL4	D-25	D-61	HNC10	
A1	BPC5	D-34N	D-62	HNC9	
BB7	BT5	D-34S	E1	E2-LF	

The storage areas could not be associated with a fragility curve since they were outside of a levee. Instead, with project frequency curves for base and future conditions were created for these storage areas as done for without project conditions. Refer to the methodology established in Section 2.5.3.2 for without project condition frequency curve development.

2.6   HYDRAULIC DESIGN

Design criteria established for levee and structure design has evolved since Hurricane Katrina. Much of the methodology used for analyzing statistical storm frequency in the southeastern part of Louisiana has changed significantly. Overtopping criteria defined for levee and structure design has also changed. The study initiated prior to 2005, therefore many of the results established previous to the new design criteria were reviewed. The design effort in this report was separated into two main sections.

- 1% Annual Exceedance Probability Storm Surge Risk Reduction System (1% AEP Alternative)
- 3% Annual Exceedance Probability Storm Surge Risk Reduction System (3% AEP Alternative)

The 1% AEP Alternative will calculate levee and structure designs based on the latest design criteria. The 3% AEP Alternative refers to the level of risk reduction authorized by congress prior to the establishment of the latest design criteria. Both alternatives were based on the same levee alignment (Figure 23). Levee and structure design calculations will be performed for both 1% AEP Alternative and 3% AEP Alternative scenarios. The designs will then be evaluated and compared. The most suitable alternative will be selected for further study. All background design plots for levees, structures, and wave loads are given in can be furnished upon request.

2.6.1   1% AEP Storm Surge Risk Reduction System Levee Designs

Levee designs were established for each with project condition based on statistical results from the hydrodynamic modeling effort. All designs were performed by engineers in the USACE MVN Hydraulics & Hydrologic Branch. The alignment was broken into hydraulic reaches based on surge and wave response in front of the alignment. Sub reaches that have similar surge elevations are kept in the same main reach while corners and turns in the alignment sometimes indicated a need for starting a new reach. The hydraulic reaches stayed consistent throughout all levee designs (Figure 23). The reaches are as follows-

1. Reach K, L
2. Reach H3, I1, I2, I3, J1, J2, J1, J3
3. Reach H2
4. Reach G1, G2, G3, H1
5. Reach E2, E1, F2, F1
6. Reach B
7. Reach B2
8. Reach A South of GIWW
9. Reach A North of GIWW
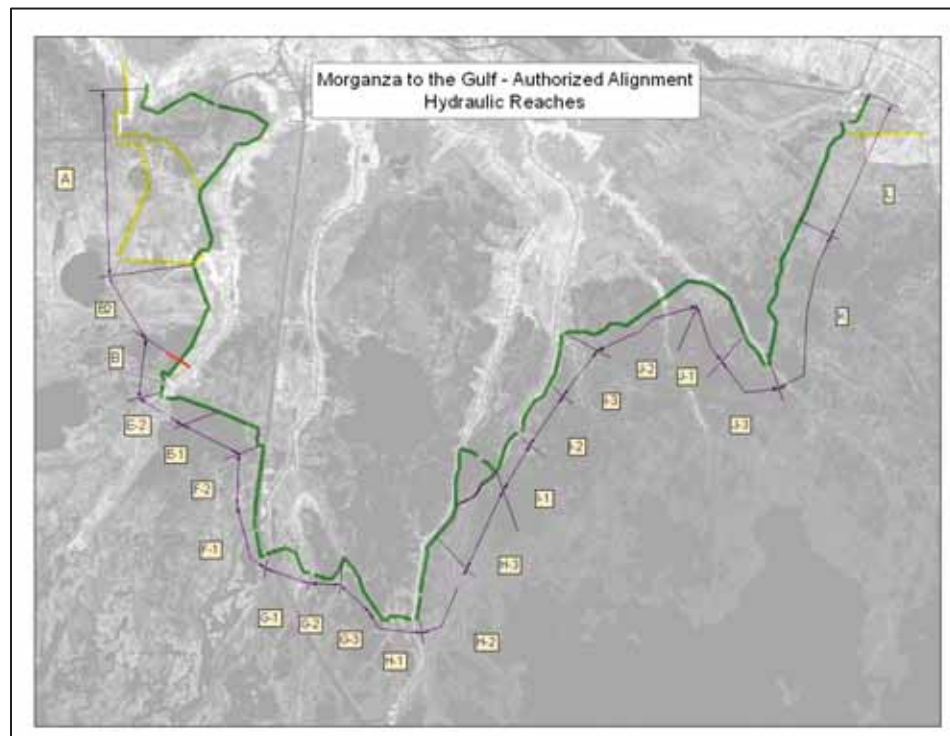
Figure 23 - Hydraulic Reaches associated with the MTG Authorized Alignment

Reach A is not the end of the authorized alignment. Referring back to Figure 23, the levee continues along the high ridge west of Reach A until tying into higher ground. The section west of Reach A will be considered later in this report. The same hydraulic reaches were used in both the 1% AEP Alternative and 3% AEP Aternative analyses to keep all designs comparable. The design procedure applied is common to all levee designs performed by USACE MVN Hydraulics & Hydrologic Branch. A step-wise procedure will now be given for levee design.

1. Choose a hydraulic reach to perform a levee design. For an example, Reach E will be used.

2. Determine which of the 320 points are in front of Reach E and find the point with maximum surge elevation as well as the point with maximum Hs and Tm. In most cases the surge and wave points are the same; however they can be different if maximums do not occur at the same point over the reach.

3. Note the standard deviations associated with SWE, Hs, and Tm.

4. If a berm will be added to the flood-side of the levee then a berm factor will need to be calculated. If no berm will be included skip to step 7.

5. Use the Dutch-originating software PC-Overslag to calculate a linear overtopping rate, q, corresponding to various possible geometries for the levee. The value of q is calculated in liters per second per meter and cannot exceed 1.5 l/s per meter (0.1 cfs/ft) as per HSDRRS guidelines, for example, Figure 24 and Figure 25.

6. Take the calculated q with surge and wave parameters and proceed to a customized excel sheet with automated calculations from the Dutch TAW manual. The TAW manual has equations related to wave run-up and overtopping associated with dikes. The excel sheet will calculate a berm factor for a given geometry (Figure 26).

7. Run a probabilistic Monte Carlo simulation using surge and wave parameters along with corresponding berm factor and geometry details. The probabilistic calculations produce 50% and 90% confidence overtopping rates q50 and q90, respectively, based on the Van der Meer overtopping equation. Use 10,000 iterations for the Monte Carlo simulations (Figure 27).

8. Check the q50 and q90 values. According to HSDRRS guidelines, the q50 cannot exceed 0.01 cfs/ft and the q90 cannot exceed 0.1 cfs/ft. When all criteria is satisfied the ending levee elevation is the final design elevation.

9. Iterations are sometimes needed to produce the most efficient levee elevation. If the overtopping rates are too low then decrease the levee height by 0.5 feet. If overtopping is too high increase by 0.5 feet until criteria is met.

As outlined in the New Orleans Hurricane Protection Manual the Monte Carlo Analysis described in the procedure above is executed as follows-

1. Draw a random number between 0 and 1 to set the exceedance probability p.

2. Compute the water elevation from a normal distribution using the mean 1% surge elevation and standard deviation as parameters and with an exceedance probability p.

3. Draw a random number between 0 and 1 to set the exceedance probability p.

4. Compute the wave height and wave period from a normal distribution using the mean 1% wave height/wave period and the associated standard deviation and with an exceedance probability p.

5. Repeat step 3 and 4 for the three overtopping coefficients independently.

6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in step 2, 4, and 5.

7. Repeat steps 1-5 a large number of times (i.e., N=10000)

8. Compute the 50% and 90% confidence limit of the overtopping rate (i.e., q50 q90)
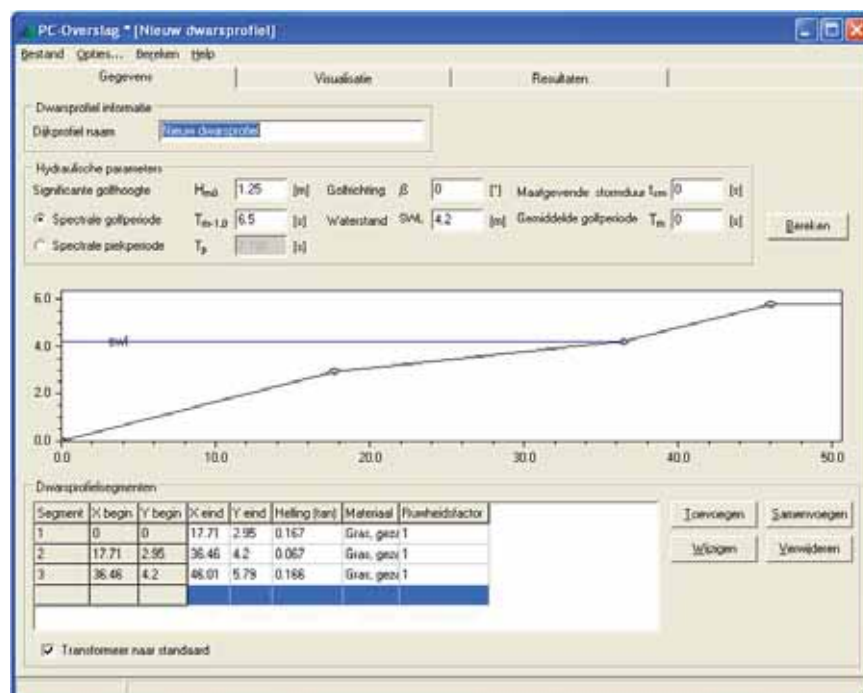
Figure 24 - Example of PC-Overslag input screen. Levee geometry information is shown along with surge and wave values

Figure 25 - Example of PC-Overslag result screen

Figure 26 - Example of TAW Manual berm factor calculation sheet
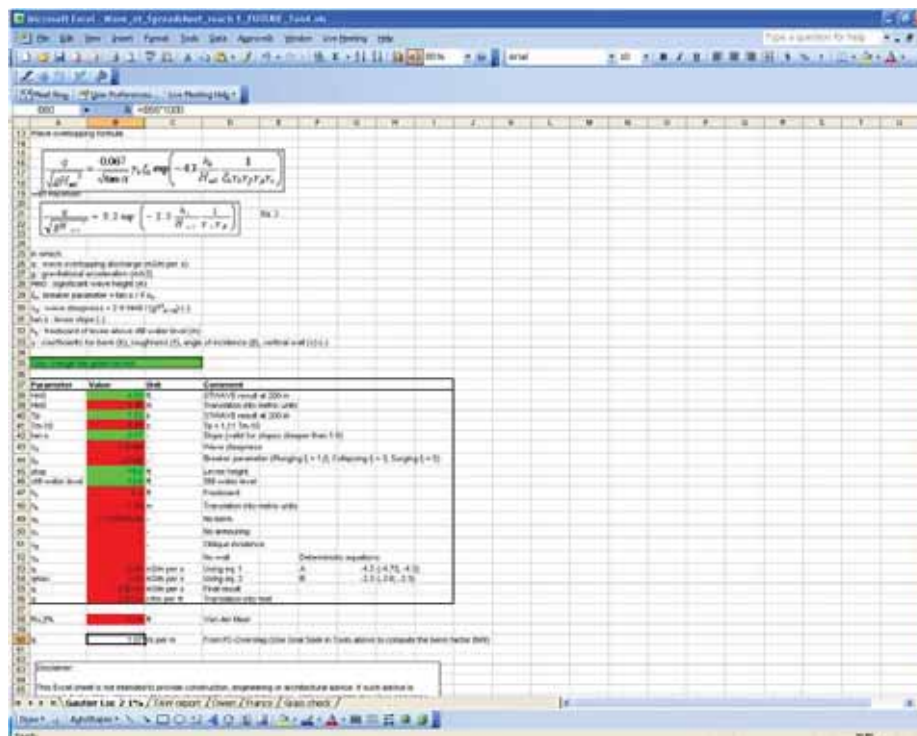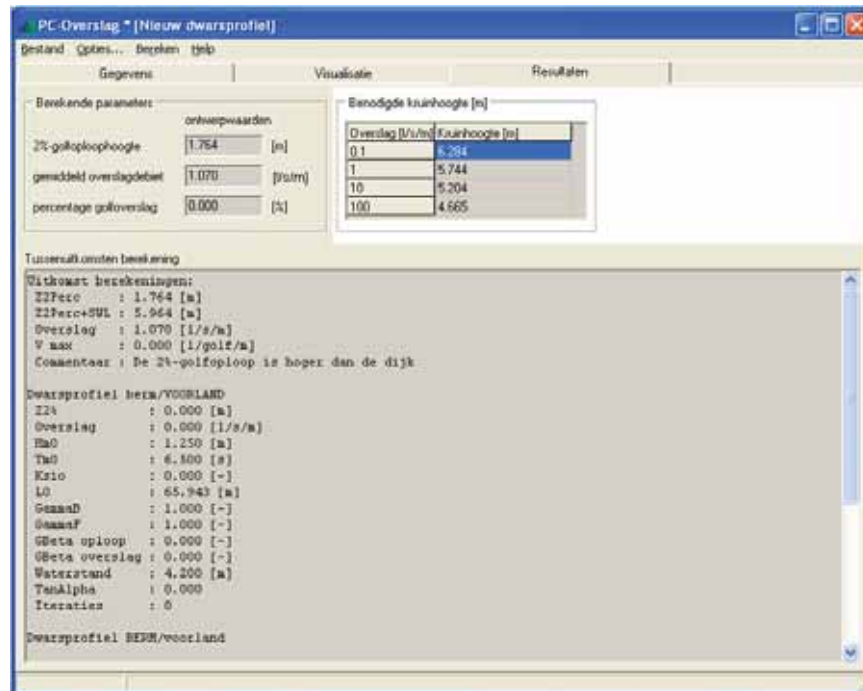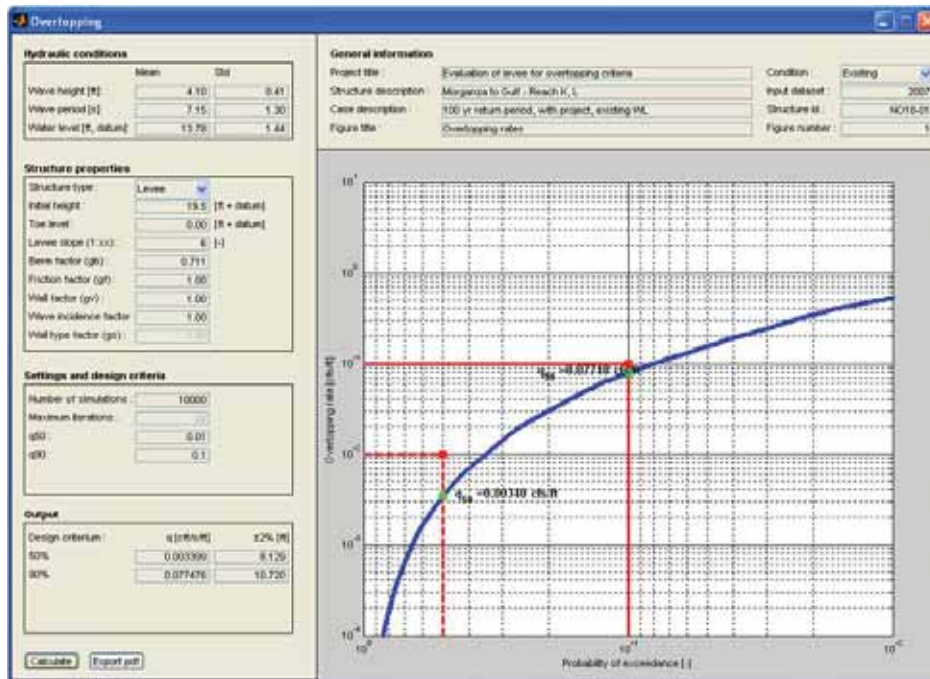
Figure 27 - Example of the Matlab overtopping script used to compute levee design elevation

2.6.1.1   Authorized Alignment

2.6.1.1.1 Existing Conditions

In addition to existing condition 1% chance exceedance designs, 1.33% and 2.0% (75 and 50 year, respectively) designs were also completed at request of the project management team. Existing condition design values for each frequency are provided in Table 33, Table 34, and Table 35. The values in the tables were taken directly from ERDC's statistical computations. The standard deviations for Hs and Tm are 10% and 20% of their respective values. In addition to the stepwise procedure given above, more detailed levee design methodology is provided in the HSDRRS Design Elevation Report (U.S. Army Corps, 2010). The final existing condition designs for all percent chance exceedance events are given in Table 36, Table 37, and Table 38.

Table 33 - 1% AEP Existing Condition frequency design characteristics

1% Exceedance	Surge J-Point	Wave J-Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Period (s)	
Reach			mean	std		mean	std	mean	std
K, L	174	103	14.94	1.52	16.9	4.5	0.45	6.7	1.34
H3, I1, I2, I3, J2, J1, J3	96	96	15.58	1.36	17.3	6.4	0.64	7.1	1.42
H2	45	31	14.6	1.17	16.1	6.2	0.62	7.9	1.58
G1, G2, G3, H1	36	28	14.28	1.13	15.7	5.9	0.59	8.1	1.62
F2, F1	41	41	13.78	1.11	15.2	6.1	0.61	7.2	1.44
E2, E1	83	83	14.63	1.29	16.3	4.8	0.48	6	1.20
B	90	90	11.4	0.99	12.7	3.2	0.32	7.1	1.42
B2	116	129	10.12	1.03	11.4	2.6	0.26	4.5	0.90
A-North of GIWW	183	183	8.04	0.99	9.3	2.5	0.25	5	1.00
A-South of GIWW	180	140	9.26	1.09	10.7	3.2	0.32	5.7	1.14

Table 34 - 1.33% Existing Condition frequency design characteristics

1.33% Exceedance	Surge J-Point	Wave J-Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Period (s)	
Reach			mean	std		mean	std	mean	std
K, L	174	103	13.78	1.44	15.6	4.1	0.41	6.5	1.30
H3, I1, I2, I3, J2, J1, J3	96	96	14.52	1.28	16.2	5.8	0.58	7.1	1.42
H2	45	31	13.63	1.11	15.1	5.8	0.58	7.8	1.56
G1, G2, G3, H1	36	28	13.3	1.07	14.7	5.4	0.54	8	1.60
F2, F1	41	41	12.85	1.05	14.2	5.6	0.56	7.2	1.44
E2, E1	83	83	13.55	1.22	15.1	4.3	0.43	5.8	1.16
B	90	90	10.49	0.93	11.7	2.7	0.27	7.1	1.42
B2	116	129	9.14	0.97	10.4	2.2	0.22	4.2	0.84
A-North of GIWW	183	183	7.22	0.94	8.4	2.1	0.21	4.8	0.96
A-South of GIWW	180	140	8.33	1.03	9.6	2.8	0.28	5.5	1.10

Table 35 - 2% Existing Condition frequency design characteristics

2% Exceedance	Surge J-Point	Wave J-Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Period (s)	
Reach			mean	std		mean	std	mean	std
K, L	174	103	12.1	1.32	13.8	3.5	0.35	6.3	1.26
H3, I1, I2, I3, J2, J1, J3	96	96	12.94	1.18	14.5	5	0.50	7	1.40
H2	45	31	12.18	1.02	13.5	5	0.50	7.8	1.56
G1, G2, G3, H1	36	28	11.82	0.98	13.1	4.6	0.46	7.9	1.58
F2, F1	41	41	11.42	0.97	12.7	4.7	0.47	7	1.40
E2, E1	83	83	11.92	1.12	13.4	3.7	0.37	5.5	1.10
B	90	90	9.05	0.86	10.2	2	0.20	7	1.40
B2	116	129	7.62	0.89	8.8	1.5	0.15	3.8	0.76
A-North of GIWW	183	183	5.99	0.86	7.1	1.6	0.16	4.5	0.90
A-South of GIWW	180	140	6.9	0.94	8.1	2.1	0.21	5.2	1.04

Table 36 - MTG 1% Existing Condition designs

1% Existing Condition Levee Designs	Top of Levee	Levee Slope	Top of Berm	Berm Slope	Toe of Berm	Levee Slope	0.2% Exceedance Design Elevation
K, L	26.5	1:4					20.7
	21.5	1:8					
	21.5	1:6	15	1:15	8	1:6	
H3, I1, I2, I3, J2, J1, J3	30.5	1:4					20.7
	23.5	1:8					
	23.0	1:6	16	1:15	11.5	1:6	
H2	30.0	1:4					19
	23.0	1:8					
	22.5	1:6	15	1:15	8.5	1:6	
G1, G2, G3, H1	29.0	1:4					18.5
	23.0	1:8					
	22.0	1:6	14.5	1:15	8.5	1:6	
F2, F1	27.5	1:4					17.9
	21.5	1:8					
	21.5	1:6	14	1:15	8	1:6	
E2, E1	26.0	1:4					19.5
	20.5	1:8					
	21.0	1:6	15	1:15	10	1:6	
B	19.5	1:4					15.1
	17.0	1:8					
	16.0	1:6	11.5	1:15	8.5	1:6	
B2	15.5	1:4					14.0
	14.0	1:8					
	14.0	1:6	10.5	1:15	7.5	1:6	
A-North of GIWW	13.5	1:4					11.8
	13.0	1:5					
	12.5	1:6					
	12.0	1:8					
	12.0	1:6	8	1:15	4.5	1:6	
A-South of GIWW	17.0	1:4					13.3
	14.0	1:8					
	13.5	1:6	9.5	1:15	6	1:6	

Table 37 - MTG 1.33% Existing Condition designs

1.33% Existing Condition Levee Designs	Top of Levee	Levee Slope	Top of Berm	Berm Slope	Toe of Berm	Levee Slope	0.2% Exceedence Surge Elevation
K, L	20	1:8					n/a
	19.5	1:6	14	1:15	10	1:6	
H3, I1, I2, I3, J2, J1, J3	22	1:8					n/a
	22	1:6	14.5	1:15	9	1:6	
H2	21.5	1:8					n/a
	21	1:6	14	1:15	8	1:6	
G1, G2, G3, H1	21.5	1:8					n/a
	20	1:6	13.5	1:15	8	1:6	
F2, F1	20	1:8					n/a
	20	1:6	13	1:15	7.5	1:6	
E2, E1	19	1:8					n/a
	19	1:6	14	1:15	9.5	1:6	
B	15.5	1:8					n/a
	14.5	1:6	10.5	1:15	8	1:6	
B2	12	1:8					n/a
	12	1:6	9.5	1:15	7	1:6	
A-North of GIWW	10.5	1:8					n/a
	10	1:6	7.5	1:15	5.5	1:6	
A-South of GIWW	12.5	1:8					n/a
	12	1:6	8.5	1:15	5.5	1:6	

Table 38- MTG 2% Existing Condition designs

2% Existing Condition Levee Designs	Top of Levee	Levee Slope	Top of Berm	Berm Slope	Toe of Berm	Levee Slope	0.2% Exceedence Surge Elevation
K, L	17.5	1:8					
	17	1:6	12.5	1:15	9	1:6	n/a
H3, I1, I2, I3, J2, J1, J3	20	1:8					
	19.5	1:6	13	1:15	8	1:6	n/a
H2	19.5	1:8					
	18.5	1:6	12.5	1:15	7.5	1:6	n/a
G1, G2, G3, H1	19	1:8					
	17.5	1:6	12	1:15	7.5	1:6	n/a
F2, F1	17.5	1:8					
	17.5	1:6	11.5	1:15	7	1:6	n/a
E2, E1	16.5	1:8					
	16.5	1:6	12	1:15	8.5	1:6	n/a
B	13	1:8					
	12.5	1:6	9.5	1:15	7.5	1:6	n/a
B2	10	1:8					
	10	1:6	8	1:15	6.5	1:6	n/a
A-North of GIWW	8.5	1:8					
	8	1:6	6	1:15	4.5	1:6	n/a
A-South of GIWW	10	1:8					
	10	1:6	7	1:15	5	1:6	n/a

2.6.1.1.1.1 Base and Future Conditions

As with existing conditions, 1%, 1.33%, and 2.0% designs were generated for base and future conditions. Future conditions designs made use of all with-project model simulations. SLR rates for base and future conditions ranged from 0.57 feet to 4.5 feet. Based on the timeline of design/ calculations, the future condition designs were done before the 5 feet SLR runs were considered. Therefore, the analysis made use of existing condition runs, 1.15 feet SLR runs, as well as the work done as part of the 3.2 feet SLR sensitivity analysis. None of the model runs were specifically done for any of the future condition SLR rates. Linear interpolation was considered the best method to compute approximate values for each specific SLR case. All base and future condition designs were to be based on the same surge and wave points. Thus, the limiting factor here was the lack of data available at the 3.2 ft. SLR runs. Surge was only available at points 36, 45, 80, 83, 90, 96, 174, and 208. Wave points included 31, 36, 41, 90, 96, 103, 140, and 208. Through mapping, pairs of surge and wave points were established for each reach (Table 39). Refer to ERDC's report on the 3.2ft SLR sensitivity analysis for more information.

Table 39 - MTG Base (2035) and Future (2085) Design Points

Reach	Surge Pt.	Wave Pt.
K, L	174	103
H3, I1, I2, I3, J2, J1, J3	96	96
H2	45	31
G1, G2, G3, H1	36	36
E2, E1, F2, F1	83	90
B	90	90
B2	90	90
A South of GIWW	90	90
A North of GIWW	208	208

Table 40 through Table 43 provide the SWE, SWE Standard Deviation, Hs, and Tm, respectively, for all model runs and corresponding interpolated values for base and future condition SLR rates. Linear extrapolation of SLR data produced surge and wave characteristics at the 4.75 ft SLR rate. The same design procedure followed for existing condition designs was done for future conditions. A constant berm slope of 1 on 15 was used for all designs since this slope was most efficient in the existing condition design analysis. A larger number of designs had to be completed for base and future conditions due to all the SLR scenarios. Thus, a constant berm factor of 0.75 was approximated for all design cases. Design maps for base and future conditions are given in Figure 28 and Figure 29.

Table 40 - Base and Future Condition 1%, 1.33%, and 2% SWE Values for Various SLR rates

REACH	Point	SWE 100 year												SWE 75 year												SWE 50 year											
		0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75	0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75	0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75
Reach K, L	174	14.9	15.9	16.1	16.8	16.8	16.8	17.1	17.2	17.7	18.0	18.3	19.4	13.8	14.7	15.0	15.6	15.7	15.7	15.9	16.0	16.4	16.6	16.8	17.7	12.1	13.0	13.3	14.0	14.0	14.0	14.2	14.3	14.8	15.0	15.3	16.3
Reach H, I1, I2, I3, J1, J3	96	15.6	16.2	16.3	16.7	16.7	16.8	17.0	17.2	17.8	18.0	18.4	19.7	14.5	15.1	15.2	15.6	15.7	15.7	15.9	16.1	16.6	16.8	17.1	18.2	12.9	13.5	13.7	14.1	14.1	14.2	14.5	14.6	15.3	15.6	16.0	17.4
Reach H3	45	14.6	15.2	15.3	15.7	15.8	15.8	16.1	16.3	17.0	17.4	17.8	19.3	13.6	14.2	14.3	14.7	14.8	14.8	15.1	15.2	15.8	16.1	16.5	17.8	12.2	12.7	12.9	13.3	13.3	13.3	13.7	13.8	14.5	14.9	15.3	16.8
Reach G1, G2, G3, H1	36	14.3	14.7	14.8	15.1	15.2	15.2	15.5	15.6	16.2	16.5	16.9	18.2	13.3	13.8	13.9	14.2	14.2	14.2	14.4	14.5	15.0	15.1	15.4	16.3	11.8	12.3	12.4	12.8	12.8	12.8	13.0	13.2	13.7	13.9	14.2	15.3
Reach E7, E1, F, F1	83	14.6	15.1	15.2	15.5	15.5	15.6	15.8	16.0	16.6	16.8	17.2	18.5	13.6	14.0	14.2	14.5	14.5	14.6	14.7	14.8	15.2	15.4	15.6	16.4	11.9	12.5	12.6	13.0	13.0	13.0	13.2	13.3	13.7	13.9	14.2	15.1
Reach B / E / mouth of GIWW	90	11.4	12.2	12.4	12.9	13.0	13.0	13.3	13.5	14.2	14.6	15.0	16.6	10.5	11.3	11.5	12.0	12.0	12.1	12.3	12.5	13.1	13.4	13.8	15.1	9.1	9.8	10.0	10.5	10.6	10.6	10.9	11.1	11.8	12.1	12.5	14.0
Reach A North of GIWW	208	8.0	9.9	10.4	11.8	11.9	11.9	12.4	12.6	13.7	14.2	14.8	17.0	7.2	9.0	9.5	10.8	10.9	10.9	11.3	11.6	12.5	12.9	13.5	15.5	5.9	7.6	8.0	9.2	9.3	9.4	9.8	10.1	11.1	11.6	12.2	14.4

Table 41 - Base and Future Condition 1%, 1.33%, and 2% SWE Standard Deviation Values for Various SLR rates.

STANDARD DEVIATION	Point	SWE StDev 100 year												SWE StDev 75 year												SWE StDev 50 year											
		0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75	0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75	0	0.57	0.72	1.12	1.15	1.18	1.51	1.7	2.42	2.75	3.2	4.75
Reach K, L	174	1.52	1.52	1.52	1.51	1.51	1.51	1.54	1.55	1.60	1.62	1.65	1.75	1.44	1.43	1.43	1.43	1.43	1.43	1.45	1.46	1.49	1.51	1.53	1.61	1.32	1.32	1.32	1.31	1.31	1.31	1.33	1.34	1.39	1.41	1.43	1.52
Reach H, I1, I2, I3, J1, J3	96	1.36	1.35	1.35	1.34	1.34	1.35	1.37	1.38	1.43	1.45	1.48	1.58	1.28	1.28	1.27	1.27	1.27	1.27	1.29	1.30	1.34	1.36	1.38	1.47	1.18	1.17	1.17	1.17	1.17	1.17	1.19	1.21	1.25	1.29	1.32	1.44
Reach H3	45	1.17	1.18	1.18	1.19	1.19	1.19	1.22	1.23	1.28	1.31	1.34	1.46	1.11	1.11	1.12	1.12	1.12	1.13	1.15	1.16	1.20	1.23	1.25	1.35	1.02	1.02	1.02	1.03	1.03	1.03	1.06	1.07	1.13	1.15	1.19	1.30
Reach G1, G2, G3, H1	36	1.13	1.11	1.10	1.09	1.09	1.09	1.11	1.12	1.17	1.19	1.21	1.31	1.07	1.05	1.04	1.03	1.03	1.03	1.04	1.05	1.08	1.10	1.11	1.18	0.98	0.96	0.96	0.95	0.94	0.95	0.96	0.97	1.01	1.03	1.05	1.13
Reach E7, E1, F, F1	83	1.29	1.24	1.23	1.19	1.19	1.19	1.21	1.22	1.26	1.28	1.31	1.41	1.22	1.17	1.16	1.12	1.12	1.12	1.13	1.14	1.17	1.18	1.20	1.26	1.12	1.07	1.06	1.03	1.03	1.03	1.04	1.05	1.09	1.10	1.12	1.19
Reach B	90	0.99	0.99	1.00	1.00	1.00	1.00	1.03	1.04	1.10	1.13	1.16	1.28	0.93	0.94	0.94	0.95	0.95	0.95	0.97	0.98	1.03	1.06	1.09	1.19	0.86	0.86	0.86	0.87	0.87	0.87	0.90	0.91	0.97	0.99	1.03	1.15
Reach A north of GIWW	208	1.04	1.16	1.19	1.28	1.28	1.29	1.34	1.37	1.48	1.53	1.60	1.84	0.99	1.10	1.13	1.21	1.21	1.22	1.27	1.29	1.40	1.44	1.51	1.73	0.90	1.01	1.03	1.11	1.11	1.12	1.17	1.20	1.33	1.38	1.46	1.71

Table 42 - Base and Future Condition 1%, 1.33%, and 2% Wave Height Values for Various SLR rates

WAVE HEIGHT	Point	Hs 100 year												Hs 75 year												Hs 50 year											
		0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75	0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75	0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75
Reach H2	31	6.2	6.5	6.6	6.9	6.9	6.9	7.1	7.2	7.6	7.8	8.0	8.8	5.8	6.1	6.2	6.4	6.4	6.4	6.6	6.7	7.1	7.3	7.5	8.3	5.0	5.3	5.4	5.7	5.7	5.7	5.9	6.0	6.5	6.7	7.0	8.0
Reach H3, I1, I2, I3, J2, J1, J3	96	6.4	6.8	6.9	7.2	7.2	7.2	7.4	7.5	7.9	8.1	8.4	9.3	5.8	6.2	6.4	6.7	6.7	6.7	6.9	7.0	7.4	7.5	7.8	8.5	5.0	5.4	5.6	5.9	5.9	5.9	6.1	6.2	6.7	6.9	7.2	8.2
Reach G1, G2, G3, H1	36	6.1	6.4	6.5	6.7	6.8	6.8	6.9	7.0	7.4	7.6	7.9	8.7	5.6	5.9	6.0	6.3	6.3	6.3	6.4	6.5	6.8	6.9	7.1	7.6	4.9	5.2	5.3	5.5	5.5	5.5	5.6	5.7	5.9	6.0	6.2	6.6
Reach K, L	103	4.5	4.8	4.9	5.1	5.1	5.1	5.4	5.5	6.0	6.3	6.6	7.7	4.1	4.4	4.5	4.7	4.7	4.7	4.9	5.0	5.5	5.7	6.0	7.0	3.5	3.8	3.9	4.2	4.2	4.2	4.5	4.9	4.9	5.1	5.4	6.3
Reach B/B2/ E2, E1, F2, F1/ A South of GIWW	90	3.2	3.5	3.6	3.9	3.9	3.9	4.1	4.2	4.6	4.8	5.0	5.8	2.7	3.0	3.1	3.4	3.4	3.4	3.5	3.6	3.9	4.0	4.2	4.7	2.0	2.3	2.4	2.7	2.7	2.7	2.8	2.8	3.0	3.1	3.3	3.7
Reach A North of GIWW	208	2.0	2.8	3.0	3.6	3.6	3.6	3.8	4.0	4.5	4.7	5.0	6.1	1.7	2.4	2.6	3.2	3.2	3.2	3.5	3.6	4.2	4.4	4.8	5.9	1.2	1.9	2.1	2.6	2.6	2.6	2.9	3.1	3.8	4.1	4.5	5.9

Table 43 - Base and Future Condition 1%, 1.33%, and 2% Mean Wave Period Values for Various SLR rates

WAVE PERIOD	Point	Tm 100 year												Tm 75 year												Tm 50 year											
		0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75	0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75	0.00	0.57	0.72	1.12	1.15	1.18	1.51	1.70	2.42	2.75	3.20	4.75
Reach H2	31	7.9	7.9	8.0	8.0	8.0	8.0	8.1	8.2	8.4	8.5	8.6	9.1	7.8	7.9	7.9	8.0	8.0	8.0	8.1	8.2	8.4	8.5	8.7	9.2	7.8	7.9	7.9	8.0	8.0	8.0	8.2	8.2	8.6	8.7	8.9	9.6
Reach H3, I1, I2, I3, J2, J1, J3	96	7.1	7.2	7.2	7.3	7.3	7.3	7.4	7.5	7.7	7.8	7.9	8.4	7.1	7.2	7.2	7.3	7.3	7.3	7.4	7.5	7.7	7.8	7.9	8.4	7.0	7.1	7.2	7.3	7.3	7.3	7.5	7.5	7.8	7.9	8.1	8.7
Reach G1, G2, G3, H1	36	7.1	7.2	7.3	7.4	7.4	7.4	7.5	7.6	7.8	7.9	8.0	8.5	7.0	7.2	7.3	7.4	7.4	7.4	7.5	7.6	7.8	7.9	8.1	8.5	6.9	7.1	7.2	7.4	7.4	7.4	7.5	7.5	7.6	7.7	7.8	8.1
Reach K, L	103	6.7	6.8	6.9	7.0	7.0	7.0	7.2	7.3	7.6	7.8	8.0	8.8	6.5	6.7	6.8	6.9	6.9	6.9	7.1	7.1	7.5	7.6	7.8	8.5	6.3	6.5	6.6	6.8	6.8	6.8	7.0	7.0	7.4	7.5	7.7	8.4
Reach B/B2/ E2, E1, F2, F1/ A South of GIWW	90	7.1	7.1	7.0	7.0	7.0	7.0	7.2	7.2	7.6	7.7	7.9	8.6	7.1	7.1	7.0	7.0	7.0	7.0	7.1	7.2	7.4	7.5	7.7	8.2	7.1	7.1	7.0	7.0	7.0	7.0	7.1	7.2	7.4	7.5	7.7	8.2
Reach A North of GIWW	208	4.6	5.0	5.2	5.5	5.5	5.5	5.7	5.8	6.1	6.3	6.5	7.3	4.4	4.8	5.0	5.3	5.3	5.3	5.5	5.7	6.2	6.4	6.7	7.8	4.1	4.6	4.7	5.1	5.1	5.1	5.4	5.5	6.1	6.3	6.7	7.9

Figure 28 - Base Condition (2035) Levee Design Elevation Map

Figure 29 - Future Condition (2085) Levee Design Elevation Map
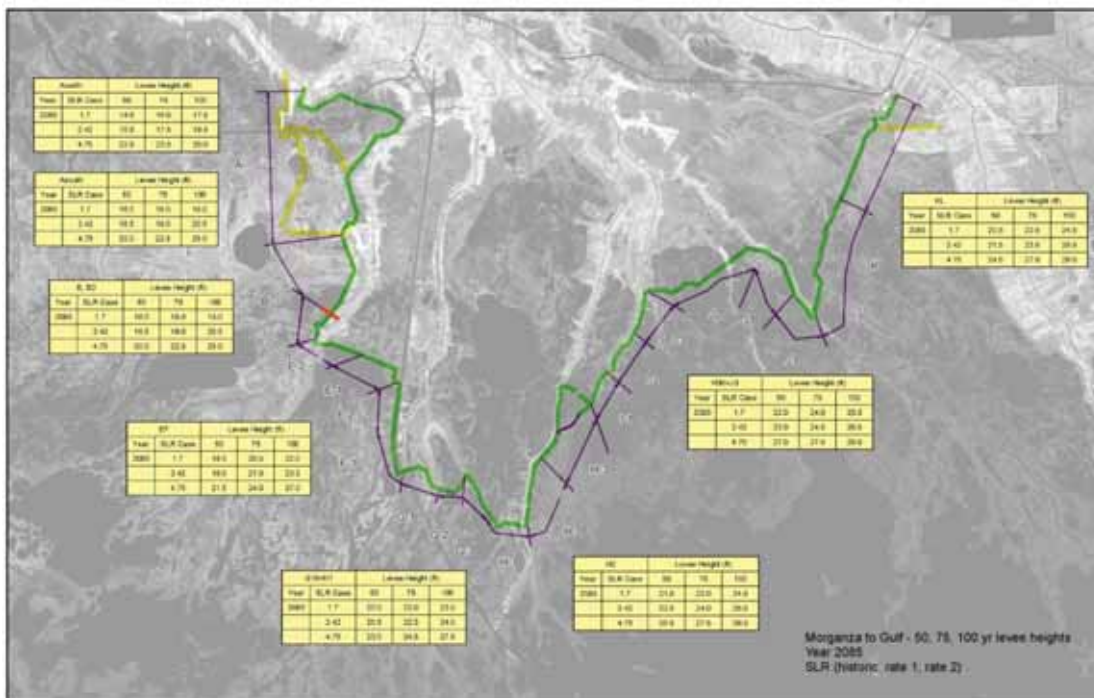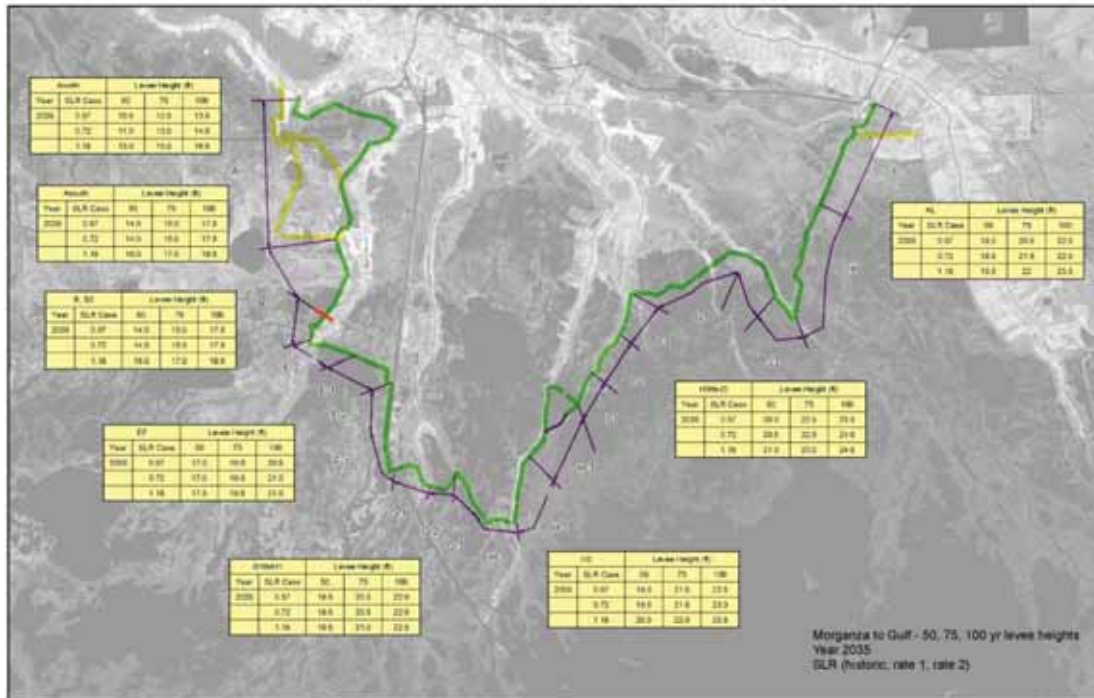
2.6.1.1.2 Barrier and Northern Alignment

In addition to designs done for the authorized alignment levee designs were completed for barrier and northern alignments. Approximate barrier and northern alignments are shown below in Figure 30. The barrier alignment was the western section of the authorized alignment stemming from the end of Reach A while the northern alignment stems from a high point in the ridge just above the end of Reach A. These alignments would also have to be broken into hydraulic reaches before doing levee design calculations. The barrier alignment was considered one continuous hydraulic reach while the northern alignment was separated into two hydraulic reaches. Only future condition designs were completed for the barrier and northern alignments.
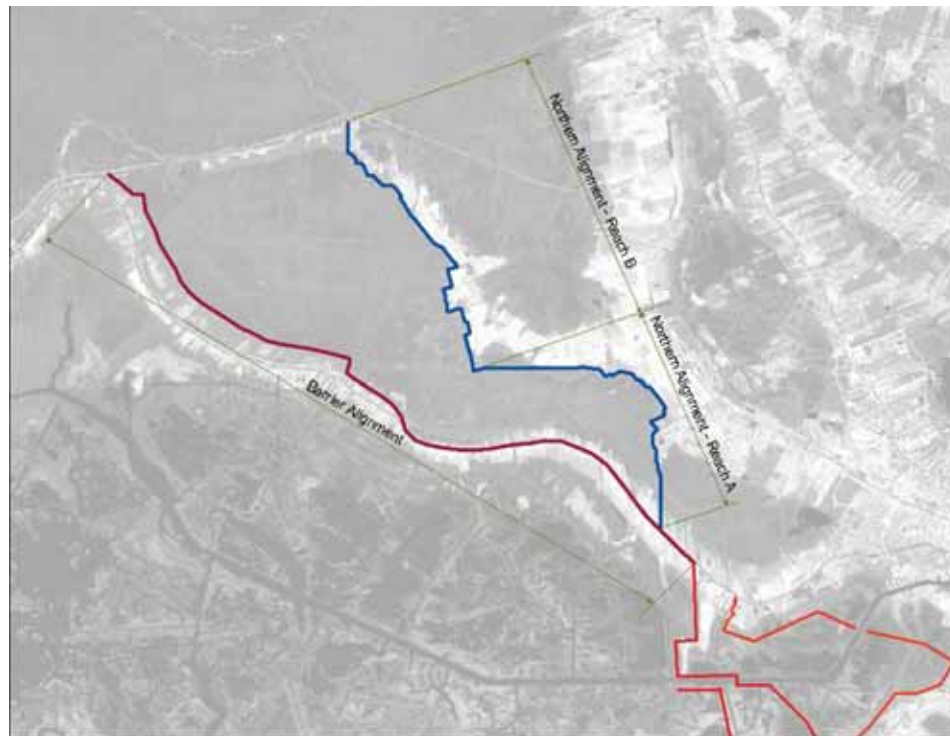
Figure 30 - Barrier and Northern Alignments. Bottom –right corner of figure depicts Reach A-North connection to Barrier Alignment

2.6.1.1.2.1   **Future Conditions**

Future condition levee designs were completed for barrier and northern alignments. Barrier alignment design values were interpolated and just as the authorized alignment future condition design values. Design methodology was similar to the other designs. The northern alignment future condition values were calculated differently. This alignment was not modeled in the main mesh due to the barrier alignment always being included in with-project conditions. Computations were made to approximate with project conditions in front of the northern alignment reaches. With and without project conditions were examined in front of the barrier alignment and average differences were taken for all design parameters (i.e., SWE, Hs, and Tm). These average differences

were applied to the without project northern alignment values to determine approximate with project conditions. The resulting values for with project conditions are given in Table 44. From this point, the designs were completed as normally done. The computed design elevations are provided in Table 45. Northern alignment surge and wave point bins contain acronym for not applicable, N/A, since multiple points in front of the northern alignment reaches were used to arrive at the final values. One of the resiliency criteria used in levee design is comparing the final levee design elevation to the 0.2% AEP value to make sure the design elevation is higher. Asterisks near the northern alignment design elevations indicate a need to raise the levee elevation above the 0.2% exceedance surge elevation.

Table 44 - Design parameters for future condition (2085) 1% levee designs. Both barrier and northern alignment data are given in the table

Reach	Surge J-Point	Wave J-Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Period (s)	
			mean	std		mean	std	mean	std
Barrier Alignment	225	225	14.08	1.4	15.9	4.9	0.49	6.2	1.24
Northern Alignment - Reach A	N/A	N/A	11.2	1.74	13.4	2.1	0.21	5.0	1.00
Northern Alignment - Reach B	N/A	N/A	11.31	1.52	13.3	1.9	0.19	6.3	1.26

Table 45 - Design elevations for future condition (2085) 1% levee designs. Both barrier and northern alignment data are given in the table

Reach	Top of Levee (ft)	Levee Slope	Top of Berm (ft)	Berm Slope	Toe of Berm (ft)	Levee Slope	0.2 % Exceedence Surge Elevation (ft)
Barrier Alignment	20	1 on 6	14.1	1 on 15	9.2	1 on 6	19.3
Northern Alignment - A	18* (16)	1 on 4	11.2	1 on 10	9.1	1 on 4	17.8
Northern Alignment - B	17* (16)	1 on 4	11.3	1 on 15	9.4	1 on 4	16.7

2.6.1.1.3 Multiple Lines of Defense Alignment

The multiple lines of defense alignment, also known as MLODS, emerged as a possible alternative alignment early in the project. The alignment consists of 10 reaches including ring levees surrounding Theriot, LA and Dulac, LA (Figure 31).
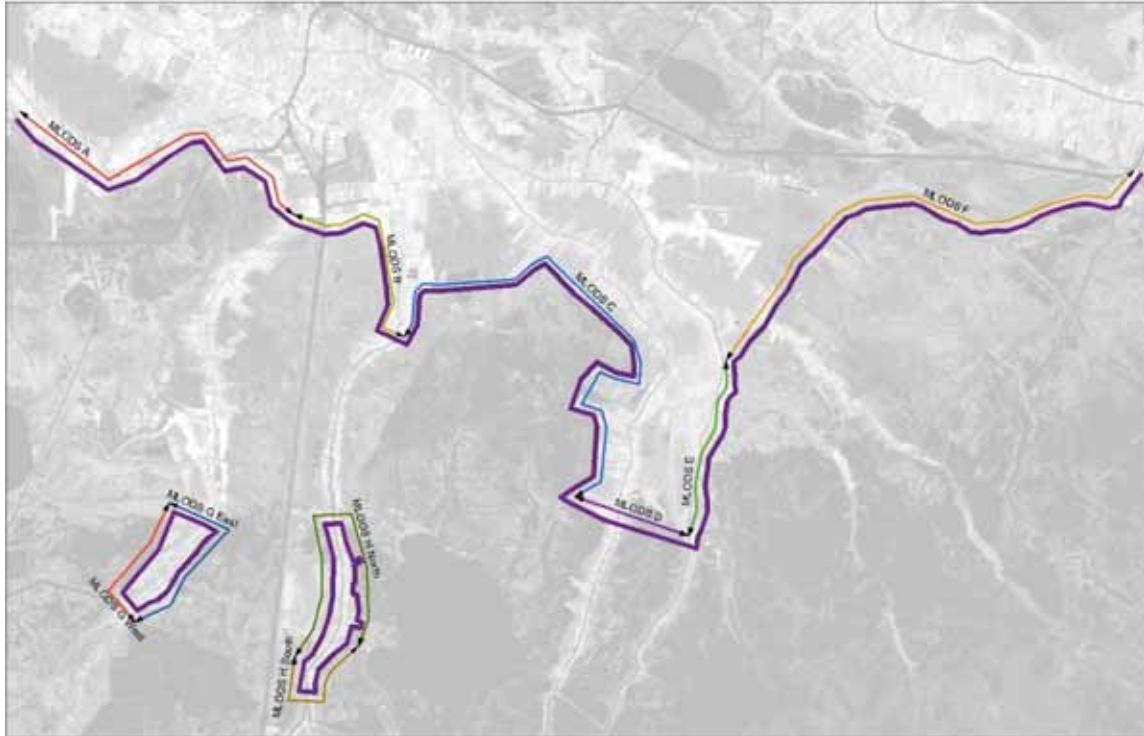
Figure 31 - MLODS alignment. Each of the 10 reaches is also given

2.6.1.1.3.1 Existing Conditions

Existing condition 1% designs were completed on the MLODS alignment. Table 46 gives all input values for each reach while Table 47 gives a summary of all design elevations.

Table 46 - Summary of 1% MLODS input values

Reach	Surge Point	Wave Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Mean Period (s)	
			mean	std		mean	std	mean	std
MLODS-A	189	208	9.8	1.18	11.3	2.2	0.22	4.8	0.96
MLODS-B	153	153	12.63	1.26	14.2	2.3	0.23	5.7	1.14
MLODS-C	148	145	14.16	1.38	15.9	3.6	0.36	5	1.00
MLODS-D	96	96	12.91	0.97	14.2	5.1	0.51	7	1.40
MLODS-E	117	106	13.26	1.09	14.7	4.8	0.48	6.8	1.36
MLODS-F	175	144	13.55	1.43	15.4	3.2	0.32	6.4	1.28
MLODS G-West	90	90	11.07	0.94	12.3	2.7	0.27	6.8	1.36
MLODS G-East	98	83	12.95	1.12	14.4	4.3	0.43	6.2	1.24
MLODS H-North	123	95	13.11	1.22	14.7	4	0.40	6.1	1.22
MLODS H-South	81	72	12.54	1.08	13.9	3.4	0.34	6	1.20

Table 47 - Summary of 1% design elevations established for MLODS alignment

Reach	Top of Levee (ft)	Levee Slope	Top of Berm (ft)	Berm Slope	Toe of Berm (ft)	Levee Slope	0.2 % Exceedence Surge Elevation (ft)
MLODS-A	13* (14.5)	1:6	10	1:15	8	1:6	14.21
MLODS-B	16* (17.5)	1:6	13	1:15	10.5	1:6	17.35
MLODS-C	19* (19.5)	1:6	14.5	1:15	11	1:6	19.33
MLODS-D	19.5	1:6	13	1:15	8	1:6	16.56
MLODS-E	19.5	1:6	13.5	1:15	8.5	1:6	17.35
MLODS-F	18* (19)	1:6	14	1:15	10.5	1:6	18.91
MLODS G-West	15	1:6	11.5	1:15	8.5	1:6	14.58
MLODS G-East	18.5	1:6	13	1:15	9	1:6	17.12
MLODS H-North	18.5	1:6	13.5	1:15	9.5	1:6	17.69
MLODS H-South	17	1:6	13	1:15	9.5	1:6	16.56

After comparing existing condition designs to that of the authorized alignment, the project management team decided to continue with the authorized alignment and forego MLODS.

2.6.1.2 Structure Designs

Structure designs were completed for a total of 27 structures throughout the authorized alignment. The structure types include locks, sector gates, flood gates, and sluice gates. A list of structures along with invert is provided below in Table 48. The acronyms GIWW and HNC refer to Gulf Intracoastal Waterway and Houma Navigation Canal, respectively. Structures are only looked at from a surge reduction perspective and not from a hydrological perspective.

Table 48 - List of structures for the authorized alignment including invert elevations

Structure	Invert (ft.)
Minors Canal Gate	-9
GIWW Sector Gate West	-12
Falgout Canal Sector Gate	-9
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach B	-4.5
Bayou Dularge Sector Gate	-7
Bayou Grand Caillou Sector Gate	-12
HNC Lock	-20
HNC Floodgate	-20
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach G2	-4.5
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E1	-4.5
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E2	-4.5
ECS- 1-6'x6' Box Culvert With Sluice Gate - Reach H1	-4.5
ECS- 6-6'x6' Box Culvert With Sluice Gate - Reach H1	-4.5
Lapeyrouse Sector Gate	-9
Placid Canal Sector Gate	-8
Bayou Petite Caillou Sector Gate	-8
Bush Canal Sector Gate	-12
Bayou Terrebonne Floodgate	-9
Humble Canal Sector Gate	-9
ECS- 5-5'x10' Box Culverts With Sluice Gates - Reach J2	-3.5
ECS- 4-5'x10' Box Culverts With Sluice Gates - Reach J2	-3.5
Point Au Chiene Sector Gate	-6
ECS- 2-6'x6' Box Culverts With Sluice Gates - Reach K	-4.5
Grand Bayou Sector Gate	-9
ECS- 6-5'x10' Box Culverts With Sluice Gates - Reach L	-4.5
Four Point Bayou Sector Gate	-6
GIWW Sector Gate East	-12

Additionally, some of the structure inverts were modified based on decisions made by the project management team. HNC structures were changed from -23 to -20 feet (-18 feet sill depth with two feet of overdredge) and GIWW structures were changed from -15 to -12 feet. For a given structure the SWE, Hs, Tm, and all other design information was needed similar to levee design. The same matlab script used for levee design was used here. The Franco and Franco overtopping formulations are used instead of the Van der Meer overtopping equation as in levee design (TAW, 2002). Choosing the option for floodwall design in the script the slope and berm factor input lines become inactive. All structures will have no slope since it is vertical and will not use a berm factor since it will be in the channel. From this point, the same type of 10,000 iteration Monte Carlo simulation is done to compute structural elevations. The q90 criterion is

the same as in levee design while the q50 criterion is modified to 0.03 cfs/ft per linear foot instead of the original levee threshold of 0.01 cfs/ft per linear foot as outlined in *Hurricane and Storm Damage Reduction System Design Guidelines (Interim), New Orleans District, Engineering Division, October 2007*.

2.6.1.2.1 Authorized Alignment Future Conditions

Authorized alignment 1% structure designs are given in Table 49. Two feet of structural superiority was added to the design elevation on all structures on the basis that it would be very difficult to rebuild, if damaged, because of disruption in services.

Table 49 - Results for 1% future condition (2085) Structure Designs

Structure	Lat	Lon	SWE (ft.)	StDev (ft.)	90% SWE (ft.)	Structure Elevation (ft.)	Structure Elevation (ft.) (2 ft. Structural Superiority)
Minors Canal Gate	29.55273	-90.79693	13.7	1.5	15.6	21.0	23.0
GIWW Sector Gate West	29.53519	-90.79261	13.7	1.5	15.6	21.0	23.0
Falgout Canal Sector Gate	29.41448	-90.78560	14.2	1.1	15.6	21.0	23.0
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach B	29.46103	-90.76387	14.2	1.1	15.6	21.0	23.0
Bayou Dularge Sector Gate	29.40828	-90.78688	16.6	1.3	18.2	23.5	25.5
Bayou Grand Caillou Sector Gate	29.34256	-90.73667	16.6	1.3	18.2	23.5	25.5
HNC Lock	29.33112	-90.73086	16.2	1.2	17.7	28.5	30.5
HNC Floodgate	29.33112	-90.73086	16.2	1.2	17.7	28.5	30.5
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach G2	29.31783	-90.69562	16.2	1.2	17.7	28.0	30.5
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E1	29.40731	-90.76983	16.6	1.3	18.2	23.5	25.5
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E2	29.39878	-90.74622	16.6	1.3	18.2	23.5	25.5
ECS- 1-6'x6' Box Culvert With Sluice Gate - Reach H1	29.30226	-90.67001	16.2	1.2	17.7	28.0	30.5
ECS- 6-6'x6' Box Culvert With Sluice Gate - Reach H1	29.29727	-90.65869	16.2	1.2	17.7	28.0	30.5
Lapeyrouse Sector Gate	29.30892	-90.64590	17.0	1.3	18.6	29.5	31.5
Placid Canal Sector Gate	29.33804	-90.63685	17.0	1.3	18.6	29.5	31.5
Bayou Petite Caillou Sector Gate	29.29672	-90.64844	16.2	1.2	17.7	28.5	30.5
Bush Canal Sector Gate	29.36816	-90.60206	17.8	1.4	19.6	31.0	33.0
Bayou Terrebonne Floodgate	29.38804	-90.58806	17.8	1.4	19.6	31.0	33.0
Humble Canal Sector Gate	29.43804	-90.56649	17.8	1.4	19.6	31.0	33.0
ECS- 5-5'x10' Box Culverts With Sluice Gates - Reach J2	29.43639	-90.53922	17.8	1.4	19.6	31.0	33.0
ECS- 4-5'x10' Box Culverts With Sluice Gates - Reach J2	29.44678	-90.51375	17.8	1.4	19.6	31.0	33.0
Point Au Chiene Sector Gate	29.41569	-90.44689	17.8	1.4	19.6	31.0	33.0
ECS- 2-6'x6' Box Culverts With Sluice Gates - Reach K	29.44617	-90.44638	17.7	1.6	19.8	27.5	29.5
ECS- 2-6'x6' Box Culverts With Sluice Gates - Reach K	29.46964	-90.43881	17.7	1.6	19.8	27.5	29.5
Grand Bayou Sector Gate	29.51833	-90.41173	17.7	1.6	19.8	27.5	29.5
ECS- 6-5'x10' Box Culverts With Sluice Gates - Reach L	29.50408	-90.41902	17.7	1.6	19.8	27.5	29.5
Four Point Bayou Sector Gate	--	--	16.2	1.2	17.7	28.0	30.0
Bayou Lafourche Gate (GIWW East)	29.58809	-90.37252	12.0	0.8	12.9	19.5	21.5

2.6.1.3   Wave Load Designs

Flood side wave loads were computed on each structure. Wave load calculations mainly determine pressure, hydrostatic force, and dynamic force on a floodwall. Similar to structure designs, wave loadings were only done for future conditions. Structural superiority was included in the total structure elevation for wave load computation. Loads were computed using a Matlab script similar to that used in the levee design effort. See Figure 32 for an example of the script user interface. A typical wave load computation uses many of the same inputs as in levee design along with some additional values. Inputs to the script will be given in the bullet list below.

- Top of Wall – Elevation of structure including structural superiority.
- Water level backside – Assumed zero unless otherwise known.
- Top of rock – Assumed equal to invert elevation of channel if no berm is present
- Top of fill – Equal to invert elevation of channel if no berm is present.
- Base Level – Invert elevation. Same as channel invert if sill is on channel bottom.
- Berm width – Width of berm if included. Zero if no berm is present.
- Cotangent of slope – Slope in front of structure. A large number indicates insignificant slope (e.g., 100).
- Structure code/description – Computation run time
- Still water level/ Wave height/ Wave period – Input corresponding to your frequency.
- Number of simulations – 10,000.
- Non-exceedance % - Compute only 90% confidence values.

The script uses all the input data to run a large number of Monte Carlo simulations (10,000) to determine 90% exceedance limit values. Wave load computations are based on the Goda formulations, published in EM 1110-2-1100, Part VI, Chapter 5, 1 June 2006). Figure 33 gives an example of a wave load computation.

Figure 32 - Example template for wave loadings. The invert elevation of the structure was used as the top of rock, fill, and base level elevations

BAYOU TERREBONE FLOODGATE 1% 2085

p_2 TOW = 33.00

SWL = 17.80 P_1

SWB = 0.00

Base El. = -9.00 P_3 TOF = -9.00

p_u

Structure dimensions :   Hydraulic conditions :

Top of wall	=	33.00 ft	Hs (mu, sigma)	=	7.90,	0.80 ft
Top of rock	=	-9.00 ft	Ts (mu, sigma)	=	7.70,	1.50 s
Top of fill	=	-9.00 ft	Water level (mu, sigma) =		17.80,	1.40 ft
Base elevation	=	-9.00 ft	Confidence band (%)	=	90	
Berm width	=	0.00 ft	No of simulations	=	10000	
Slope (1:value)	=	100.00 -				
Water level backside =		0.00 ft + datum				

Hydrostatic and wave forces on wall (90 % non-exceedance values) :

Wave type   =   irregular   breaking

p1	=	0.92	0.92	lb/sqft
p2	=	0.41	0.41	lb/sqft
p3	=	0.75	0.75	lb/sqft
pu	=	0.75	0.75	lb/sqft
F hydro	=	26.27	26.27	lb/ft
F back	=	-2.59	-2.59	lb/ft
F wave	=	32.85	32.85	lb/ft
M wave	=	661.10	661.10	ft-lb/ft
A wave	=	11.13	11.13	ft

*All levels are referenced to datum, forces and pressures are in thousands

Wave forces on sea wall according Goda formula for breaking and non breaking waves	BTFG-100	
	BTFG-100	
Bayou Terrebone Floodgate 1% 2085	Fig. 1.1	
Evaluation of sea wall designs	9S2231.01	ROYAL HASKONING

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Figure 33 - Example of a wave load result sheet

2.6.1.3.1 Authorized Alignment Future Conditions

A total of 27 wave loading cases were completed for future conditions. Section 5.2 provides a list of all structures with corresponding coordinates. All future condition design figures for 1% frequency can be furnished upon request.

2.6.2   3% AEP Storm Surge Risk Reduction System Designs

2.6.2.1   Determine Post-Katrina Level of Risk Reduction

The old 1% chance exceedance designs outlined in the 2002 Feasibility Report are shown in Table 50. These designs were done before the latest design criteria were established. In light of the new design criteria, the old 1% level of risk reduction was re-evaluated to determine the actual frequency. The 2002 designs were termed the 1% authorized designs. The process used to re-evaluate the authorized 1% authorized designs to determine the 3% AEP will now be given.

Table 50 - 1% Design characteristics and associated levee elevations from the 2002 MTG Feasibility Report

Reach	Stillwater Elevation (SWL)	Fetch		Significant Wave		Runup	Design Levee	
		Length	Depth	Height(H_s)	Period(T)		Elevation	Slope
	feet NGVD	feet	feet	feet	seconds	feet	feet NGVD	1:00
A	8 G_d ᴺᴳᵛᴰ	0.5	7	2.2	2.1	2	10	3.0
B	8	10	8	4.2	4.6	3	11	7.0
C	9	0.5	8	2.2	2.1	2	11	3.0
	9	10	9	4.6	4.7	3	12	7.5
	10	10	10	5.0	4.8	3	13	8.0
D	10	10	10	5.0	4.8	3	13	8.0
E	10	10	10	5.0	4.8	3	13	8.0
BRG	10	10	10	5.0	4.8	3	13	8.0
F	10	10	10	5.0	4.8	3	13	8.0
G	11	10	11	5.3	4.9	3	14	8.3
H	11	10	11	5.3	4.9	3	14	8.3
	12	10	12	5.6	4.9	3	15	8.5
I	11	10	11	5.3	4.9	3	14	8.3
	10	10	10	5.0	4.8	3	13	8.0
J	10	10	10	5.0	4.8	3	13	8.0
JSC(1)	10	10	10	5.0	4.8	3	13	8.0
JSC(2)	11	10	11	5.3	4.9	3	14	8.3
K	10	10	10	5.0	4.8	3	13	8.0
	9	10	9	4.6	4.7	3	12	7.5
	8	10	8	4.2	4.6	3	11	7.0
L	8	0.5	7	2.2	2.1	2	10	3.0
	7	0.5	6	2.1	2.1	2	9	3.0

Surge and wave characteristics associated with pre-Katrina 1% frequency designs given in the feasibility report needed to be updated to be comparable to work done post-Katrina. First, surge and wave characteristics from the report were associated with a new post-Katrina frequency. Analyses related to this conversion process can be

furnished upon request. From the multiple methods utilized, a new 2.86% chance exceedance frequency event was established as the 3% AEP corresponding to the old 1% frequency values in the 2002 feasibility report.

2.6.2.2 Levee Designs

2.6.2.2.1 Authorized Alignment

2.6.2.2.1.1 Base and Future Conditions

This section discusses base and future condition designs for the 2.86% frequency. The designs were based only on the intermediate SLR rate. The decision to only use the intermediate rate was made after learning from USACE MVN Economics Branch that only intermediate SLR results would go through the full economic cost-benefit analysis. Until now, only surge and wave characteristics for existing condition 2.86% frequency had been calculated. Base and future conditions also have to be established. Interpolation could not simply be performed to determine base and future condition 2.86% frequency surge and wave characteristics. A stair step method was used to develop base and future condition surge and wave characteristics. Existing condition 2.86% frequency data was used to develop base condition values then base condition 2.86% frequency data were used to develop future condition values. Since existing condition 10% data were already adjusted no further vertical or with-project adjustments would have to be made to base and future condition 10% values once they are known. All available data from Table 40 through Table 43 was used in this analysis. The methodology will now be examined in bullet format.

- Collect existing, base and future condition data available for 1%, 1.33% and 2% frequencies.

- Since the existing condition data had frequency curves developed to 10% the existing condition 2.86% SWE and SWE standard deviations were interpolated from the curve.

- The lower frequency statistics (i.e., 1%, 1.3%, and 2% values) were provided by ERDC and the higher frequency data was developed using a gage analysis as discussed in Section 2.5.3.2.

- Using the stair step method ratios of base to existing conditions at the 2% frequency were computed and assumed to be approximately equal to the ratio at the 2.86% frequency.

- With SWE and SWE standard deviation ratios calculated, the ratios were multiplied by the existing condition 2.86% frequency data to arrive at base condition 2.86% SWE and SWE standard deviation values over all reaches (Table 51).

- The same method was applied using the newly created base condition 2.86% SWE and SWE standard deviation values to compute 2.86% future condition SWE and SWE standard deviation values (Table 52).

- To calculate 2.86% Hs values, Hs to depth relationships were calculated across each reach. This is the same method used to calculate 2.86% values for existing conditions.

- The ratios were calculated at the 2% frequency and assumed to be the same at the 2.86% frequency. Depth of water was calculated using existing condition 2% Hs and SWE. Average bottom elevations were selected in front of each reach and depths were calculated by subtracting the existing condition 2% SWE.

- For reach E2, E1, F2, F1 wave characteristics were taken from point 90 which was also the same point used for reach B. However, the bottom elevation for reach E2, E1, F2, F1 was taken within the reach area instead of just using the elevation assumed for reach B.

- With known ratios a depth was now calculated using the existing condition 2.86% SWE for each reach.

- Each ratio was then multiplied by its corresponding depth to arrive at a 2.86% Hs value. In this manner all base condition 2.86% Hs values were computed.

- The same method was used to solve for future condition 2.86% frequency Hs values with base condition 2% frequency Hs values as a starting point for ratio calculation (Table 53).

The following equation was used to develop base, and future 2.86% Tm values.

$$Tm_2 = \sqrt{Hs_2 \big/ Hs_1} \times Tm_1 \qquad\qquad (2)$$

 where Hs is the significant wave height and Tm is the mean wave period. The subscript 1 corresponds to existing condition values when solving for base condition Tm and base condition when solving for future condition Tm.

Table **53** provides 2.86% surge and wave characteristics for existing, base, and future conditions.

Table 51 - Developed Future Condition (2085) 2.86% SWE and SWE Standard Deviation for each reach

SWE	Pt	SWE Base 35 yr	50 yr				SWE Future 35yr
			Base SWE	Future (2085) SWE	Ratio (fut$_{2085}$/base$_{2035}$)		
Reach K, L	174	11.75	13.3	14.8	1.11		13.09
Reach H3, I1, I2, I3, J2, J1, J3	96	12.48	13.7	15.3	1.12		13.94
Reach H2	45	11.84	12.9	14.5	1.13		13.37
Reach G1, G2, G3, H1	36	11.35	12.4	13.7	1.10		12.48
Reach E2, E1, F2, F1	83	11.41	12.6	13.7	1.09		12.45
Reach B	90	9.06	10.0	11.8	1.18		10.66
Reach A north of GIWW	208	7.09	8.0	11.1	1.38		9.79

SWE Standard Deviation	Pt	SWE StDEV Base 35yr	50 yr				SWE StDev Future 35yr
			Base StDev 2008	Future (2085) StDev	Ratio (fut$_{2085}$/base$_{2035}$)		
Reach K, L	174	1.18	1.32	1.39	1.05		1.25
Reach H3, I1, I2, I3, J2, J1, J3	96	1.11	1.17	1.26	1.08		1.20
Reach H2	45	0.97	1.02	1.13	1.10		1.07
Reach G1, G2, G3, H1	36	0.91	0.96	1.01	1.05		0.96
Reach E2, E1, F2, F1	83	1.00	1.06	1.09	1.02		1.03
Reach B	90	0.83	0.86	0.97	1.12		0.93
Reach A north of GIWW	208	0.93	1.03	1.33	1.28		1.19

Table 52 - Developed Existing, Base (2035), and Future Condition (2085) 2.86% Hs for each reach

Hs Existing	Pt	50 yr						35 yr	35 yr
		ERDC 2008 - Hs	ERDC 2008 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)		Hs - Existing	Hs - Base (2035)
Reach K, L	103	3.50	12.1	1.30	10.80	0.32		3.05	3.39
Reach H3, I1, I2, I3, J2, J1, J3	96	5.00	12.9	2.89	10.01	0.50		4.45	4.79
Reach H2	31	5.00	12.2	1.03	11.17	0.45		4.55	4.84
Reach G1, G2, G3, H1	36	4.90	11.8	1.94	9.86	0.50		4.40	4.68
Reach E2, E1, F2, F1	90	2.00	11.9	-0.98	12.88	0.16		1.83	1.92
Reach B	90	2.00	9.1	1.20	7.90	0.25		1.77	1.99
Reach A north of GIWW	208	1.20	5.9	1.05	4.85	0.25		1.03	1.50

Hs Existing	Pt	50 yr						35 yr
		2035 - Hs	2035 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)		Hs - 2085
Reach K, L	103	3.9	13.3	1.30	11.99	0.33		3.88
Reach H3, I1, I2, I3, J2, J1, J3	96	5.6	13.7	2.89	10.80	0.52		5.69
Reach H2	31	5.4	12.9	1.03	11.85	0.46		5.66
Reach G1, G2, G3, H1	36	5.3	12.4	1.94	10.48	0.50		5.31
Reach E2, E1, F2, F1	90	2.4	12.6	-0.98	13.59	0.18		2.41
Reach B	90	2.4	10.0	1.20	8.80	0.28		2.62
Reach A north of GIWW	208	2.1	8.0	1.05	7.00	0.30		2.60

Table 53 – Surge and Wave characteristics for 2.86% existing, base, and future condition designs

Reach	SWE Pt.	Wave Pt.	2008				2035				2085			
			SWE	SWE StDev	Hs	Tm	SWE	SWE StDev	Hs	Tm	SWE	SWE StDev	Hs	Tm
K,L	174	103	10.7	1.19	3.05	3.85	11.75	1.18	3.39	4.06	13.09	1.25	3.88	4.35
H3,I1,I2,I3,J2,J1,J3	98	96	11.8	1.12	4.45	4.86	12.48	1.11	4.79	4.83	13.94	1.20	5.89	5.27
H2	45	31	11.2	0.96	4.55	4.71	11.84	0.97	4.84	4.86	13.37	1.07	5.66	5.26
G1,G2,G3,H1	36	36	10.8	0.93	4.40	4.63	11.35	0.91	4.68	4.78	12.46	0.96	5.31	5.09
E2,E1,F2,F1	83	90	10.8	1.06	1.83	2.99	11.41	1.00	1.92	3.06	12.45	1.03	2.41	3.43
B	90	90	8.2	0.82	1.77	2.94	9.06	0.83	1.99	3.12	10.66	0.93	2.62	3.58
B2	90	90	8.2	0.82	1.77	2.94	9.06	0.83	1.99	3.12	10.66	0.93	2.62	3.58
A South	90	90	8.2	0.82	1.77	2.94	9.06	0.83	1.99	3.12	10.66	0.93	2.62	3.58
A North	208	208	5.2	0.81	1.03	2.24	7.09	0.93	1.50	2.70	9.79	1.19	2.80	3.56

2.6.2.2.2 Barrier Alignment

At this point in the design comparison process, the project management team decided it would be more beneficial to continue designing for only the barrier alignment. The northern alignment was considered less feasible as the project continued going forward. Hydraulic reaches for the barrier alignment were kept constant for both 1% AEP and 3% AEP designs. Designs were completed for a 2.86% frequency event.

2.6.2.2.2.1 Existing Conditions

Existing condition designs were not completed for the Barrier Alignment. This was a decision made by the project management team.

2.6.2.2.2.2 Future Conditions

There was one hydraulic reach representing the barrier alignment. The same MTG design point used in the 1% AEP alternative barrier design, point 225, was used here. Also, the same methodology used to compute 2.86% future condition values for authorized alignment was followed here to compute future conditions. Existing condition data were used to develop base condition values then base condition data were used to ultimately develop future condition design values. With lack of data, ratios were greatly used in the computations. Several with-project frequencies were used in this analysis. Of those frequencies, 10% data were needed. The 10% data values in front of the Barrier alignment needed adjusting for with project conditions as well as vertical datum. Before continuing any further the 10% frequency data were adjusted. A with-project correction was applied first by calculating differences between with and without project SWE values at various points around 225 for a 1%, 1.33%, and 2% events. The resulting differences were averaged at each frequency and plotted. A 10% frequency with project correction was extrapolated from the plot. An adjustment was applied next as the ground truthing correction. The ground truthing work peformed by Roy Dokka is discussed in detail within ERDC's final modeling report (Cialone et al., 2010). It was assumed the Dokka adjustment was approximately constant in the project

area regardless of frequency. This was validated through a check of multiple frequencies. The 10% values are now consistent with other "with project" frequency values (Table 54). All SWE, Hs, and Tm input data used to compute future condition design values are given in Table 55.

Final design values are provided towards the end of the step-wise process. In some cases comments are given in a figure cell to explain how a value was computed. Logarithmic interpolation was performed to compute an existing condition 2.86% SWE and standard deviation values for point 225. From this point the step wise procedure follows that developed for authorized alignment base and future condition computations. In the case of developing a future conditions Tm a constant wave steepness of 4% was used. This is also consistent with the value used to for authorized alignment base and future conditions. A levee design was then computed for the single barrier alignment reach Table 56.

Table 54 - Computations needed to develop 10% AEP with project frequency value

	Without Proj Value								
Pt 225	10yr								
Raw FEMA	3.1								

B2EDHLB3: With Project Adjustment: The raw 10yr FEMA data was for without project conditions. This adjustment determines the differences between WP and WOP to calculate an avg adjustment to the 10yr data.

	Returns								
	100yr			75yr			50yr		
	Pt 233	Pt 225	Pt 221	Pt 233	Pt 225	Pt 221	Pt 233	Pt 225	Pt 221
With Proj	7.35	7.9	7.88	6.59	7.11	7.04	5.47	5.92	5.8
Without Proj	7.37	7.69	7.73	6.66	7	6.97	5.59	5.96	5.86
Difference	-0.02	0.21	0.15	-0.07	0.11	0.07	-0.12	-0.04	-0.06
Average		0.11			0.04			-0.07	

		(WP-WOP) Adj		B2EDHLB3: Approximate 10yr value taken from extrapolation.
	10yr	Value		
	3.1	-0.5		
Final After WP Adjustment		3.6		

B2EDHLB3: This adjustment is to incorporate the ground truthing correction. It's done by taking the average difference (ERDC-FEMA) for 100yr without project values.

Return			
		100	
	Pt 233	Pt 225	Pt 221
Without Proj (FEMA)	7.4	8.2	7.8
Without Proj (ERDC)	7.37	7.69	7.73
Difference	-0.03	-0.51	-0.07
Average		-0.20	

		WOP (ERDC-FEMA)
		Adj Value
Final After WP Adjustment	10yr	
	3.1	-0.20
		2.9

*All values are in feet.

Table 55 - Computations for developing 3% AEP future condition design parameters

		ERDC Values - SWE - Existing Conditions (0 ft SLR)					SWE Existing		SWE StDEV Existing Conditions
SWE Existing - 2008	Pt	10	50	75	100		35 yr		35 yr
Barrier Alignment	225	2.90	5.92	7.11	7.9	>>>>>>	5.2		0.6

		50 yr				35 yr - 2035			
Calculate SWE Futures - 2035	Pt	ERDC 2008 (no slr)	Base 2035 (0.72 slr)	Ratio (base$_{2035}$/exis$_{2010}$)		SWE - Base (0.72 ft slr)			
Barrier Alignment	225	5.9	8.1	1.368	>>>>>>	7.11			

		50 yr				35 yr - 2035			
Calculate SWE StDev Futures - 2035	Pt	StDev 2008 (no slr)	StDev 2035 (0.72 slr)	Ratio (base$_{2035}$/exis$_{2010}$)		SWE StDev - Base (0.72 ft slr)			
Barrier Alignment	225	0.91	1.10	1.209	>>>>>>	0.73			

		50 yr						35 yr	35 yr
Hs Existing	Pt	ERDC 2008 - Hs	ERDC 2008 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)		Hs - Existing	Hs - Base (2035)
Barrier Alignment	225	1.40	5.9	1.05	4.85	0.29	>>>>>>	1.20	1.75

		35 yr							
Tm Existing/Futures - 2008/2035	Pt	Constant Wave Steepness	2008 - Tm	Base 2035 - Tm					
Barrier Alignment	225	0.04	2.4	2.9					

		50 yr				35 yr - 2085			
Calculate SWE Futures - 2085	Pt	2035 (0.72 slr)	2085 (2.42 slr)	Ratio (fut$_{2085}$/fut$_{2035}$)		SWE - Future (2.42 ft slr)			
Barrier Alignment	225	8.1	11.6	1.43	>>>>>>	10.19			

		50 yr				35 yr - 2085			
SWE StDev Futures - 2085	Pt	StDev 2035 (0.72 slr)	StDev 2085 (2.42 slr)	Ratio (fut$_{2085}$/fut$_{2035}$)		SWE StDev - Future (2.42 ft slr)			
Barrier Alignment	225	1.10	1.21	1.10	>>>>>>	0.80			

		50 yr						35 yr	
Hs Existing	Pt	2035 - Hs	2035 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)		Hs - 2085	
Barrier Alignment	225	2.3	8.1	1.05	7.05	0.33	>>>>>>	2.98	

		35 yr							
Tm Existing/Futures - 2085	Pt	2085 - Tm							
Barrier Alignment	225	3.8							

Table 56 - Final future condition (2085) 2.86% Levee Design for Barrier Alignment

2.86% Exceedence - Future (2085) With Project Hydraulic Conditions

1% Exceedance	Surge J-Point	Wave J-Point	Surge Elevation (ft)		90% Surge Elevation (ft)	Significant Wave Height (ft)		Period (s)	
Reach			mean	std		mean	std	mean	std
Barrier Alignment	225	225	10.19	0.8	11.2	2.98	0.45	3.8	0.30

Reach	Top of Levee (ft)	Levee Slope	Top of Berm (ft)	Berm Slope	Toe of Berm (ft)	Levee Slope	Minimum Hydraulic Requirement for Structure Elevations	0.2% Exceedence Surge Elevation
Barrier Alignment	13.0	1 on 6	10.2	1 on 12	7.2	1 on 6		N/A

- all elevations are in NAVD88 (2004.65)

2.6.2.3   Structure Designs

Approximately 27 structures were designed in addition to all levee designs. Structure types included mainly flood, sector, and sluice gates. All structures were located along the MTG authorized alignment. Structures are only looked at from a surge reduction perspective and not from a hydrological perspective. Table 48 provides the invert elevation of each structure. Structure inverts were provided by the project management team with input from the Terrebonne Parish Levee Board. Modifications were made to some of the original structure inverts. Structures were only designed based on the intermediate SLR rate for future conditions. The overall design procedure for a structure is very similar to that of a levee except for specific exceptions. The Franco and Franco overtopping formulations are used in structure design instead of the Van der Meer overtopping equation as in levee design (TAW, 2002). The 50% confidence overtopping limit is raised from 0.01 to 0.03 cfs per linear foot as outlined in *Hurricane and Storm Damage Reduction System Design Guidelines (Interim), New Orleans District, Engineering Division, October 2007*. Also, in most cases a structure is not built on a berm. Therefore, structure slopes are normally set to zero. With no berm the PC-Overslag step is removed from the process.

Structure designs use the same type of surge and wave information as needed for levee design. Surge and wave points were assumed to be the same for each reach as those used in levee design. All structures were first grouped according to the reach that it was located within. Every structure within a particular reach was given the same surge and wave value associated with that reach. Consequently, structures within a given reach will most likely have the same structure elevation. Two structures with the same surge and wave design inputs could possibly develop two different design elevations if channel depth for a structure limits wave height. If this becomes the case the resulting structural elevations should still be fairly similar.

2.6.2.3.1 Authorized Alignment Future Conditions

Authorized alignment 2.86% structure designs are given in Table 57. Two feet of structural superiority was added to the design elevation on all structures on the basis that it would be very difficult to rebuild, if damaged, because of disruption in services.

Table 57 - Future condition (2085) authorized alignment 2.86% structure designs

Structure	Lat	Lon	SWE (ft.)	StDev (ft.)	90% SWE (ft.)	Structure Elevation (ft.)	Structure Elevation (ft.) (2 ft. Structural Superiority)
Minors Canal Gate	29.55273	-90.79693	9.8	1.2	11.3	14.0	16.0
GIWW Sector Gate West	29.53519	-90.79261	9.8	1.2	11.3	14.0	16.0
Falgout Canal Sector Gate	29.41448	-90.78560	10.7	0.9	11.9	14.5	16.5
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach B	29.46103	-90.76387	10.7	0.9	11.9	14.0	16.5
Bayou Dularge Sector Gate	29.40828	-90.78688	12.5	1.0	13.8	16.0	18.0
Bayou Grand Caillou Sector Gate	29.34256	-90.73667	12.5	1.0	13.8	16.0	18.0
HNC Lock	29.33112	-90.73086	12.5	1.0	13.7	20.5	22.5
HNC Floodgate	29.33112	-90.73086	12.5	1.0	13.7	20.5	22.5
ECS- 6-6'x6' Box Culverts With Sluice Gates - Reach G2	29.31783	-90.69562	12.5	1.0	13.7	20.5	22.5
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E1	29.40731	-90.76983	12.5	1.0	13.8	16.0	18.0
ECS- 9-6'x6' Box Culverts With Sluice Gates - Reach E2	29.39878	-90.74622	12.5	1.0	13.8	16.0	18.0
ECS- 1-6'x6' Box Culvert With Sluice Gate - Reach H1	29.30226	-90.67001	12.5	1.0	13.7	20.5	22.5
ECS- 6-6'x6' Box Culvert With Sluice Gate - Reach H1	29.29727	-90.65869	12.5	1.0	13.7	20.5	22.5
Lapeyrouse Sector Gate	29.30892	-90.64590	13.4	1.1	14.7	22.0	24.0
Placid Canal Sector Gate	29.33804	-90.63685	13.4	1.1	14.7	22.0	24.0
Bayou Petite Caillou Sector Gate	29.29672	-90.64844	12.5	1.0	13.7	20.5	22.5
Bush Canal Sector Gate	29.36816	-90.60206	13.9	1.2	15.5	23.0	25.0
Bayou Terrebonne Floodgate	29.38804	-90.58806	13.9	1.2	15.5	23.0	25.0
Humble Canal Sector Gate	29.43804	-90.56649	13.9	1.2	15.5	23.0	25.0
ECS- 5-5'x10' Box Culverts With Sluice Gates - Reach J2	29.43639	-90.53922	13.9	1.2	15.5	23.0	25.0
ECS- 4-5'x10' Box Culverts With Sluice Gates - Reach J2	29.44678	-90.51375	13.9	1.2	15.5	23.0	25.0
Point Au Chiene Sector Gate	29.41569	-90.44689	13.9	1.2	15.5	23.0	25.0
ECS- 2-6'x6' Box Culverts With Sluice Gates - Reach K	29.44617	-90.44638	13.1	1.3	14.7	19.0	21.0
ECS- 2-6'x6' Box Culverts With Sluice Gates - Reach K	29.46964	-90.43881	13.1	1.3	14.7	19.0	21.0
Grand Bayou Sector Gate	29.51833	-90.41173	13.1	1.3	14.7	19.0	21.0
ECS- 6-5'x10' Box Culverts With Sluice Gates - Reach L	29.50408	-90.41902	13.1	1.3	14.7	19.0	21.0
Four Point Bayou Sector Gate	--	--	12.5	1.0	13.7	20.5	22.5
Bayou Lafourche Gate (GIWW East)	29.58809	-90.37252	9.7	0.6	10.4	15.0	17.0

2.6.2.3.2 Barrier Alignment

2.6.2.3.2.1 Future Conditions

A total of 6 structure designs were done for the barrier alignment. No structures were designed for the northern alignment. The same methodology used in Table 49 was used here to develop the future condition design parameters for selected points. Since only 3 unique surge and wave point combinations existed, the future condition computations were done only with the three combinations. The full future condition computation table is broken into two sections (Table 58 and Table 59). Based on the resulting design parameters the future condition designs are provided in Table 60.

Table 58 - Part 1 - Barrier alignment future condition (2085) 2.86% design parameter computations

		ERDC Values - SWE - Existing Conditions (0 ft SLR)					SWE Existing		SWE StDEV Existing Conditions
SWE Existing - 2008	Pt	10	50	75	100		35 yr		35 yr
Barrier Structure Type 1	233	2.00	5.47	6.59	7.35		4.7		0.80
Barrier Structure Type 2	215	2.30	5.96	7.2	8.05		5.2		0.85
Barrier Structure Type 3	261	1.50	4.91	5.96	6.69		4.2		0.80

		50 yr				35 yr - 2035			
Calculate SWE Futures - 2035	Pt	ERDC 2008 (no slr)	Base 2035 (0.72 slr)	Ratio $(base_{2035}/exis_{2010})$		SWE - Base (0.72 ft slr)			
Barrier Structure Type 1	233	5.5	7.7	1.406		6.61			
Barrier Structure Type 2	215	6.0	8.2	1.367		7.11			
Barrier Structure Type 3	261	4.9	7.4	1.509		6.34			

		50 yr				35 yr - 2035			
Calculate SWE StDev Futures - 2035	Pt	StDev 2008 (no slr)	StDev 2035 (0.72 slr)	Ratio $(base_{2035}/exis_{2010})$		SWE StDev - Base (0.72 ft slr)			
Barrier Structure Type 1	233	0.87	1.01	1.161		0.93			
Barrier Structure Type 2	215	0.93	1.07	1.151		0.98			
Barrier Structure Type 3	261	0.87	1.01	1.161		0.93			

		50 yr						35 yr	35 yr
Hs Existing	Pt	ERDC 2008 - Hs	ERDC 2008 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)		Hs - Existing	Hs - Base (2035)
Barrier Structure Type 1	233	1.30	5.5	1.35	4.15	0.31		1.05	1.65
Barrier Structure Type 2	221	1.30	6.0	1.12	4.88	0.27		1.09	1.60
Barrier Structure Type 3	261	0.90	4.9	1.18	3.72	0.24		0.73	1.25

Table 59 - Part 2 - Barrier alignment future condition (2085) 2.86% design parameter computations

Tm Existing/Futures - 2008/2035	Pt	Constant Wave Steepness	2008 - Tm	Base 2035 - Tm			
			35 yr				
Barrier Structure Type 1	233	0.04	2.3	2.8			
Barrier Structure Type 2	221	0.04	2.3	2.8			
Barrier Structure Type 3	261	0.04	1.9	2.5			

Calcuate SWE Futures - 2085	Pt	2035 (0.72 slr)	2085 (2.42 slr)	Ratio (fut$_{2085}$/fut$_{2035}$)		SWE - Future (2.42 ft slr)	
			50 yr			**35 yr - 2085**	
Barrier Structure Type 1	233	7.7	11.2	1.45		9.57	
Barrier Structure Type 2	215	8.2	11.4	1.39		9.91	
Barrier Structure Type 3	261	7.4	10.9	1.47		9.29	

SWE StDev Futures - 2085	Pt	StDev 2035 (0.72 slr)	StDev 2085 (2.42 slr)	Ratio (fut$_{2085}$/fut$_{2035}$)		SWE StDev - Future (2.42 ft slr)	
			50 yr			**35 yr - 2085**	
Barrier Structure Type 1	233	1.01	1.15	1.14		1.06	
Barrier Structure Type 2	215	1.07	1.19	1.11		1.09	
Barrier Structure Type 3	261	1.01	1.11	1.10		1.02	

Hs Existing	Pt	2035 - Hs	2035 - SWE	Bottom Elev. (ft.)	Depth (ft)	Ratio (Hs/Depth)	Hs - 2085
			50 yr				**35 yr**
Barrier Structure Type 1	233	2.3	7.7	1.35	6.35	0.36	2.98
Barrier Structure Type 2	221	2.2	8.2	1.12	7.08	0.32	2.78
Barrier Structure Type 3	261	1.9	7.4	1.18	6.22	0.31	2.48

Tm Existing/Futures - 2085	Pt	2085 - Tm
		35 yr
Barrier Structure Type 1	233	3.8
Barrier Structure Type 2	221	3.7
Barrier Structure Type 3	261	3.5

Table 60 - Future condition (2085) barrier alignment structure designs corresponding to a 2.86% frequency.

35 YEAR BARRIER ALIGNMENT DESIGNS

STRUCTURE	SWE Pt	Wave Pt	SWE (ft.)	StDev (ft.)	90% Surge (ft.)	Hs (ft.)	Hs StDev (ft.)	Tm (s)	Tm StDev (s)	Struc Invert (ft.)	Design Elev. (ft.)
Shell Canal E	233	233	9.57	1.06	10.93	3.0	0.5	3.8	0.3	-12.0	16.0
Shell Canal W	233	233	9.57	1.06	10.93	3.0	0.5	3.8	0.3	-10.0	16.0
Elliot Jones Canal	233	233	9.57	1.06	10.93	3.0	0.5	3.8	0.3	-8.0	16.0
NAFTA	215	221	9.91	1.09	11.31	2.8	0.5	3.7	0.3	-12.0	16.0
Humphries Canal	215	221	9.91	1.09	11.31	2.8	0.5	3.7	0.3	-8.0	16.0
Bayou Black	261	261	9.29	1.02	10.60	2.5	0.5	3.5	0.3	-12.0	15.0

The design elevation contains 2 feet of structural superiority.

2.6.2.4   Wave Load Designs

Wave loads corresponding to a 2.86% frequency were computed based on the structure designs given in Table 60. The matlab script was used here to produce the load results. Wave loads were not computed for the northern alignment because no structures were designed.

2.6.2.4.1 Authorized and Barrier Alignments

2.6.2.4.1.1 Future Conditions

Wave load output corresponding to a 2.86% frequency event for both authorized and barrier alignment can be furnished upon request. All structures included in the 1% wave load computations for authorized alignment were included here for 2.86% computations. As mentioned previously a total of 6 structures were used for wave loading computations for the barrier alignment.

2.6.3   Environmental Control Structures

2.6.3.1   Introduction

The Environmental gate sizing analysis was completed by ERDC. A summary of the analysis is provided here, while the complete report can be furnished upon request. The hydrologic and hydraulic analysis of the proposed Environmental Flow Control Structures (EFCS) for the isolated system areas in Reaches 4, 3, 2, B, G, H, J2, K and L of the MTG project. An isolated system area is a near-shore area of marsh or open water that will be separated from the open tidal zone by the MTG levee alignment. An EFCS allows drainage of interior stormwater and provides limited tide exchange for areas isolated by the new levee. Isolated system areas in the MTG project vary in size from several thousand acres to many square miles. The EFCS are intended to be closed during storm surge events and are not designed for navigation passage. Storm gates sizing is discussed in Section 2.6.3.6. The analysis and findings assume that the existing drainage system will function the same after the proposed levee construction. Therefore; only isolated system areas are considered in locating and sizing EFCS.

2.6.3.2   Isolated System Area by Reach

The **Reach B** system is comprised of one isolated system area (ISA). Reach B culvert group 1 will drain 2.25 square miles from a portion of sub-basins BD-1. The new MTG levee will be constructed along the alignment of the existing levee on the west side of FDA 8-2C and 8-2D and extend north along the western boundary of BD-1 until it crosses the GIWW. Topography along the culvert location has elevations +2.5 ft NAVD88 2004.65 or higher will not be subject to tidal influence. All sub-basins north of the GIWW, GW-17 and portions of GW-16 and BD-1 drain to the GIWW and will be transported out of the basin by the GIWW.

The **Reach 4** system is comprised of one isolated system area (ISA), two forced drainage areas (FDA), and two pump stations. Reach 4 culvert group 1 will drain 0.5 square miles including portions of sub-basins E1 and FDA D-42. Reach 4 culvert group 2 will drain 1.57 square miles including the north end of E-1 and FDA D-44. Topography along the culvert locations have elevations +2.5 ft NAVD88 2004.65 or

higher will not be subject to tidal influence. All sub-basins north of the Black Bayou will drain to and be transported out of the basin by Black Bayou.

The **Reach 3** system is comprised of one ISA. Reach 3 culvert group 1 will drain 0.75 square miles including portions of sub-basins E2 and E2-LF. Reach 3 culvert group 2 will drain 1.45 square miles including portions of E2 and E2-LF. Topography along the Reach 3 culvert locations have elevations +2.5 ft NAVD88 2004.65 or higher will not be subject to tidal influence. All sub-basins north of the Black Bayou will drain to and be transported out of the basin by Black Bayou.

The **Reach 2** system is comprised of one ISA including sub-basins E-2, E2-LF and GW-18. Reach 2 culvert groups 1 and 2 will drain 3.13 square miles. Topography along the Reach 2 culvert locations have elevations +2.5 ft NAVD88 2004.65 or higher will not be subject to tidal influence. All sub-basins north of the Black Bayou will drain to and be transported out of the basin by Black Bayou.

The **Reach E** will not have an ISA due to the high ground along the southern end of the reach. Rainfall events will be passed through the Houma Navigation Canal.

The **Reach G** system is comprised of one ISA HNC9 and one FDA D-10. Reach G culvert group 23 will drain 6.32 square miles. Assume all pumps will not be operational during storm events. Reach G will be subject to tidal activity.

The **Reach H** system is comprised of one ISA including LB1, LB2, LB3, LB4, LB5, and HNC10 and six FDA's 1-5, D60, D-36, 3-1B, 5-1A, and 5-1B. Reach H culvert group 24 and 25 will drain 91.83 square miles. Reach H will be subject to tidal activity.

The **Reach J2** system is comprised of two ISA's including ISA W, ISA E and seven FDA's 1-4, D-03, 4-1, D-25, D-02, 1-4, and D-69. Reach J2 culvert group 26, 27, and 28 will drain the storage area. Reach J2 will be subject to tidal activity.

The **Reach K and L** system is comprised of one ISA including SL1, SL2, SL3, PAC1. Reach K and L culvert group 29, 30 and 31 will drain 43.34 square miles. Reach K and L will be subject to tidal activity.

2.6.3.3   Environmental Flow Control Structure Design Criteria

The Project Delivery Team (PDT) and Habitat Evaluation Team (HET) established the following criteria for determining the size of the MTG Environmental Flow Control Structures (EFCS):

1. The 10% AEP 24-hour precipitation event is the design condition for the EFCS,

2. Performance of the EFCS should be checked for the 1% AEP 24-hour precipitation event,

3. The increased water level in the isolated system areas due to the 10% AEP 24-hour precipitation runoff should recede to the external tide zone water surface level within four to seven days when the storm surge gates are open,

4. The initial water surface level in the isolated system area should be the medium 2% AEP sea level rise,

5. The EFCS should be box culverts,

6. The length of culverts should be as short as possible,

7. The invert of culverts should be as close to the existing bed as possible,

8. The slope of culverts should be zero (no slope),

The average flow velocities during peak flood or ebb tides through the EFCS should not exceed 2.6 ft/s.

2.6.3.4   Hydrology

Design stormwater inflow for each reach was determined by hydrologic modeling of the 10 percent AEP and 1 percent AEP exceedance precipitation events. Stormwater runoff modeling was performed with the Hydrologic Engineer Center Hydrologic Modeling System (HEC-HMS 3.5). Further detail on the hydrologic analysis can be found in the full report MTG – Environmental Flow Control Structures Study and can be furnished upon request.

2.6.3.5   Hydraulic Analysis of the Environmental Flow Control Structures

The MTG Project creates a system of forced drainage areas connected by pumps and open channels that are unique for each reach along the proposed levee. The HET criteria requires the EFCS drain the increased water level in the isolated system areas due to the 10% AEP 24-hour precipitation runoff to the external tide zone water surface level within four to seven days when the storm surge gates are open. Performance of the EFCS was also checked for the 1% AEP 24-hour precipitation event. The average flow velocities during peak flood and ebb tides through the EFCS should not exceed 2.6 ft/s. The results of the HEC-RAS simulations provide the basis for the EFCS size recommendations.

2.6.3.6   Preliminary Environmental Flow Control Structure Sizes

Preliminary sizes for environmental flow control structures (EFCS) for the MTG Proposed Levee Reaches were suggested in the 2000 feasibility report. In the feasibility study documentation was a system map that suggested locations and sizes of

EFCS for the MTG project. The EFCS sizes suggested in the feasibility report and system map are summarized in Table 61.

Table 61 - Preliminary sizes and locations of EFCS for all Levee Reaches

Table 3- Environmental Structures*,**					
Levee Reach	Structure Geometry	Invert	Height	Coordinates	
				X	Y
B					
20	6-6x6 box culverts	-4.5	23	29°27'39.72"N	90°45'49.93"W
E					
21	9-6x6 box culverts	-4.5	25.5	29°24'26.31"N	90°46'11.40"W
22	9-6x6 box culverts	-4.5	25.5	29°23'55.60"N	90°44'46.40"W
G-2					
23	6-6x6 box culverts	-4.5	30.5	29°19'4.20"N	90°41'44.24"W
H-1					
24	1-6x6 box culverts	-4.5	30.5	29°18'8.15"N	90°40'12.05"W
25	6-6x6 box culverts	-4.5	30.5	29°17'50.16"N	90°39'31.28"W
J-2					
26	5-5x10 box culverts	-3.5	33	29°26'11.02"N	90°32'21.19"W
27	4-5x10 box culverts	-3.5	33	29°26'48.39"N	90°30'49.51"W
28	5-5x10 box culverts	-3.5	33	29°27'16.38"N	90°30'0.34"W
K					
29	2-6x6 box culverts	-4.5	29.5	29°26'46.20"N	90°26'46.96"W
30	2-6x6 box culverts	-4.5	29.5	29°28'10.71"N	90°26'19.71"W
L					
31	6-6x6 box culverts	-4.5	29.5	29°30'11.73"N	90°25'9.78"W
Barrier Plan-Reach 4	1-6x6 box culvert	-4.5	23	29°40'10.935"N	91°0'20.012"W
	1-6x6 box culvert	-4.5	23	29°39'40.218"N	90°59'0.639"W
	1-6x6 box culvert	-4.5	23	29°38'48.282"N	90°58'17.71"W
Barrier Plan-Reach 3	1-6x6 box culvert	-4.5	23	29°36'45.149"N	90°54'17.183"W
	1-6x6 box culvert	-4.5	23	29°36'7.937"N	90°50'58.563"W
Barrier Plan-Reach 2	1-6x6 box culvert	-4.5	23	29°36'4.44"N	90°50'48.434"W
	1-6x6 box culvert	-4.5	23	29°35'36.08"N	90°50'5.268"W

2.6.3.7 Conclusions and Final Environmental Flow Control Structures

Details on the hydrologic and hydraulic analysis of the proposed EFCSs for the reaches of the MTG Project can be furnished upon request. Design criteria were discussed and the precipitation depth-duration-frequency of all reaches was determined using published data. USGS gages, referenced previously in this report were used to determine tide stages. The sea level rise used was provided by USACE, MVN and was 2.42 feet. A HEC-HMS precipitation model was developed to derive the inflow hydrographs for a HEC-RAS simulation of the EFCS. All Reaches were modeled in HEC-RAS. Trial box culvert sizes were evaluated with the HEC-RAS model. Based on the hydraulic modeling, modifications of the original EFCS geometry were necessary to meet the HET tidal criteria (Section 2.6.3.3). Existing site conditions should be defined by site visits to determine the conditions and elevations at the site to check that culvert

elevations are close to the ground elevations. Table 62 lists the levee reaches, culvert group number, recommended geometry, recommended invert elevation, and recommended latitude and longitude for the EFCS in the MTG project.

Table 62 - Recommended culvert sizes and location based on HEC-RAS simulation

Levee Reach	EFCS Geometry	Invert Elevation	Latitude	Longitude
Culvert number				
B				
20	3-6x6 box culverts	-1.5	9°30'55.99"N	90°46'0.83"W
E				
	Culverts not needed			
G				
23	6-8x6 box culverts	-4.5	29°19′4.20″N	90°41′44.24″W
23A	6-8x6 box culverts	-4.5	29°18′49.21″N	90°40′5.29″W
H				
24	1-6x6 box culverts	-4.5	29°18′8.15″N	90°40′12.05″W
25	6-6x6 box culverts	-4.5	29°17′50.16″N	90°39′31.28″W
J2				
26	14-6x6 box culverts	-4.5	29°26'12.21"N	90°32'22.13"W
27	14-6x6 box culverts	-4.5	29°26'33.45"N	90°31'15.87"W
28	10-10x6 box culverts	-4.5	29°27'20.46"N	90°29'59.10"W
K & L				
30	8-8x8 box culverts	-2.5	29°28′10.71″N	90°26′19.71″W
31	8-8x8 box culverts	-2.5	29°30′11.73″N	90°25′9.78″W
Barrier Plan				
Reach 4				
	1-6x6 box culverts	-1.5	29°37'37.76"N	90°56'50.86"W
	1-6x6 box culverts	-1.5	29°38'48.28"N	90°58'17.71"W
Barrier Plan				
Reach 3				
	1-6x6 box culverts	-1.5	29°37'16.87"N	90°55'18.70"W
	1-6x6 box culverts	-1.5	29°36'46.87"N	90°54'21.89"W
Barrier Plan				
Reach 2				
	1-6x6 box culverts	-1.5	29°36'1.93"N	90°50'48.67"W
	1-6x6 box culverts	-1.5	9°35'36.05"N	90°50'5.27"W

2.6.4   Gate Structures

2.6.4.1   ADH Modeling

2.6.4.1.1 Introduction

The U.S. Army Engineer Research and Development Center (ERDC) performed a number of engineering studies in support of efforts to determine the proper sizes of six proposed structures (Bush Canal, Bayou Terrebonne, Lapeyrouse Canal, Placid Canal, Bayou Petit Caillou, and Humble Canal). These structure sizes were determined through numerical modeling using the Adaptive Hydraulics Code (ADH). AdH is a state-of-the-art code developed by the U.S. Army ERDC to simulate both saturated and unsaturated groundwater, overland flow, three-dimensional Navier-Stokes flow, and two- or three-dimensional shallow water problems (Berger, et al., 2010). The study area along with the proposed levee alignment and structure locations are shown in Figure 34. The primary objective was to determine the smallest structures that resulted in reasonable velocity fields for the six proposed locations.

Figure 34 - Proposed Levee Alignment and Structure Locations

Once all navigational and environmental structure sizes were determined using ADH, the final configuration was modeled using a previously validated TABS-MDS model. This model determined the effects of the levee system on the salinity conditions throughout the system. Discussion of the TABS-MDS systemwide model can be found in Section 2.7.1.

2.6.4.1.2 Model Development

An existing RMA2 mesh of the south central Louisiana coast was provided by the New Orleans District. This initial mesh was created by Dr. Joseph V. Letter, Jr., for an Atchafalaya Bay study using RMA2 (Donnell et al.,1991). It was later modified by Mr. David Elmore and again by Ms. Amena Henville (both of MVN) for the MTG project, also using RMA2. This initial mesh, shown in Figure 35, extends from the Atchafalaya Bay on the west to Port Fourchon on the east. It contains a large area to the west of the study area that was not necessary for the current numerical model study.
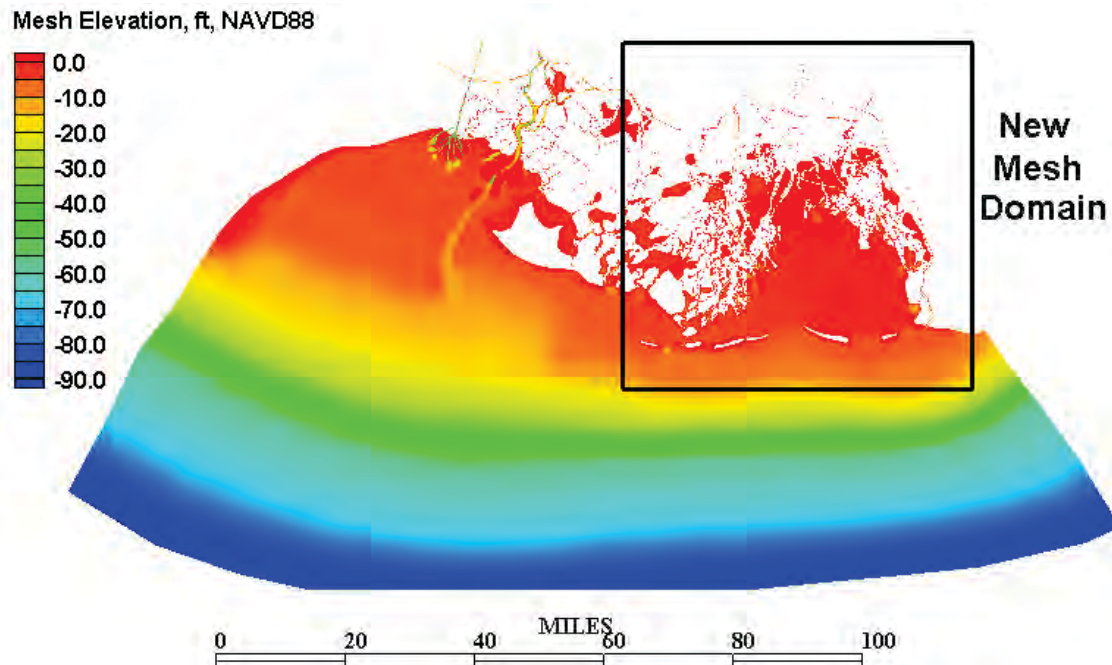
Figure 35 - Initial RMA2 mesh developed for previous study, with current study area indicated

The western area outside the greater study location was removed and the remaining mesh converted to an ADH compatible format for the Bush Canal study (McAlpin et al. 2009). For the current MTG gate sizing study, the resolution in the study area near proposed gate locations was significantly increased. Bathymetry data were also taken near the proposed structure locations by the CHL field crew and incorporated into the model.
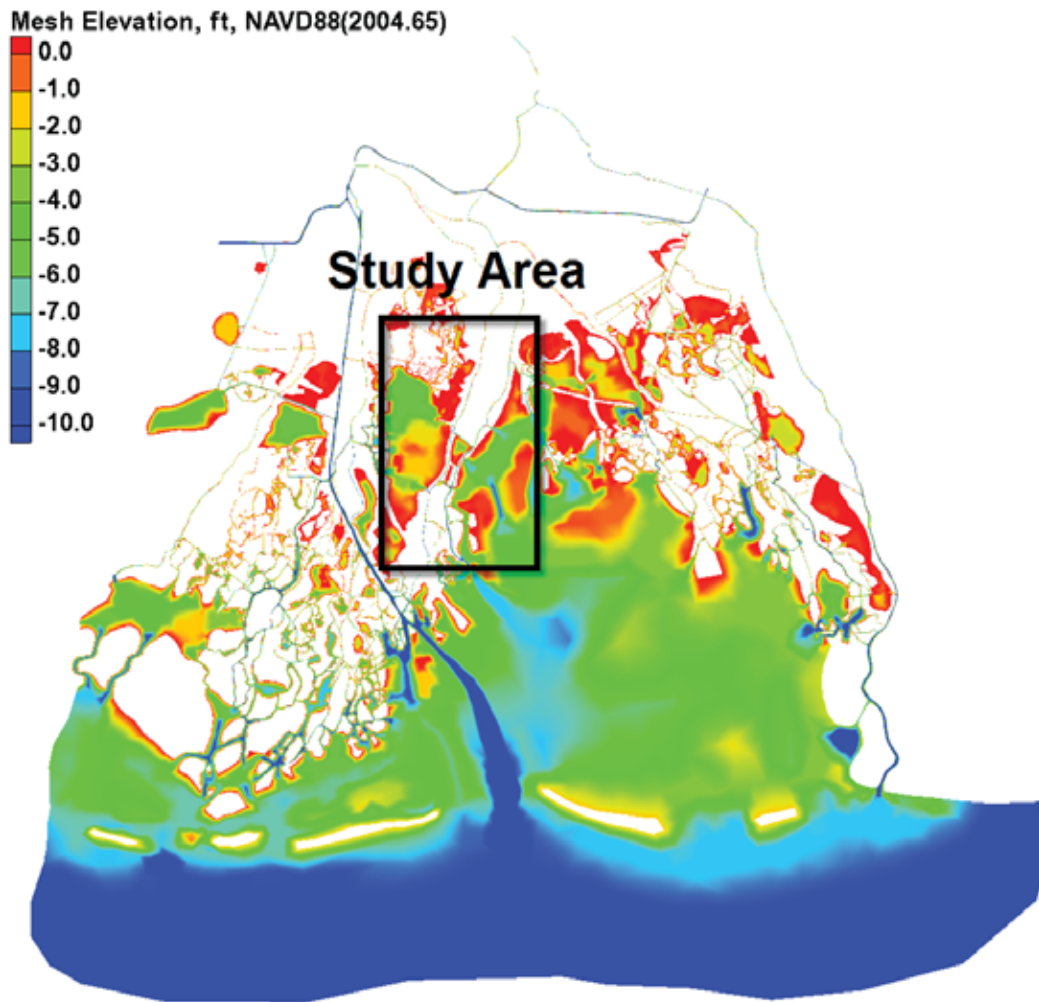
Figure 36 - Model Domain for Current Study

These mesh modifications were performed in the Surface Water Modeling System (SMS), a graphical user interface developed for use in setting up and running numerical models (Aquaveo 2009). The final model domain and bathymetry used in this study are shown in Figure 36. The horizontal and vertical coordinate systems were State Plan 83, Louisiana South, ft and NAVD88(2004.65), ft respectively. Further model information including boundary condition development and model validation can be furnished upon request.

2.6.4.1.3 Design Alternatives

The aim of the design alternatives was to determine appropriate structure sizes that would also result in reasonable velocity fields. The initial design included navigational structures and environmental structures. This began with an initial configuration, Plan 1 (Figure 37), that was arranged based on preliminary model results. The evolution of the plan configurations (Plan 1 –Plan 6) consisted solely of modifications to the Placid Canal and Bayou Petit Caillou structures. All remaining navigational and environmental

structures were left unchanged from the initial plan configuration. The modifications of these structures from Plan 1 to Plan 6 are provided in Table 63 at the end of this section along with a bar plot comparison(Figure 53) of the maximum velocities for those two structures for all modeled configurations. The final configurations, Plan 6 and Plan 7 (Plan 6 with environmental structures closed), are also discussed in this section. It should be noted that the structure configurations are conceptual in nature as finalized designs have not been created.
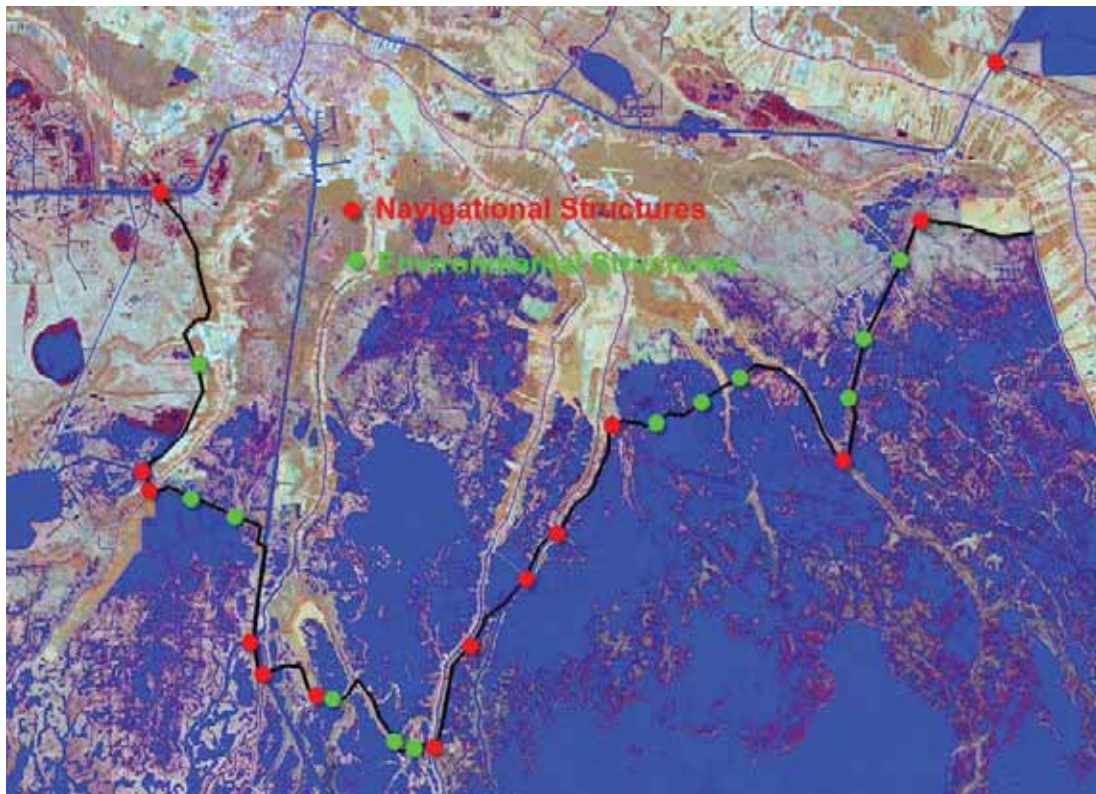
Figure 37 - Structure locations

The GIWW west of Houma structure, shown in Figure 38, consisted of two 125 ft wide structures with bottom elevation of -20 ft, NAVD88(2004.65).
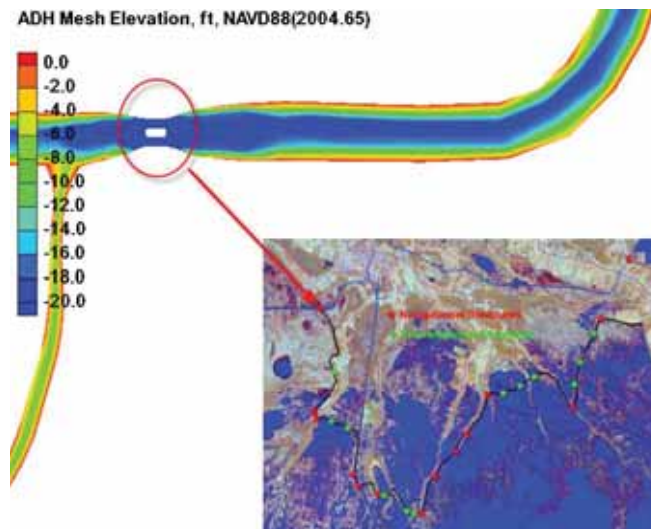
Figure 38 - GIWW West of Houma Structure

The formerly free-flowing Marmande Canal is joined by a set of six 6'x6' culverts. For this modeling effort the widths for these culverts were combined into one culvert with a width of 36 ft and a bottom elevation of -4.5 ft NAVD88(2004.65)). The model representation of this configuration is shown in Figure 39.
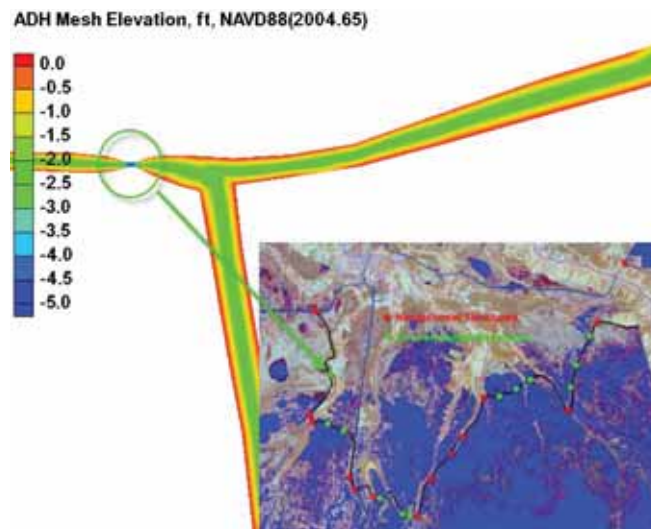
Figure 39 - Model representation of the Marmande Canal culverts

The Falgout Canal (upper left circle) and the Bayou Dularge (lower circle) structures are shown in Figure 40. The Falgout Canal structure consists of one 56 ft sector gate and three 46 ft sluice gates with -9 ft NAVD88(2004.65) bottom elevations. The Bayou Dularge structure consists of one 56 ft sector gate with a bottom elevation of -7 ft NAVD88(2004.65).
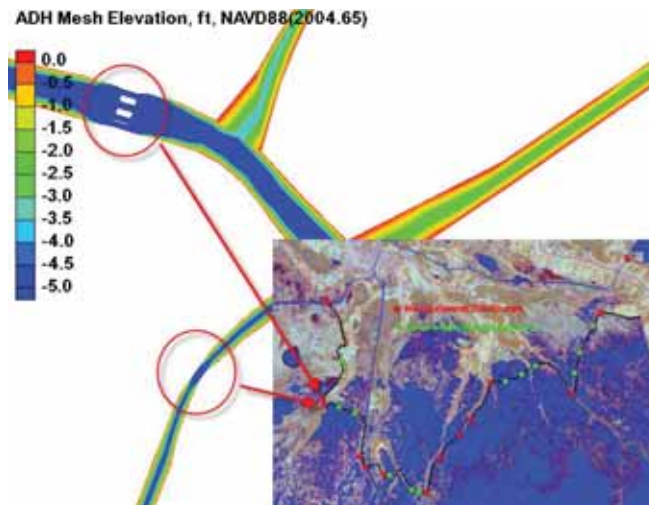
Figure 40 - Model representation of the Falgout Canal and Bayou Dularge structures

Two sets of nine 6'x6' culverts are located along Falgout Canal. In the model, the two sets of culverts are represented as single culverts with widths of 54 ft and bottom elevations of -4.5 ft NAVD88(2004.65). This configuration is shown in Figure 41.
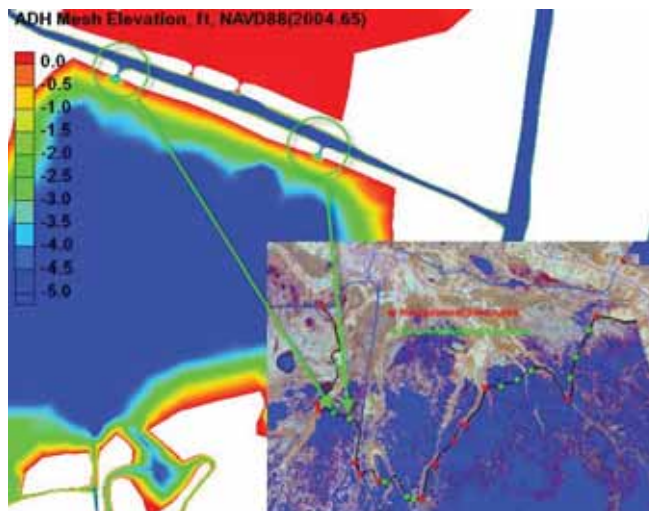
Figure 41 - Model representation of the two sets of culverts located along Falgout Canal.

The Bayou Grand Caillou structure, shown in Figure 42 in the top circle, consists of three 46' sluice gates and one 56' sector gate possessing bottom elevations of -12 ft, NAVD88(2004.65). The Houma Navigation Canal structure consists of a 250 ft wide structure and a 110 ft wide lock, both with bottom elevations of -23 ft, NAVD88(2004.65). The -23 ft bottom elevation for this structure is based on the 50% Plans and Specifications (P&S) for the Houma Navigational Canal. The Houma Navigational Canal structure will also consist of ten 10 ft wide sluice gates, each with a 5 ft vertical opening (from -2 ft to -7 ft NAVD88(2004.65). Four of these sluice gates will be located on the eastern side of the structure and four on the western side of the structure. Two will be located between the lock and the sector gate structures. This configuration can be observed in Figure 42.
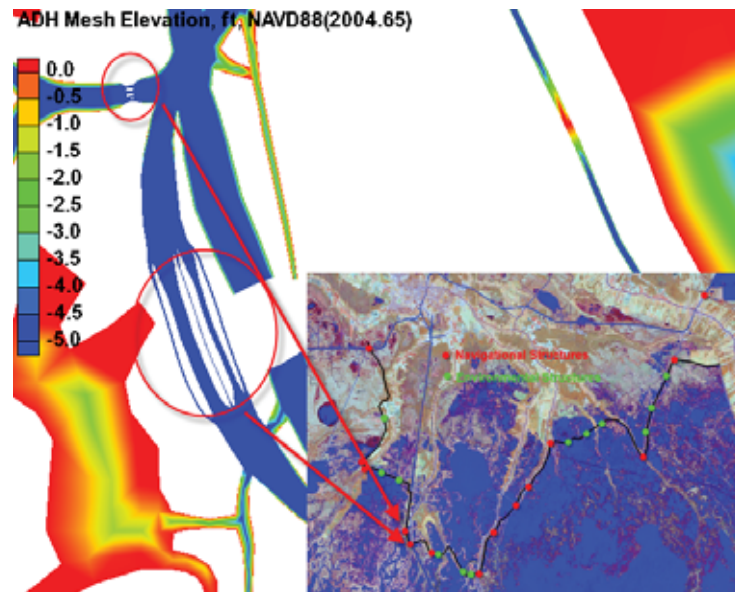
Figure 42 - Model representation of the Bayou Grand Caillou structure and the Houma Navigational Canal structure and lock

The structure located on Bayou Fourpoints (red circle on left in Figure 43) had a 30 ft sector gate with a -8 ft NAVD88(2004.65) bottom elevation. It should also be noted that two existing earthen plugs were removed and another plug was installed farther to the North as described in Permit MVN-2008-518-CT (U.S. Army Corp of Engineers, 2008). A set of six 6'x6' culverts are located just east of Bayou Fourpoints (green circle in Figure 43). This structure was represented as a single culvert with a 36 ft width and a bottom elevation of -4.5 ft, NAVD88(2004.65).
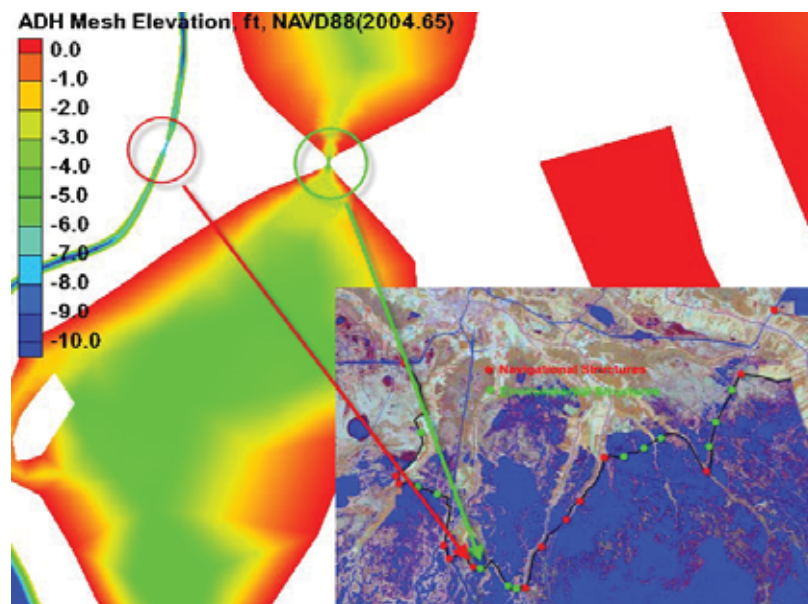
Figure 43 - Model representation of the Bayou Fourpoints structure and one culvert set

The left circle in Figure 44 is a single culvert with a width of 6 ft and a bottom elevation of -4.5 ft, NAVD88(2004.65). Another set of six 6 ft wide culverts are shown in Figure

44 (right circle). This culvert set also has a bottom elevation of -4.5 ft, NAVD88(2004.65).
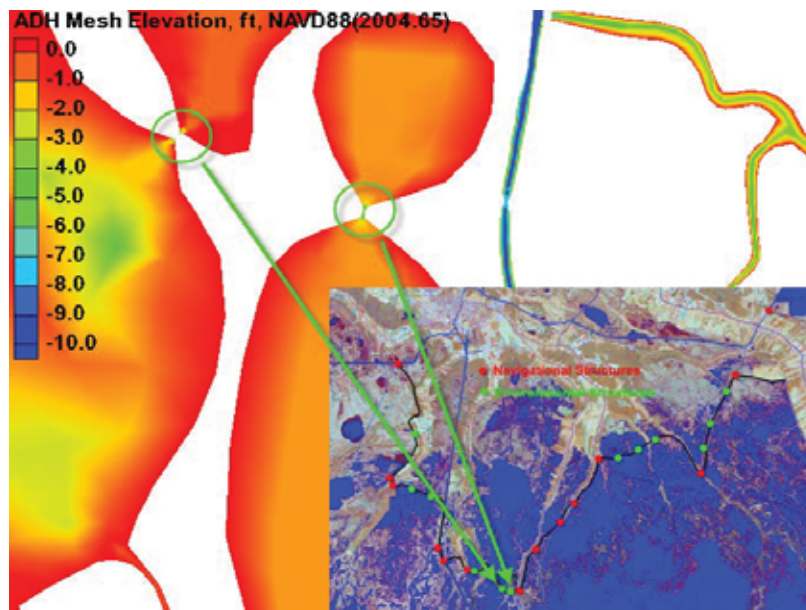
Figure 44 - Two culvert sets west of Bayou Petit Caillou

The Bayou Petit Caillou structure shown in Figure 45, is a 56 ft wide sector gate with two 46 ft wide sluice gates with bottom elevations of -8 ft, NAVD88(2004.65).
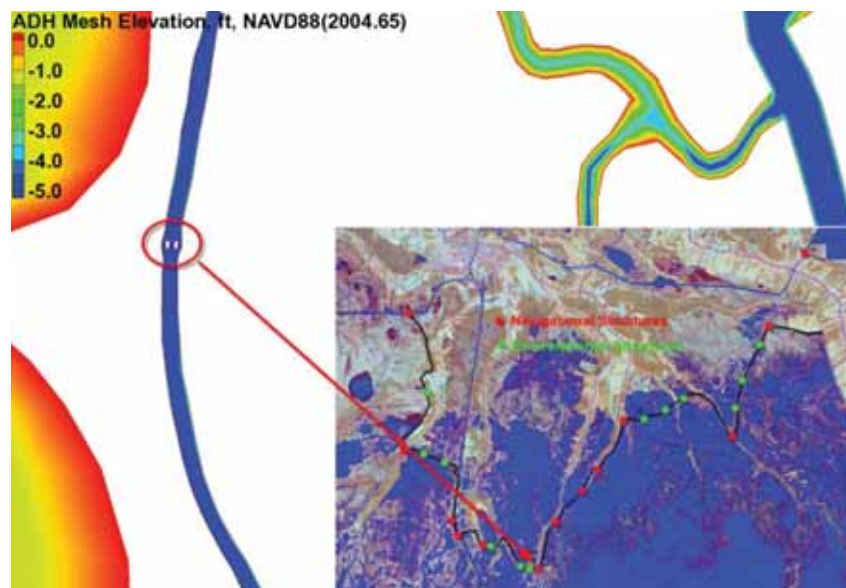
Figure 45 - Model representation of the Bayou Petit Caillou structure and the Lapeyrouse Canal closure

The Placid Canal structure, shown in Figure 46, consists of one 56 ft sector gate and two 46 ft wide sluice gates with a bottom elevation of -8 ft, NAVD88(2004.65).
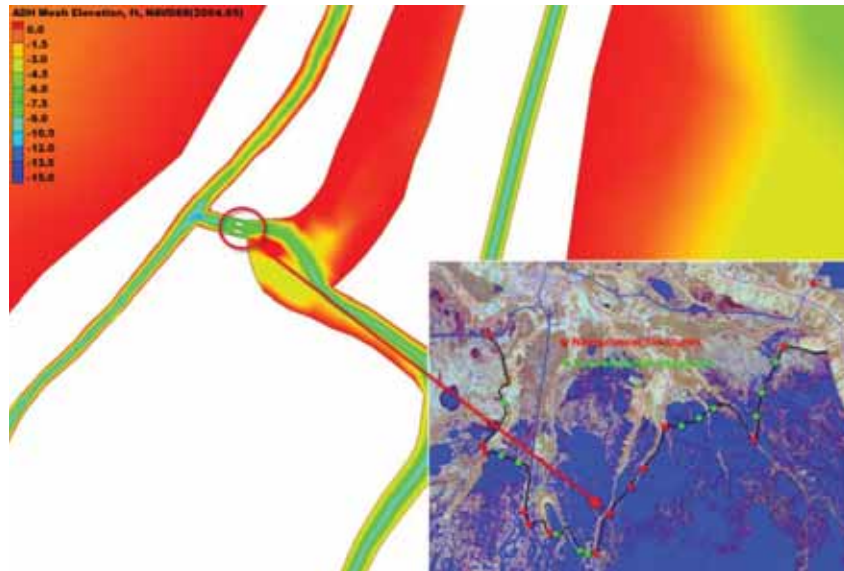
Figure 46 - Model representation of the Placid Canal structure

The Bush Canal structure, the lower circle shown in Figure 47, consists of three 46 ft sluice gates and one 56 ft sector gate. This structure had a bottom elevation of -12 ft, NAVD88(2004.65). The Bayou Terrebonne structure (upper circle) is a 56 ft sector gate with a bottom elevation of -9 ft, NAVD88(2004.65).
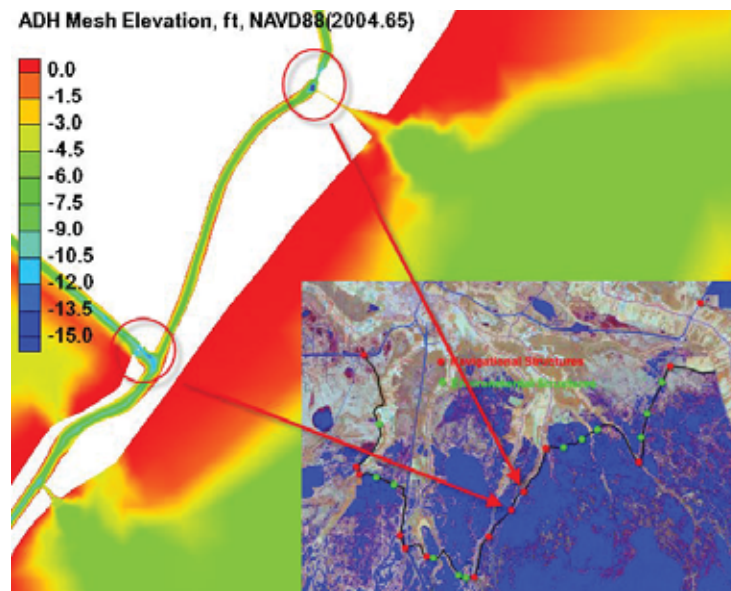
Figure 47 - Model representation of the Bush Canal and Bayou Terrebonne structures

The Humble Canal structure, shown in Figure 48, is a 56 ft sector gate with a bottom elevation of -9 ft, NAVD88(2004.65).
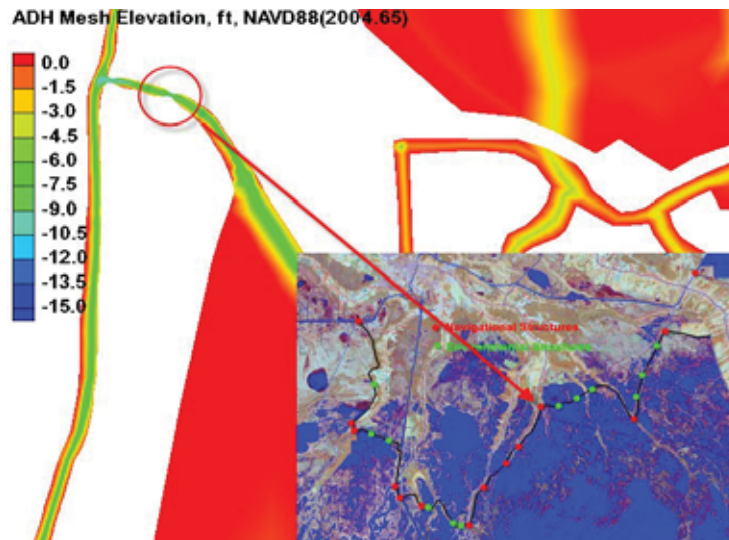
Figure 48 - Model representation of the Humble Canal structure

Shown in Figure 49 are three sets of culverts located in the Wonder Lake area (also known as the Montegut Wildlife Management Area). The left set of culverts consists of five 5' x 10' culverts which were represented in the model as having a width of 50 ft with a bottom elevation of -4.5 ft, NAVD88(2004.65). The center set of culverts consists of four 5' x 10' culverts which are represented in the model as having a width of 40 ft and a bottom elevation of -4.5 ft, NAVD88(2004.65). The right most set of culverts is a set of five 5' x 10' culverts located on the eastern part of Wonder Lake. This set had a width of 50 ft and a bottom elevation of -4.5 ft, NAVD88(2004.65).
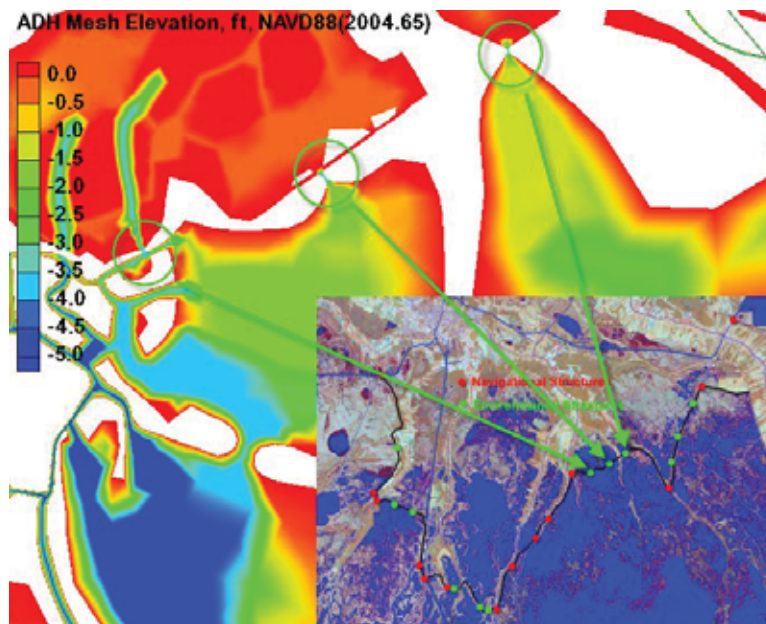
Figure 49 - Model representation of the Wonder Lake (Montegut Wildlife Management Area) culvert configuration

The Pointe-aux-Chenes structure, lower circle shown in Figure 50, has a 56 ft width with a bottom elevation of -6 ft, NAVD88(2004.65). Two 6' x 6' culverts, located along Grand Bayou Canal(upper circle shown in Figure 50) were represented as a single culvert with a width of 12 ft and a bottom elevation of -4.5 ft, NAVD88(2004.65).
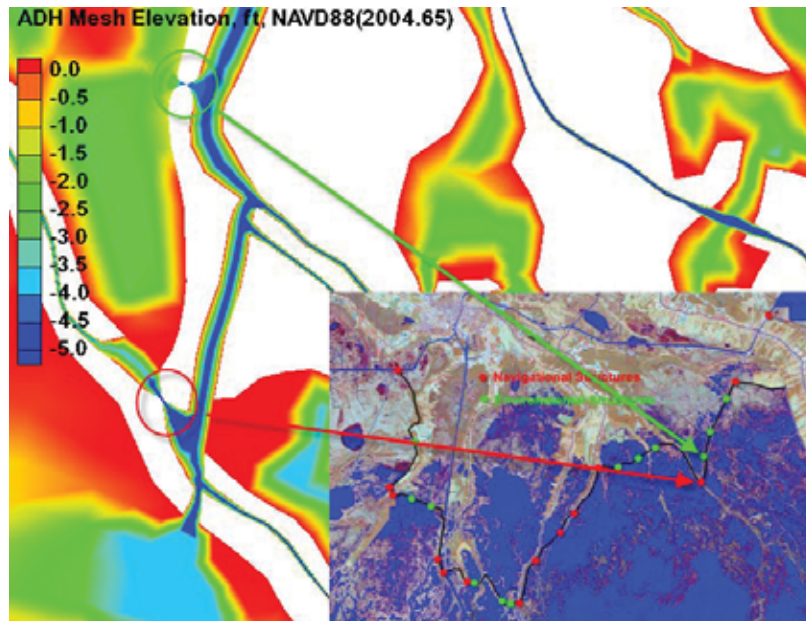
Figure 50 - Model representation of the Pointe-aux-Chenes structure and a culvert set

Two sets of two 6' x 6' culverts connected to Grand Bayou are shown in Figure 51. These two culvert sets were represented in the model as single culverts with widths of 12 ft with bottom elevations of -4.5 ft, NAVD88(2004.65).
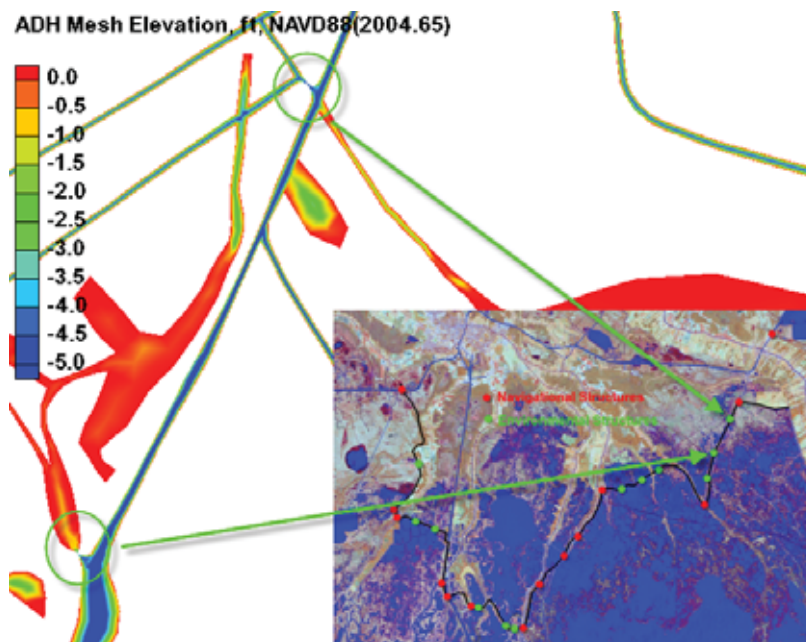
Figure 51 - Model representation of the culvert sets connected to Grand Bayou

Figure 52 is the structure located on Grand Bayou. This structure has a 56 ft wide sector gate with two 46 ft wide sluice gates with a bottom elevation of -9 ft, NAVD88(2004.65). The initial plan called for three sluice gates for this location. However, only two sluice gates are recommended by the final plan. This is due to the width of the channel and the small measured discharges for this area.

Figure 52 - Model representation of the Grand Bayou structure
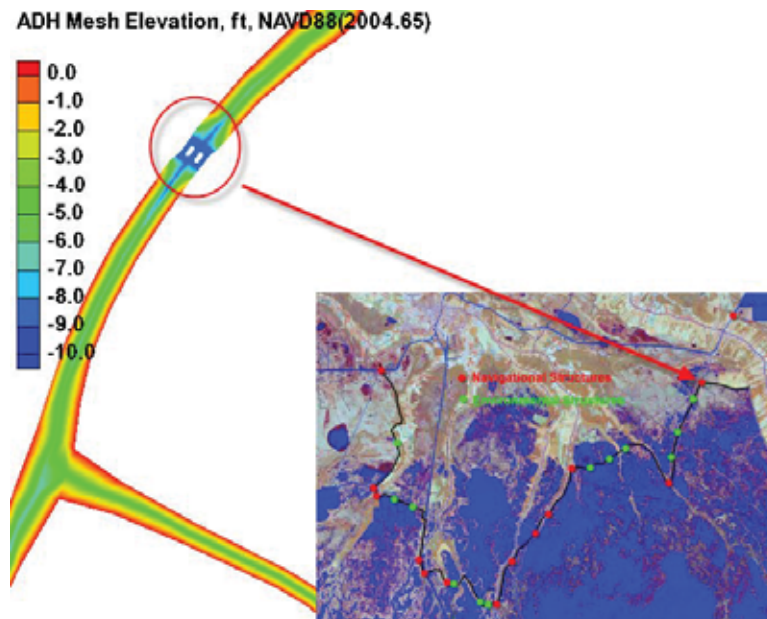
Table 63 - Evolution of the Plan Configuration for Placid Canal and Bayou Petit Caillou

Alternative Number	Placid Canal Sector Gate Width (ft)	Sluice Gates for Placid Canal	Bayou Petit Caillou Sector Gate Width (ft)	Sluice Gates for Bayou Petit Caillou
Plan 1	56	0	56	0
Plan 2	30 (Barge Gate)	0	56	0
Plan 3*	56	0	56	0
Plan 4	80	0	80	0
Plan 5	100	0	100	0
Plan 6**	56	2 – 46 ft Sluice Gates	56	2 – 46 ft Sluice Gates
Plan 7**	56	2 – 46 ft Sluice Gates	56	2 – 46 ft Sluice Gates

*Plan 3 has an added structure on Lapeyrouse Canal (56 ft wide sector gate with a bottom elevation of -10 ft, NAVD88(204.65). The Lapeyrouse Canal structure was eliminated for all other plan configurations.

**Plan 7 is the Plan 6 configuration with all Environmental Structures closed.
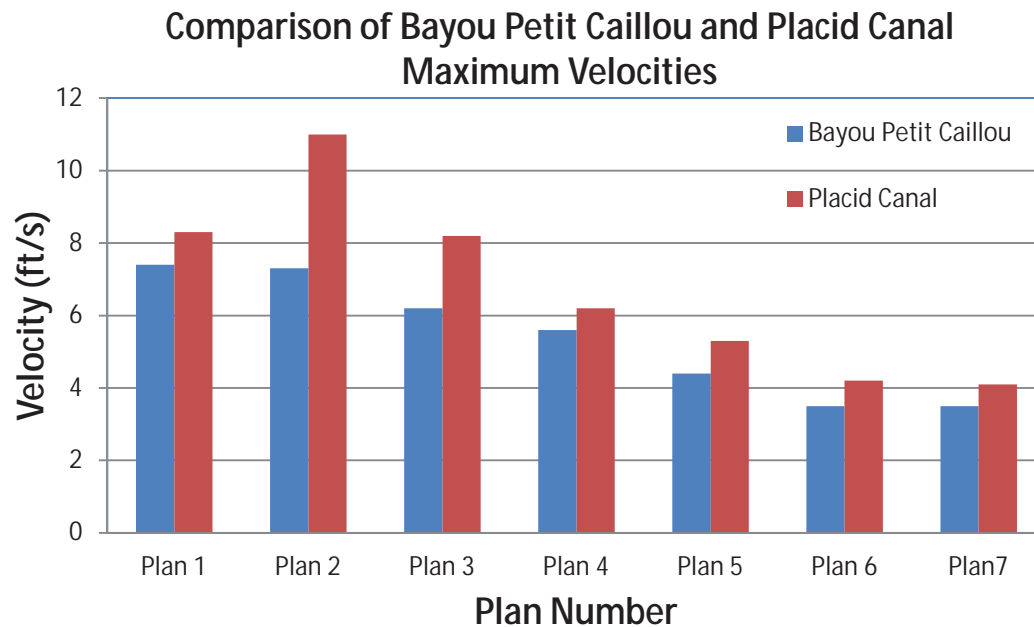
Figure 53 - Comparison of Petit Caillou and Placid Canal Maximum Velocities

2.6.4.1.4 Results

The plan simulations were performed using the same tidal, wind, and inflow forcing as those used during the base validation time period. This should be a reasonably accurate representation of a normal fall time period with the occurrence of several frontal passage events. These frontal passages occur approximately 2 times per month based on data observations, and can produce significant increases and decreases in water level over a short period of time thereby creating higher velocities throughout the system. From observations of measured data, these types of meteorological events are much less likely to occur during the summer months and therefore make the fall conditions more appropriate for these types of plan evaluations.

The inflow conditions for the western GIWW location are slightly higher than normal as this flow is directly related to the Atchafalaya River flow. The Atchafalaya River flow for the base validation time period was approximately 450 kcfs (Wax Lake Outlet flow plus the Atchafalaya River at Morgan City flow), which is slightly less than the 10% AEP flow of 570 kcfs (FEMA 2010). The flow for the base validation time period was approximately a 50% AEP flow event; therefore this flow event was a slightly higher than normal flow but not significantly so, again making it a good time period for this type of analysis.

A percentile analysis was performed on the Base Plan, Plan 6, and Plan 7 model simulations. Plots of the velocity exceedance values are presented in Figure 54 to Figure 56. Additional data including velocity exceedance plots and figures showing water surface differences can be furnished upon request. Percentile analysis prevents a biasing of the results by a single large event that investigation of the maximums alone would create. It should be noted that the model output velocity in these figures are for the sector gates only.
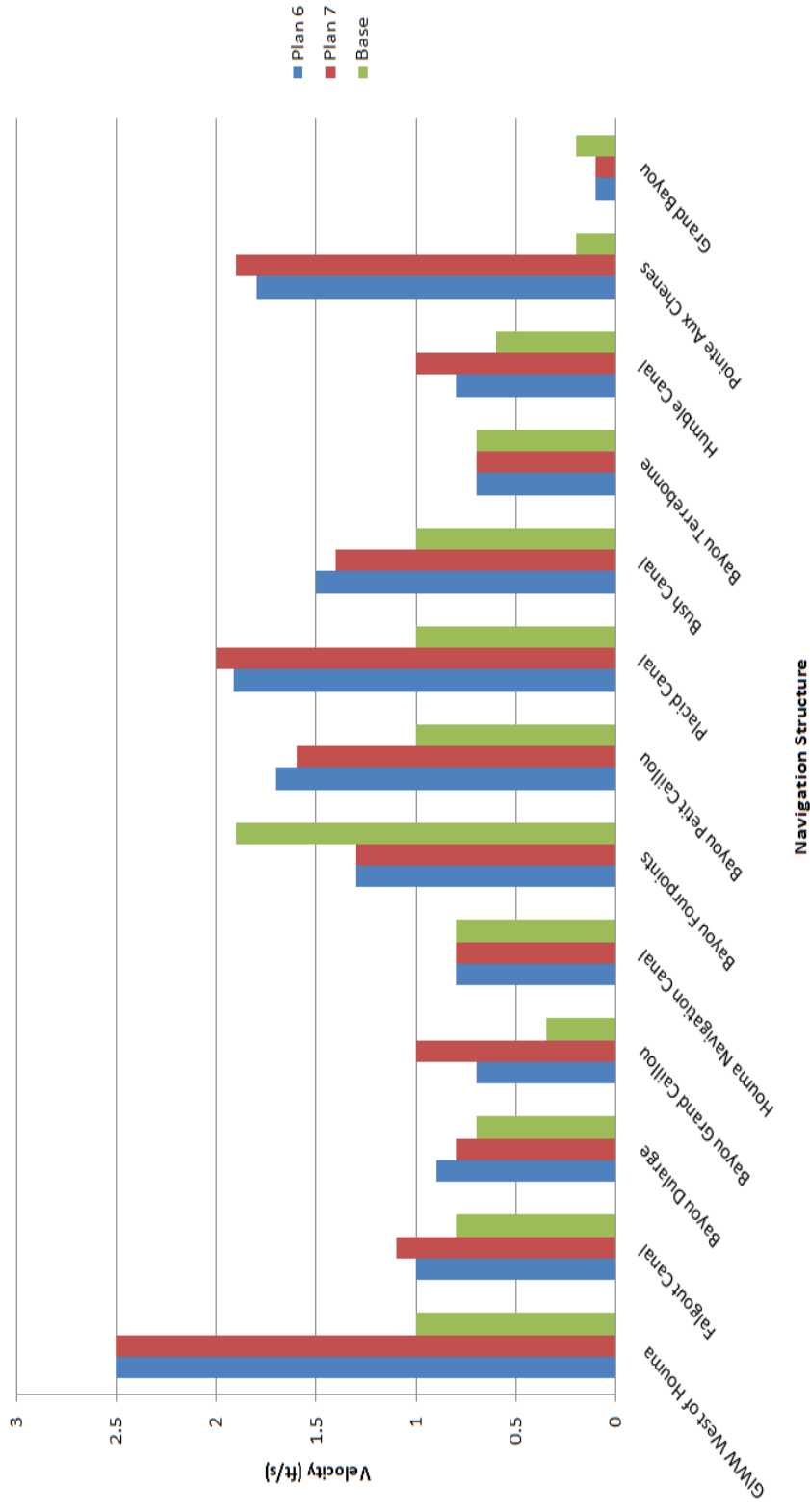
Figure 54 - 50 Percent Exceedance Velocities for Plan 6, Plan 7 and Base Model Simulations

10th Percentile Velocity Comparisons
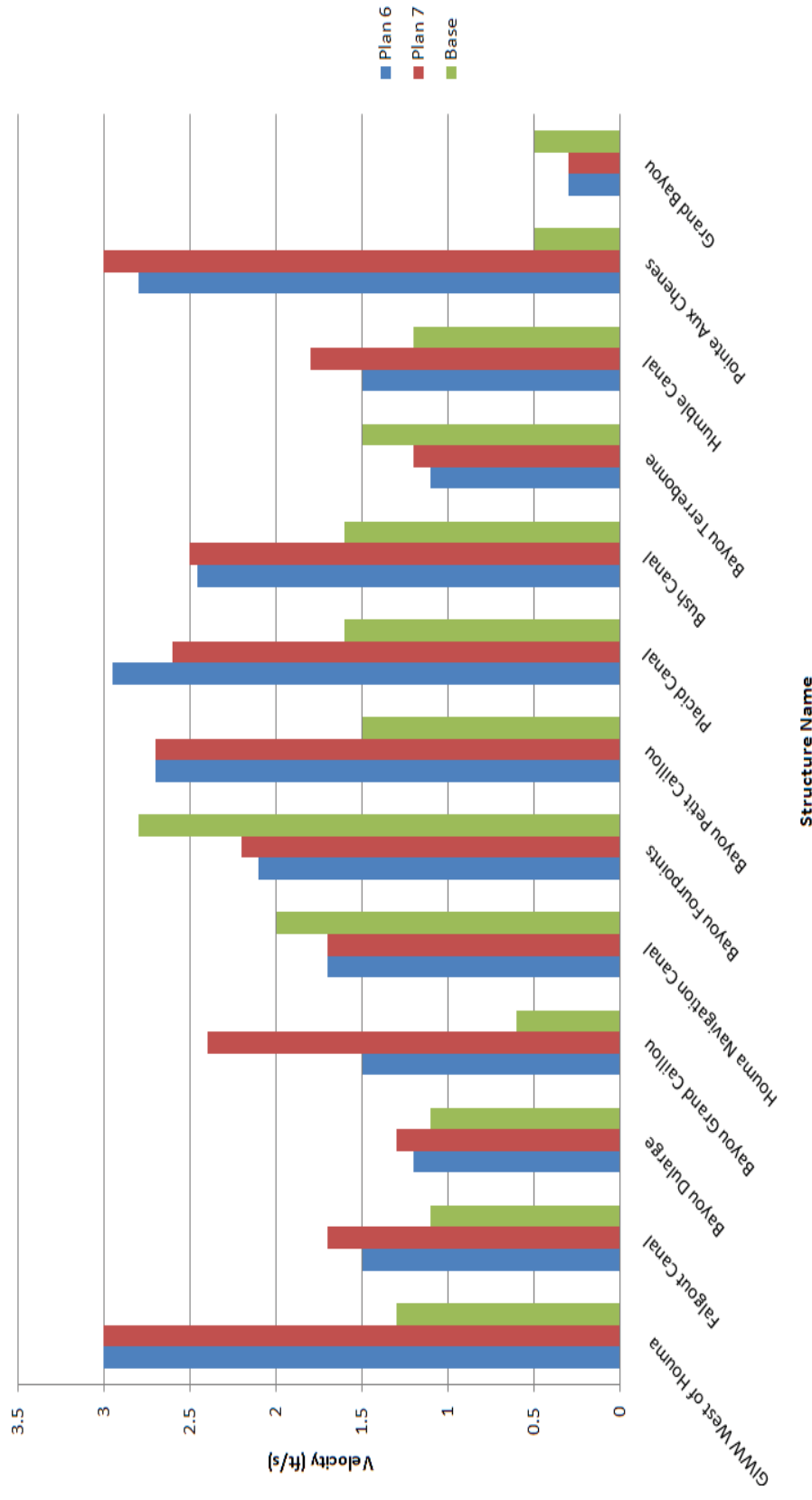
Figure 55 - 10 Percent Exceedance Velocities for Plan 6, Plan 7 and Base Model Simulations

Max Percentile Velocity Comparisons
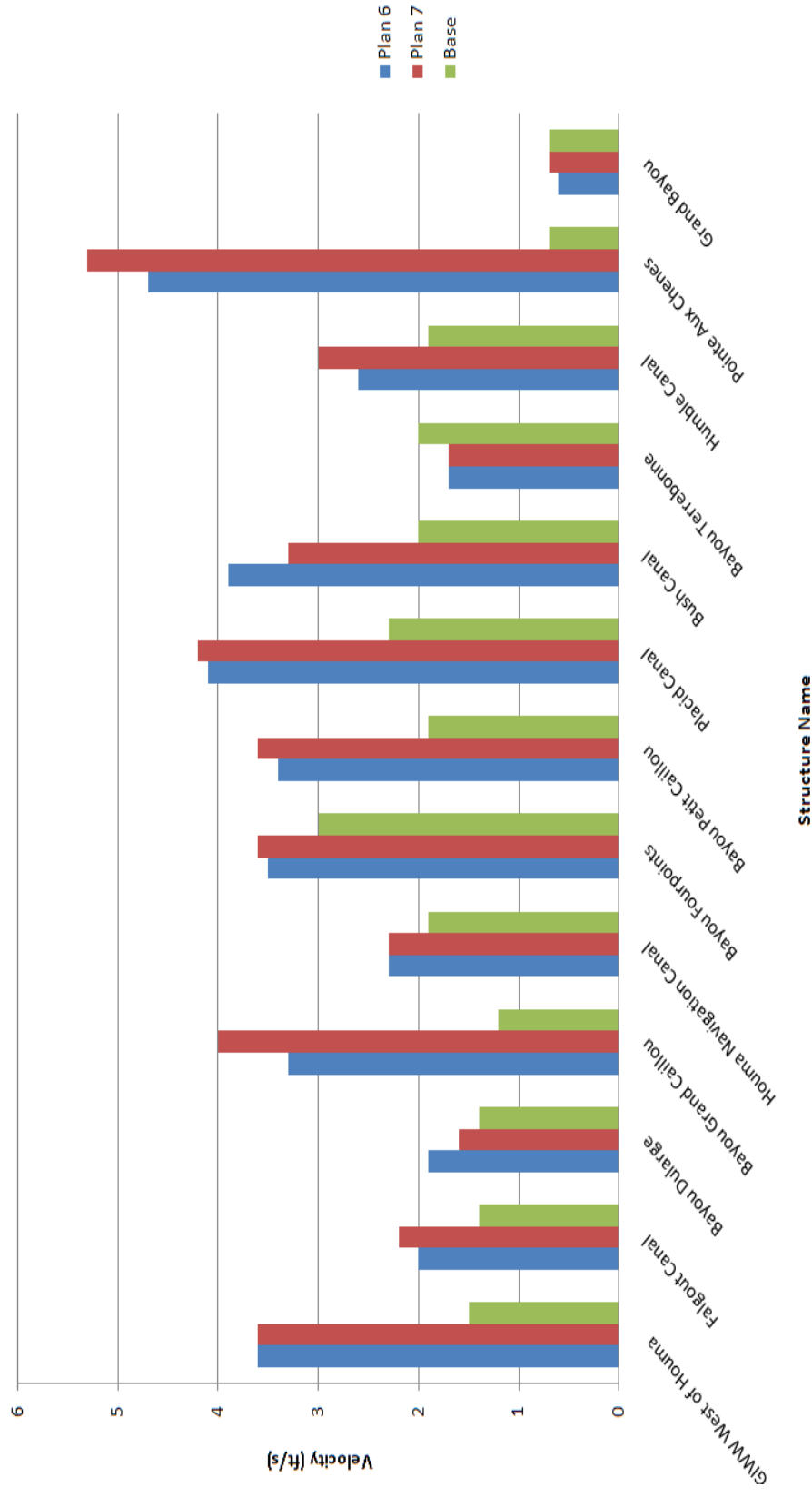
Figure 56 - Maximum Velocities for Plan 6, Plan 7 and Base Model Simulations

2.6.4.1.5 Conclusions

Some of the maximum velocities did reach as high as 3.5 ft/s or more but those were maximum velocities over an approximately three month time frame consisting of multiple frontal passages. The exceedance curves allow MVN the option of performing a cost to benefit analysis on select structures where the cost of increasing a particular structure size may not warrant the relatively minor decrease in the prevalent velocities. The results show that the changes associated with the Plan 6 and Plan 7 configurations produce minimal effects on the velocity fields, less than 1 ft/s change from the base, for the majority of the structures (Falgout Canal, Bayou Dularge, Houma Navigational Canal, Bayou Fourpoints, Bush Canal, Bayou Terrebonne, and Grand Bayou). The remaining structures (GIWW West of Houma, Bayou Grand Caillou, Bayou Petit Caillou, Placid Canal, and Pointe-aux-Chenes) have higher velocity increases but the 10th percentile exceedance velocities are below 3 ft/s for all navigational structures. This indicates that velocities above 3 ft/s would not be regular occurrences.

The 50th percentile exceedance velocities are below 2 ft/s for all navigational structures except the GIWW West of Houma structure (2.5 ft/s). These 50th percentile exceedance velocities indicate that on average the velocities for all the structures (except GIWW West of Houma at 2.5 ft/s) are below 2 ft/s. These values should be taken as an indication of the expected velocities for each structure location.

Previous salinity modeling considered configurations with all navigational structures open and all environmental structures closed. To ensure no significant changes occurred in the velocity fields due to the closure of these environmental structures, Plan 7 (Plan 6 with all environmental structures closed) was modeled. As expected, the Plan 7 model results produced minimal changes in velocity (less than ~0.3 ft/s). This indicates that the closure of the environmental structures for ecological/biological reasons should not have a significant impact on the velocities in the navigational structures.

2.6.4.2   TABS Modeling

2.6.4.2.1 Introduction

Subsequent to the ADH modeling, the U.S. Army Corps of Engineers Vicksburg District completed further analysis for the MTG floodgates using TABS - MDS with updated geometry. TABS-MDS (TABS – Multi-Dimensional Sediment) is the ERDC version of RMA-10 with sediment transport. TABS-MDS performs one-, two-, and three-dimensional shallow water hydrodynamic calculations with salinity transport coupled to the hydrodynamics (King 1988). Where structures were analyzed in both studies, the TABS -MDS modeling was used to determine the final gate sizing.

The TABS-MDS modeling incorporates two-dimensional system flow reproductions for open floodgate widths (for the above listed sites). These planned structures are part of the overall flood and storm surge risk reduction work in Terrebonne and Lafourche Parishes. These proposed structures will allow navigation passage through the current levee alignment, but are designed for closure during storm surges to protect Houma, La, and vicinity. Model results show resulting velocity flow fields across the proposed gate widths and head differential across the structure as well as combined system impacts from all structures in place. Further information is provided showing the effective flow field at the GIWW floodgates that develop with passing vessel/barge tow combinations. The loaded vessels (barge tows) passing through the proposed floodgates occupy a significant cross sectional area, thus causing a local velocity increase. Strategic boundary condition development and vessel presence simulations offer an accurate estimate of navigation conditions.

2.6.4.2.2 Model Development and Verification

Existing USACE model studies were reviewed, and a combination strategy of using the recently-completed simulations and bathymetry from other studies was compiled to build the subject floodgate simulation model in both a cost-effective and quality fashion. Major components originated with an ERDC TABS MD study (Figure 57), an MVN modeling effort for the Barataria Basin (Figure 58), and parts of a coastal Louisiana ADCIRC model. Bathymetry, boundary conditions, and proposed gate location site flows were gleaned from these existing studies, and also from new field data sets at the proposed gate locations. The existing model verification efforts from other studies offered an unusually high level of quality assurance since the floodgate sites were along the GIWW and Houma Navigation Canal vicinities, where verification efforts, technical review, and USACE approval were either complete or nearly complete for other modeling projects, namely the ERDC TABS MD study and the Barataria Basin study. Model development began with merging two versions of ERDC models that cover the GIWW, Houma Navigation Canal (HNC), and Atchafalaya Bay. The resulting mesh/domain also includes added bathymetry in the areas north of Larose, LA, to improve the simulations at the GIWW Larose floodgate site.
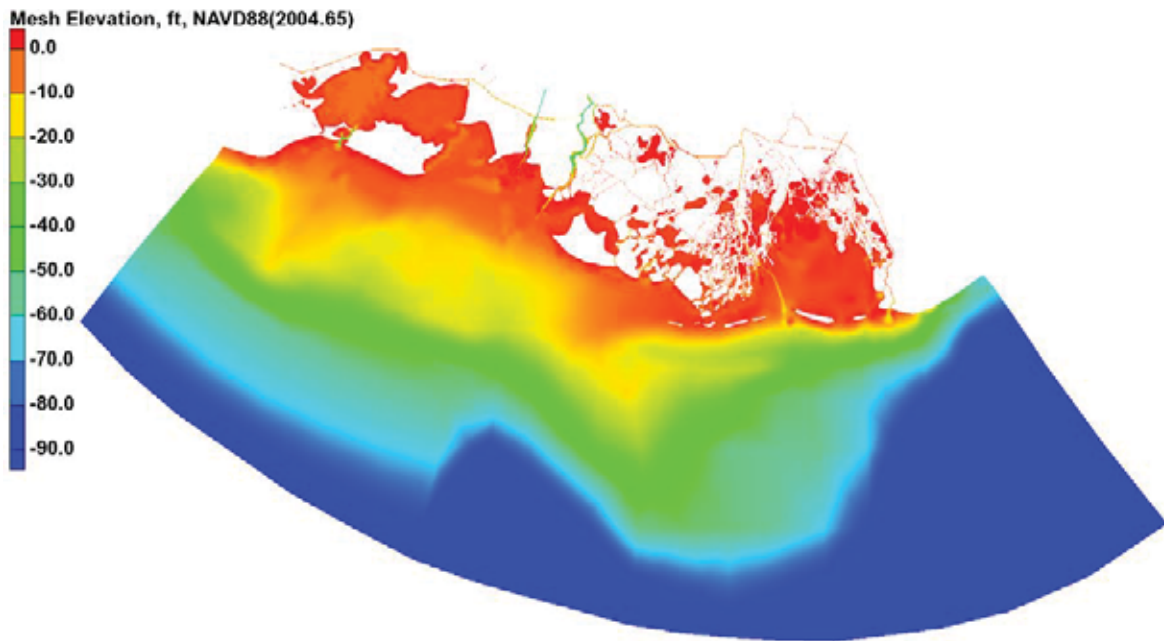
Figure 57 - ERDC TABS MD study model domain (McAlpin, 2010)
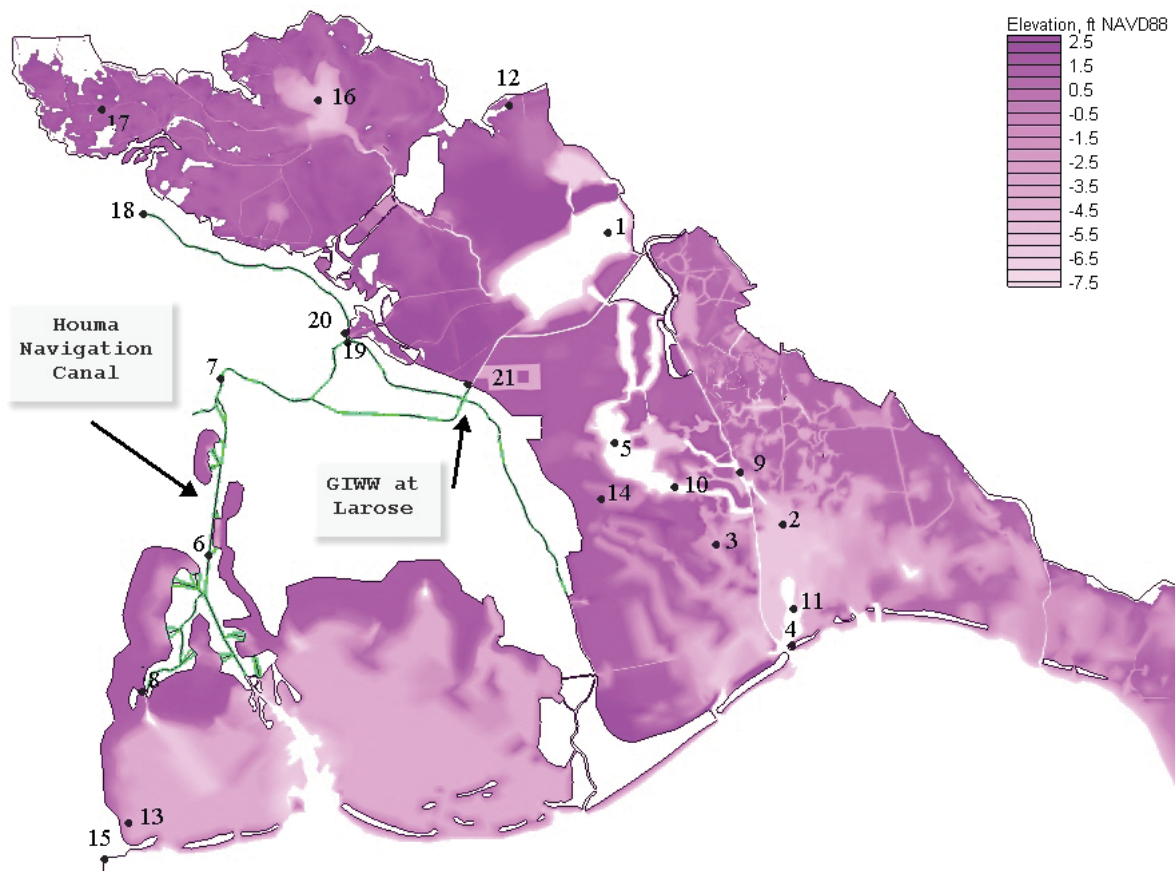
Figure 58 - MVN Barataria Basin Model Domain showing Key Locations of Floodgate Model Flow Compatibility

Model development and verification included 3 phases. Phase 1 began with studying the time-series simulations from the ERDC TABS MD. Phase 2 evolved a steady state approach using design flows, and Phase 3 consisted of a stage-discharge verification effort using field data. Ideally and often necessarily, field data is available at the beginning of a model study, but sufficient existing model development and simulations were already completed, and previous work was considered minimally adequate for the floodgate evaluations. Thus the Phase 2 steady-state development served as the actual model verification. Phase 3 field data arrived after gate-width simulations were completed, but was included as a comparative and qualitative verification effort.

Phase 1 - The time-series simulations for the ERDC model were reviewed for a 6-month period that ran from January to June in 2004. Details on the time-series simulations can be furnished upon request. The proposed gate locations are identified, and model data were extracted along the channel centerline. Table 64 highlights the highest velocity magnitude at each floodgate location for the 6 month period.

Table 64 - Highest Flow Velocities at the Proposed Floodgate Locations

Proposed Site	Velocity, fps
GIWW at Larose	0.75
Grand Bayou	0.75
Minors Canal	*
GIWW at Houma	1.7
Falgout Canal	2
Bayou DuLarge	2.2
Bayou Grand Caillou	0.75

*The Minors Canal site was added after Phase 1 evaluations and is addressed in the following phases of verification.

Velocity magnitudes were used to determine acceptable gate widths for navigation in this study, and water surface elevations across the simulated gate constrictions are also reported. The New Orleans Westbank (WBV) project delivery team established navigable velocity criteria of up to 3 mph (4.4 fps) with much input from the navigation industry. This same criteria was applied to the navigable MTG GIWW project sites. The gate designs were based on meeting two conditions: 1) providing enough width for vessel passage and, 2) providing enough flow cross sectional area at the gate sites so that the constricted flow velocities would neither exceed the navigation criteria nor result in channel scouring problems. The Phase 1 data review (Table 64) indicated that existing condition velocities were generally low, and that controlling width may be a function of vessel width requirements at most of the sites. MVN strategy for storm surge and flood risk reduction improvements where gates were required was to design adequate vessel passage and augment flow passage with adjoining sluice gates as needed. This cost-effective approach was applied to the proposed MTG floodgates.

<u>Phase 2</u> - A tow navigating through one of the planned floodgates would take only a few minutes, and no significant changes in channel stage or flow would be encountered during the short passage time. The short time interval associated with a passing vessel was the basis for a steady-state flow condition that represented maximum navigation difficulty. (*The passing vessel would generate additional navigation issues because its presence would further reduce cross-sectional area at the floodgate constriction,* and this important effect is evaluated in Part III of this report.) In addition to the ERDC TABS-MD study, two additional sources of information were available to define stage-discharge relationships at MTG floodgate sites, the 2-dimensional MVN Barataria Basin model (Teeter, 2010), and USGS stage and flow data (USGS Professional Paper 1672). Like the subject floodgate model, the Barataria Basin model was also developed using the RMA2 finite element code, and it offered a comprehensive evaluation of flow in the GIWW at Larose, LA. Flows through the Houma Navigation Canal were based on the USGS data set.

Model development for the 7 proposed gate sites included grouping the gate sites by region. The northern (roughly east-west) gate sites (Figure 59) along the GIWW and vicinity include Minors Canal, GIWW at Houma, GIWW at Larose, and Grand Bayou. Three additional floodgate sites (Figure 60) were evaluated along channels that connect to the mid- to lower-Houma Navigation Canal (HNC), and these are presented as the Mid-HNC sites. Since the proposed sites were up to 25 miles apart, initial strategy included verifying that a given steady state event simulation could define the most difficult navigation conditions at all 7 proposed sites. The study goal was to reproduce high flow and velocity combinations that provide the highest level of difficulty to navigation for the open gate conditions.

Table 65 lists the probability-based discharges on the GIWW at Larose, LA, in the eastward flow direction using USGS data and existing model boundary condition development for the MVN Barataria Basin Model Study.

Table 65 - GIWW flows at Larose, LA (eastward)

Flow, cfs	Probability, %
5335	99.9
4327	99
3430	95
2920	90

The GIWW flow at Larose, at times, reverses direction, but stage and discharge data show that when flows move westward at this site, magnitudes are much lower than the more common easterly flows. USGS records (1997-1999) show the maximum recorded westward flow here at 1860 cfs (Prof. Paper 1672). This same reference lists median eastward flow at 2670 cfs with a maximum measurement of 4,930 cfs.

Figure 59 - Floodgate sites along the GIWW
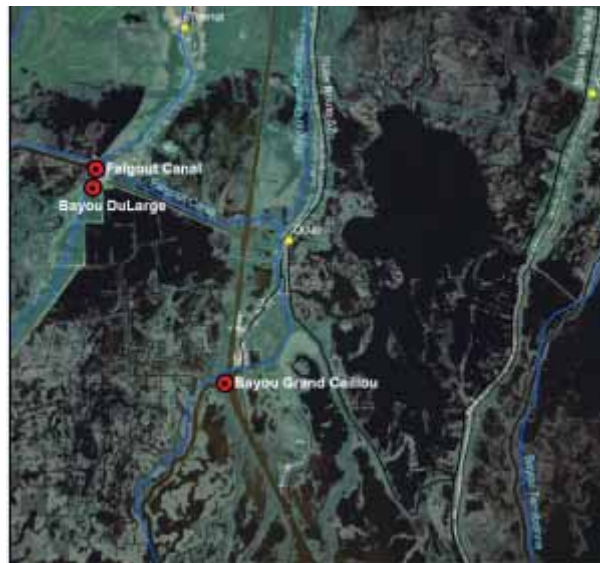
Figure 60 - Floodgate sites along the mid-Houma Navigation Canal vicinity

The USGS data maximum for the 1997-1999 records falls between the 99 % and 99.9% values in Table 65. Model boundary condition adjustments for the maximum eastward flow at Larose resulted in a model simulation flow of 4800 cfs. This value was very close to the 99.9% probability of not exceeding, and was selected as the Larose site design flow. The Barataria Model flow comparison to the USGS data added consistency and confidence that the flow maximums in the USGS data set generally represent the highest level of difficulty for navigation.

The USGS recorded flows at the proposed gate site in the GIWW West of Houma range from 4980 to 11800 cfs. Adjusting the Amelia LA inflow (within its recorded range) to achieve the design flow at Larose resulted in a flow of 12,100 cfs at the GIWW Larose Site. This was slightly higher than the recorded maximum, but was considered roughly equivalent and completed the design flows for the Northern Gate Sites. No USGS data were available at the proposed Grand Bayou and Minors' Canal sites, and the design

flow selections at these sites are addressed in the Phase 3 effort below.

The USGS discharge information on the HNC near Dulac, LA, ranged from 4200 to 13,700 cfs over the 1997-1999 record. This data was near the remaining 3 mid-HNC floodgate sites, but model simulations indicate the major driving forces for these sites are from the West, more influenced by Atchafalaya River flows and conveyance from the Atchafalaya to the southeast and east directions along the interconnected canals and bayous including Bayou Penchant. The mid-HNC gate sites are also addressed in the following Phase 3 section.

Simulation strategy began using stage and discharge conditions from the ERDC TABS MD model and adjusting the GIWW inflow from the model inflow boundary condition at Amelia, LA, until the design discharge was achieved at the Larose, LA, outflow boundary. USGS data indicated that the flow at Amelia can move north or south, but that the higher GIWW discharges eastward approaching Houma and eventually at Larose occur when the Amelia flow is south into the GIWW. USGS average flow eastward approaching Houma is listed at 5700 cfs. The resulting HNC discharge at Dulac was 8400 cfs. This event was used as the base and plan condition for the proposed gate configurations. A low Gulf tidal boundary (-0.5 ft) was applied with the design flows to account for increases in velocities due to tide. Figure 61 shows the initial Floodgate model, and Figure 62 shows the revised (reduced) domain where the Gulf of Mexico boundary was moved inland to improve steady state water level agreement at the lower HNC.
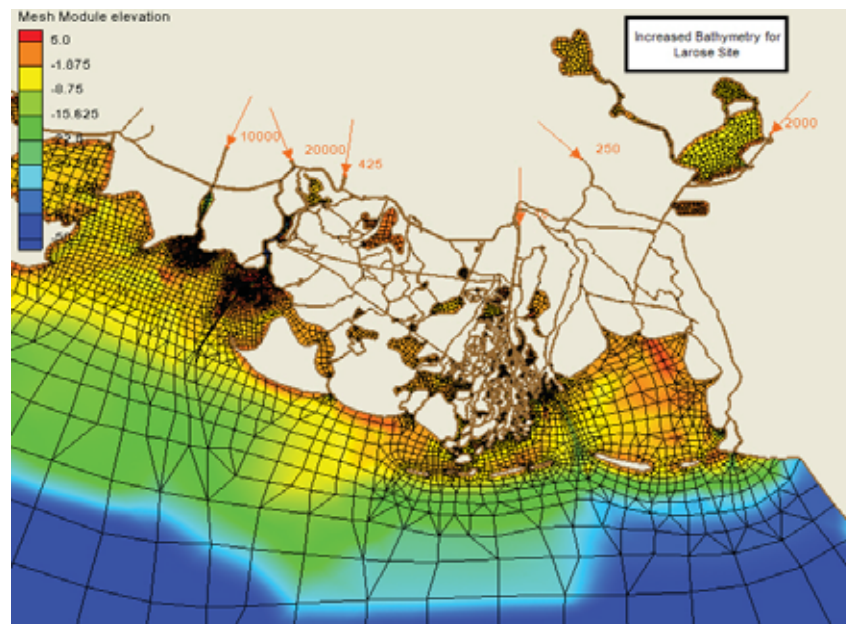
Figure 61 - Initial Floodgate Model Domain
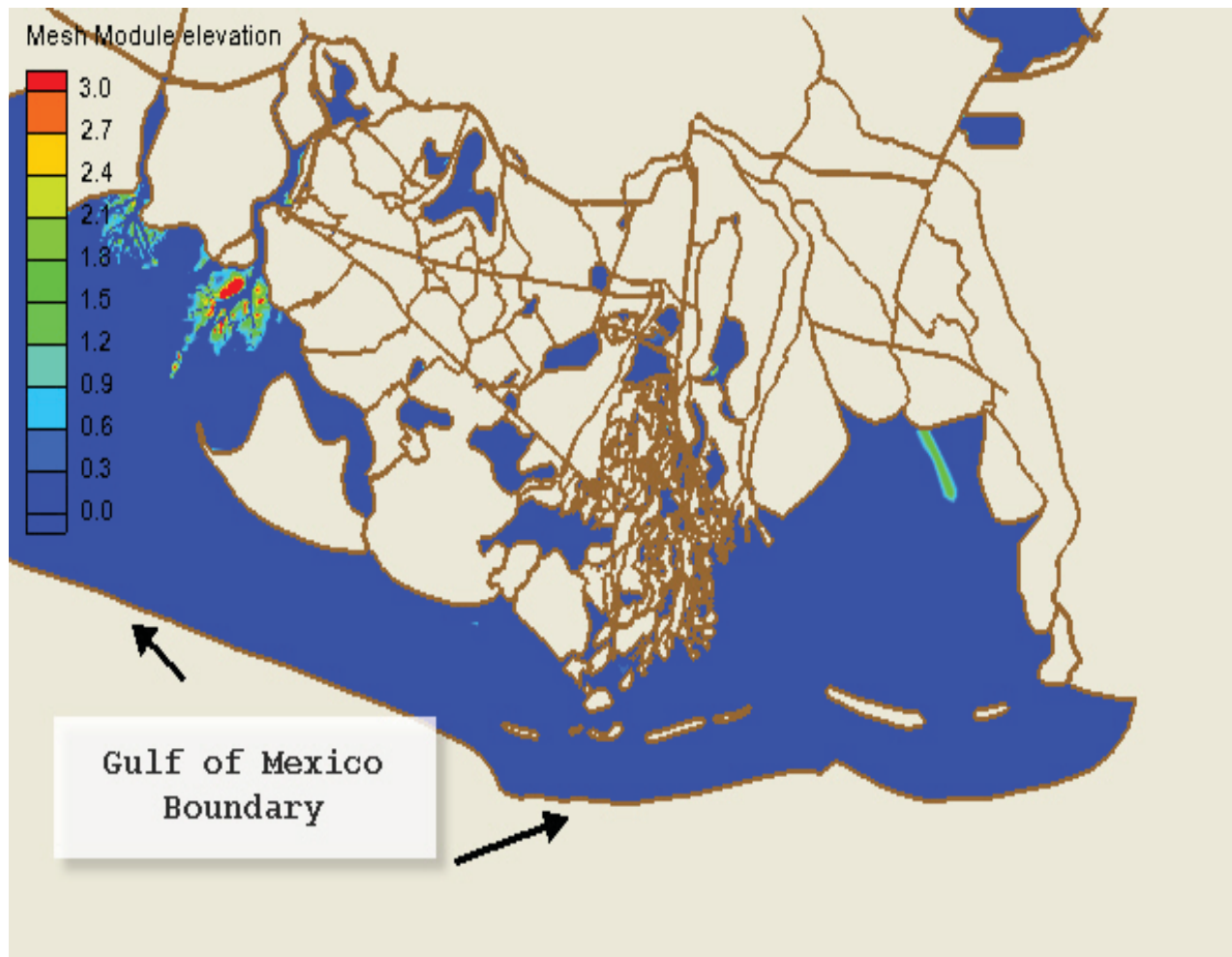
Figure 62 - Model Domain showing Gulf Boundary

Phase 3 - The field data set arrived after scheduled draft report submission, but offered a comparison for the selected design flows. Like the Minors Canal and Grand Bayou sites, the mid-HNC sites were compared to the Phase 1 ERDC model peak values and to the June 2010 ERDC field data. Table 66 summarizes the design flow and velocity at each proposed floodgate site with the comparable peak flow and velocities from the Phase 1 ERDC model study and the recent June 2010 Field Data set peak flow and velocities. Table 66 shows both reasonable agreement between some of the peak stage and velocity design values and differences at others. Noteworthy is the complex network of large river, canal, and bayous with tidal exchanges in the project area.

Table 66 - Floodgate Model Design Flows compared to Existing Model and Field data Peak Flow and Velocity data

Location	Floodgate Model Design Flow		ERDC Phase 1		ERDC Phase 3 Field Data	
North Gate Sites	Discharge (CFS)	Velocity (ft/s)	Peak Discharge (CFS)	Peak Velocity (ft/s)	Peak Discharge (CFS)	Peak Velocity (ft/s)
GIWW @ Houma	12132	1.09	15400	1.70	8632	1.84
GIWW @ Larose	4797	0.78	6410	0.75	7706	1.48
Minors Canal	150	0.30	*	*	*	*
Grand Bayou	379	0.58	678	0.75	677	0.62
Mid - HNC Gate Sites						
Falgout Canal	2632	0.91	3534	2.00	6021	1.71
Bayou Dularge	819	1.30	603	2.20	603	0.47
Bayou Grand Caillou	2052	0.51	*	0.75	*	*

* No data at the proposed gate site.

Part II- Floodgate Simulations

General-   The hydraulic effects of the proposed floodgates to navigation were evaluated using the flow magnitude created by the physical constriction at the floodgates and the head differential across the structure as defined by water surface elevations. The 2 dimensional modeling completed offers an accurate assessment of flow impacts through the gates using the design flows. The 7 floodgate sites are presented in Figure 63 - Figure 76, showing the design flow field with the gate or gate/sluice gate configurations as defined at MVN. Table 67 defines the gate dimensions and configurations simulated, and Table 68 shows the existing Condition and With Gate or Gate + Sluice Gates velocities. Gate simulations were completed with sluice gates open and with sluice gates closed.

Table 67 - 4 Floodgate and Sluice Gate Configurations

Location	Gate Width (ft)	Sluice Width (ft)	Depth (ft)
GIWW @ Houma	2 X 125 = 250	None	-16
	Single 175	4 @ 25 ft each	-16
Minor's Canal	Single 56	None	-9
Falgout Canal	Single 56	3 @ 46 ft each	-9
Bayou DuLarge	Single 56	None	-7
Bayou Grand Caillou	Single 56	3 @ 46 ft each	-12
Grand Bayou Canal	Single 56	3 @ 46 ft each	-9
GIWW @ Larose	Single 125	None	-16
	Single 175	2 @ 25 ft each	-16
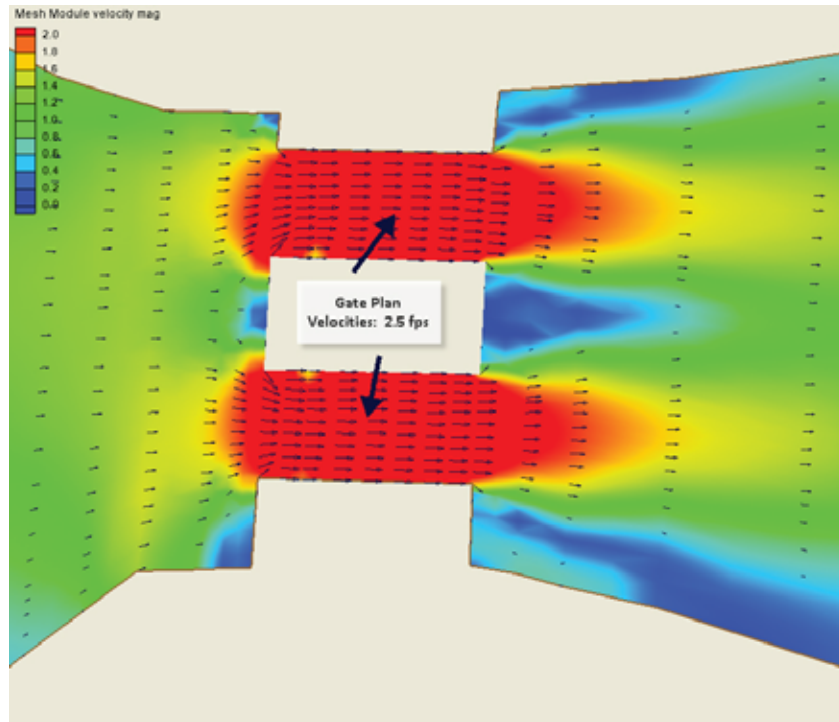
Figure 63 - GIWW at Houma with two 125 ft wide sector gates, no sluice gates
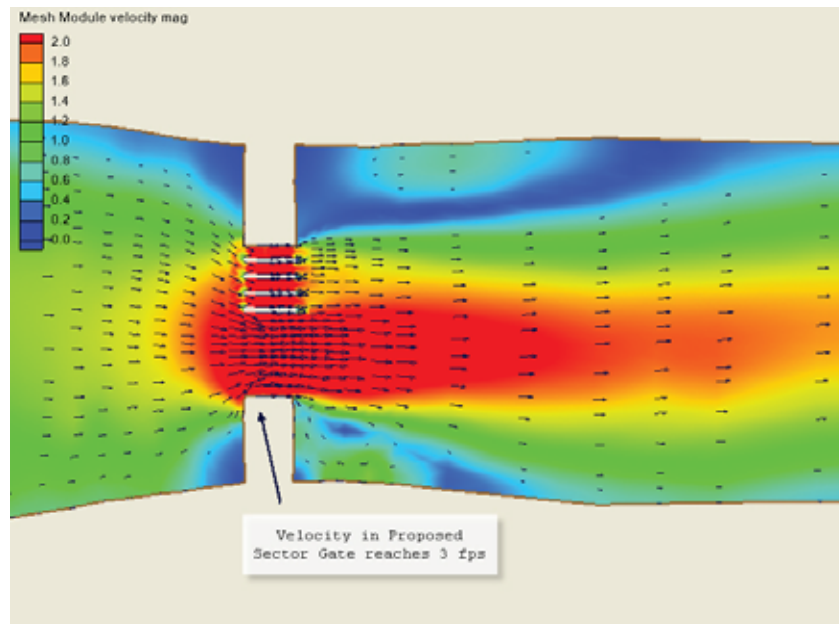
Figure 64 – GIWW at Houma for single 175 ft sector gate with four, 25 ft wide sluice gates
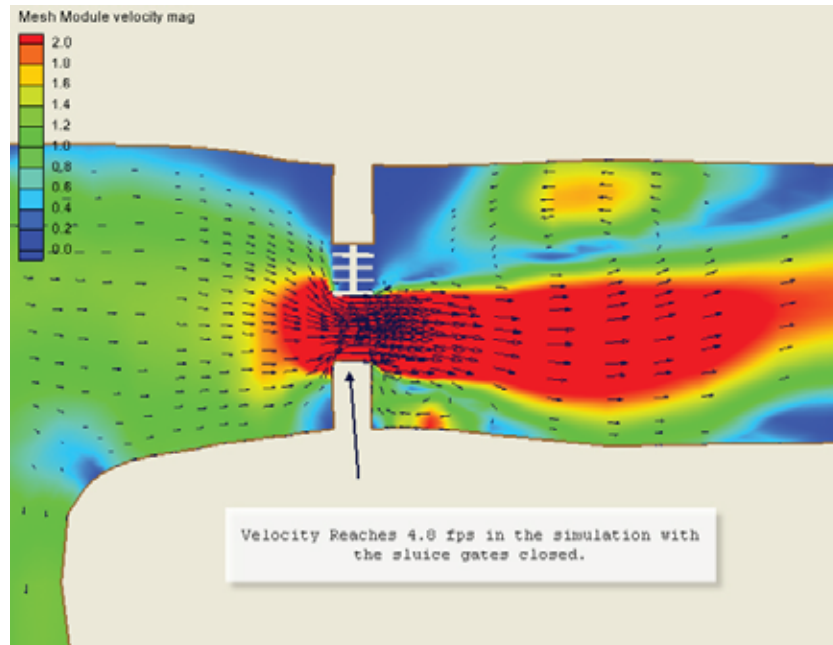
Figure 65 - GIWW at Houma for single 175 ft sector gate with four, 25 ft wide sluice gates closed
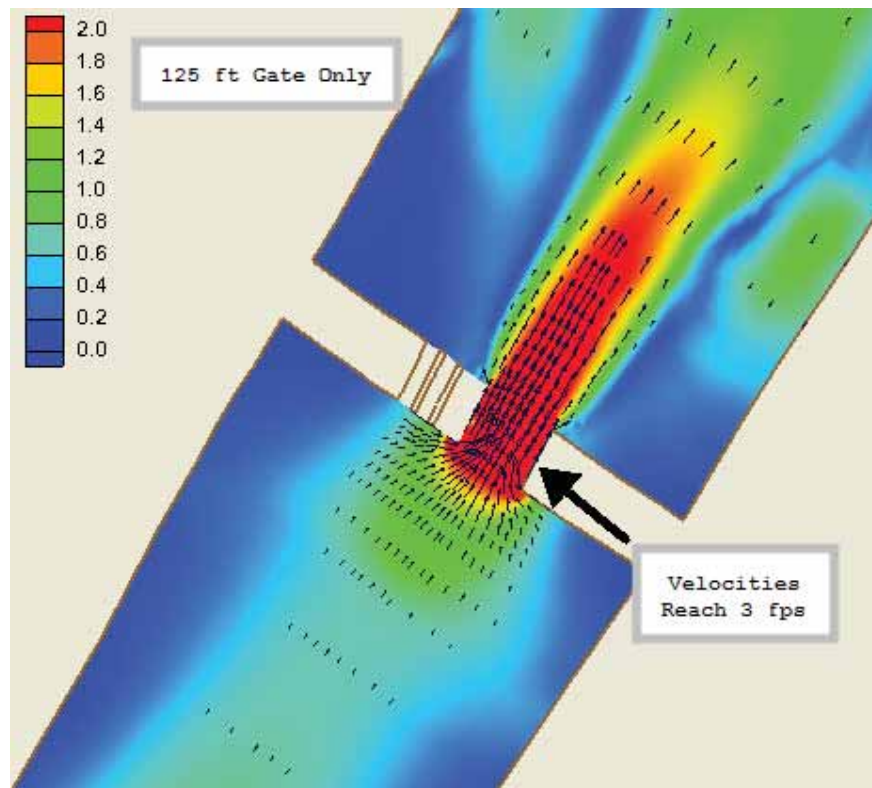
Figure 66 - GIWW at Larose for single 125 ft wide sector gate
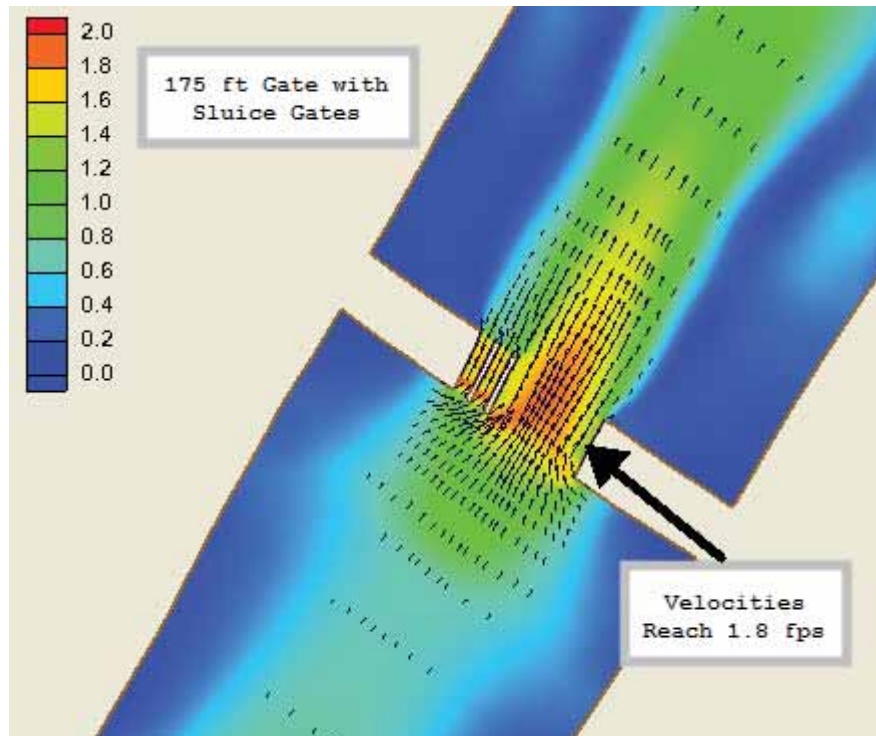
Figure 67 – GIWW at Larose for single 175 ft wide sector gate with two, 25ft sluice gates
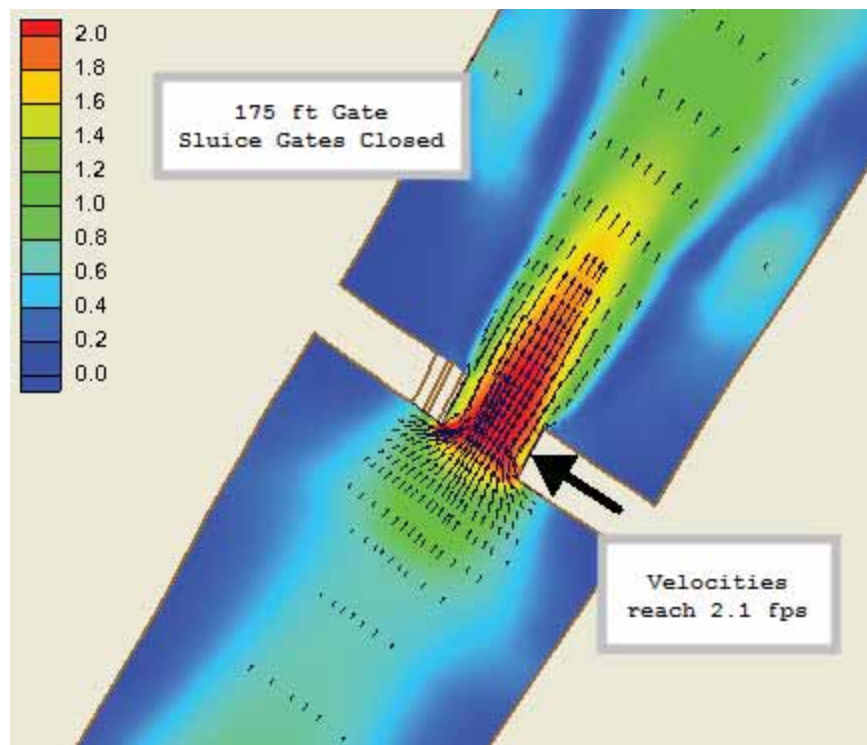
Figure 68 - GIWW at Larose for single 175 ft wide sector gate with two, 25ft sluice gates closed
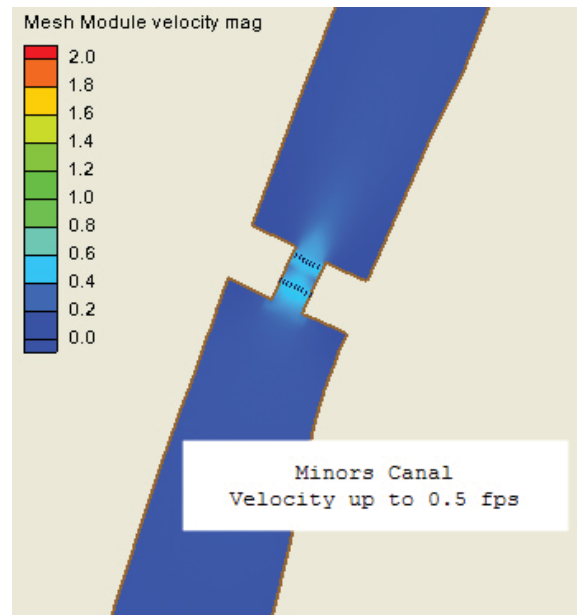
Figure 69 - Minors Canal with single 56 ft wide sector gate, no sluice gates
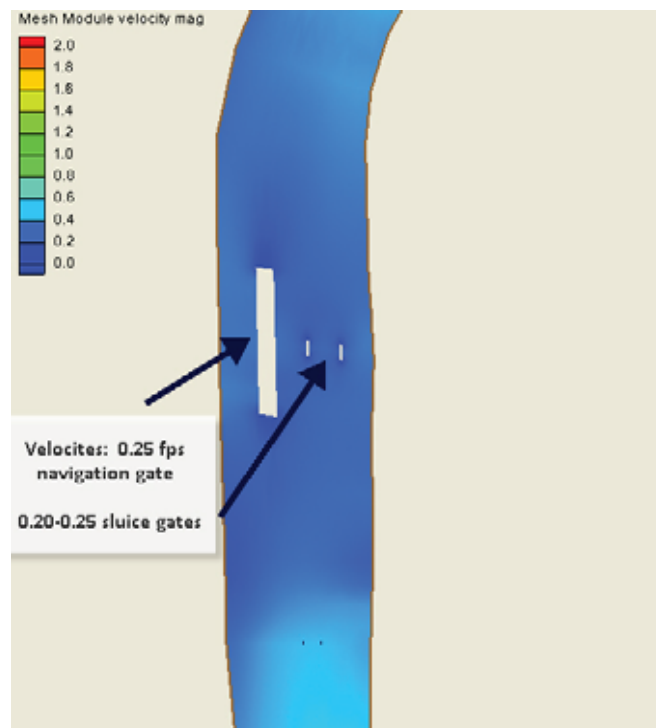
Figure 70 - Grand Bayou with single 56 ft wide sector gate, three 46 ft wide sluice gates
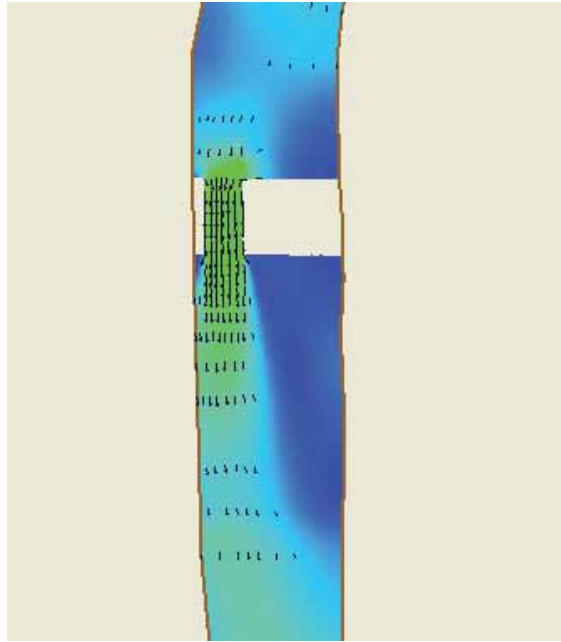
Figure 71 - Grand Bayou with single 56 ft wide sector gate, three, 46 ft wide sluice gates closed

Falgout Canal
GATE: 1.8 fps
Sluices: 2.0 fps
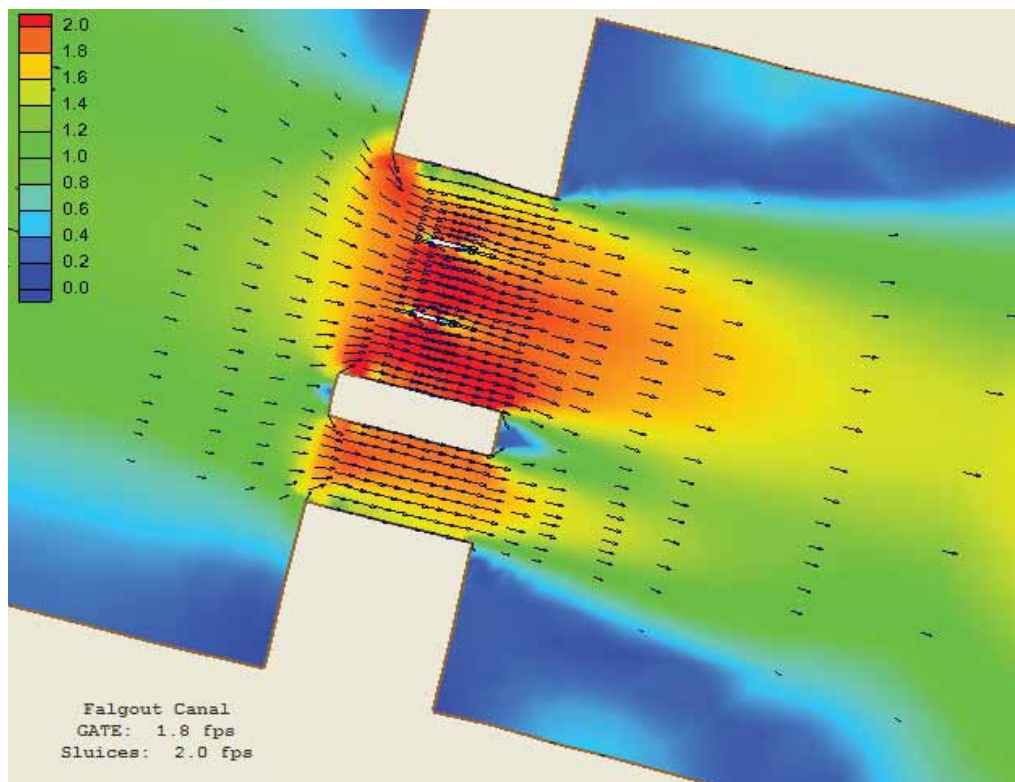
Figure 72 - Falgout Canal with single 56 ft sector gate, three 46 ft wide sluice gates open
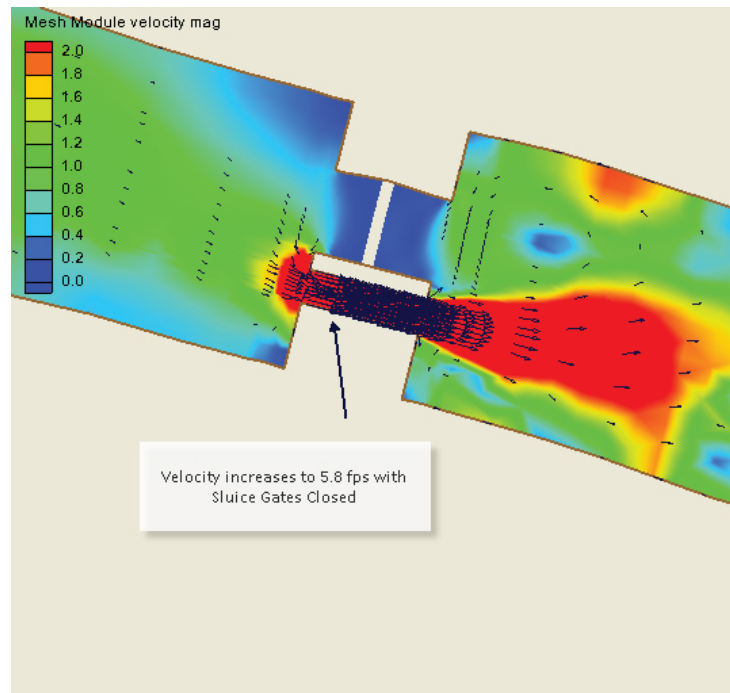
Figure 73 - Falgout Canal with single 56 ft sector gate, three 46 ft wide sluice gates closed
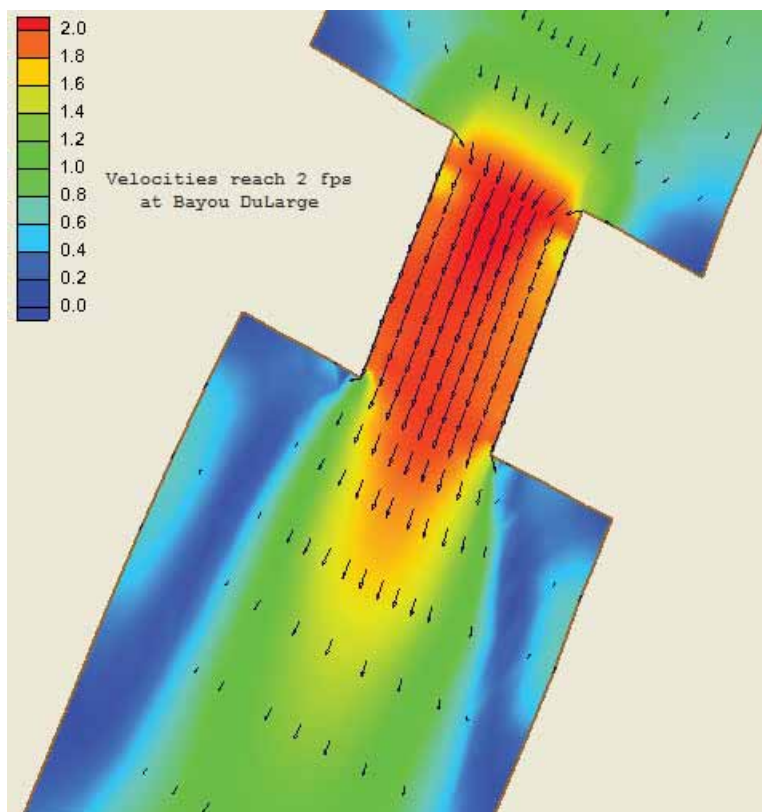
Figure 74 - Bayou Dularge with single 56 ft sector gate, no sluice gates
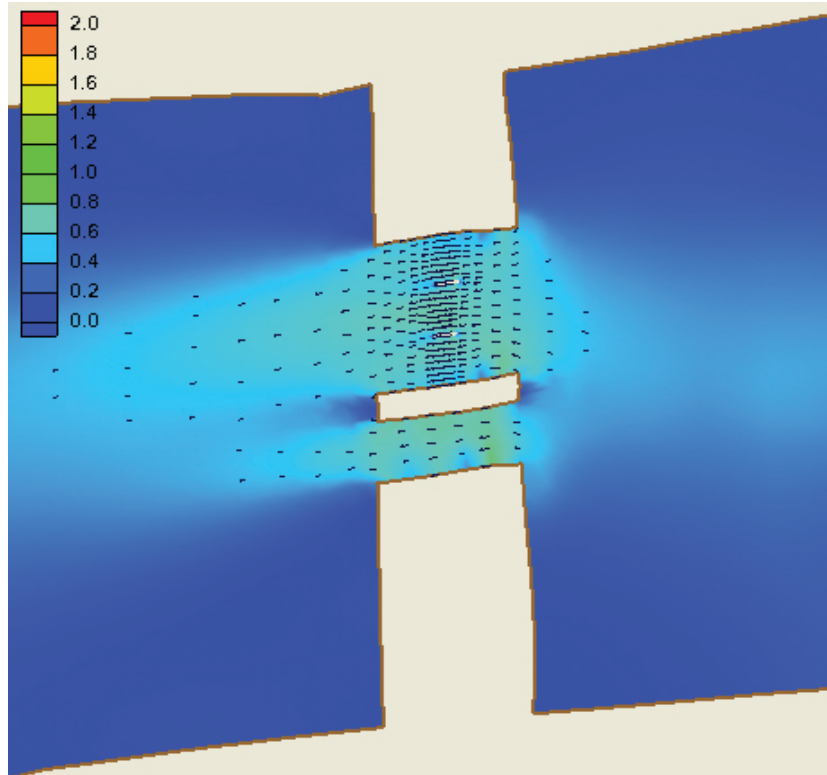
Figure 75 – Bayou Grand Caillou with single 56 ft wide sector gate, three 46 ft wide sluice gates
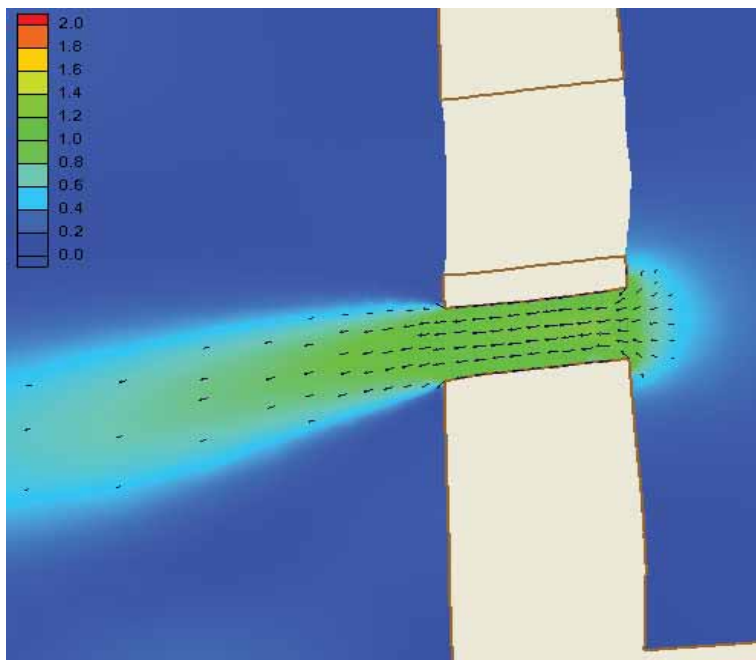
Figure 76 - Bayou Grand Caillou with single 56 ft wide sector gate, three 46 ft wide sluice gates closed

Table 68 - Existing Condition and With Gate or Gate + Sluice Gates

Location	Model Velocities, FPS	
North Gate Sites	Existing Condition Velocity, fps	Open Gate/Open Sluice gate velocities
GIWW @ Houma- Two 125 ft Sector Gates only Single 175 ft Sector with 4-25 ft sector gates Open Single 175 ft Sector with 4-25 ft sector gates Closed	1.09	2.5 3.0 4.8
GIWW @ Larose- 125 ft Navigation gate only 175 ft Navigation gate 175 ft Navigation gate + 2 sluice gates	0.78	3.0 2.1 1.8
Minors Canal	0.29	0.5
Grand Bayou- 56 ft Navigation Gate only 56 ft Navigation Gate + 3 sluice gates	0.58	1.1 0.5
Mid - HNC Gate Sites		
Falgout Canal- 56 ft Navigation Gate only 56 ft Navigation + 3 sluice gates	0.91	5.8 1.8
Bayou Dularge	1.30	2.0
Bayou Grand Caillou 56 ft Navigation Gate only 56 ft Navigation Gate + 3 sluice gates	0.56	1.1 0.8

Details on the plan view floodgate existing condition versus floodgate and floodgate/sluice combinations be furnished upon request. Proposed floodgate simulations logically cause an increase in velocity over existing condition velocities. However, Bayou Grand Caillou and Grand Bayou show lower gate plan constriction velocities than their existing condition. This occurred because the gate sill elevation for the gate/sluice plans at these sites is deeper than the surrounding channel bottom elevations, increasing channel area and slowing velocities.

Part III Vessel Presence Velocity Accelerations.

Velocity Acceleration Problem - When a vessel passes a waterway constriction such as one of the proposed floodgates, additional impacts may be significant. The moving vessel occupies a cross-sectional area as it passes the gate constriction, and the additional loss of cross section further increases velocities. In wide, deep waterways, the impact of vessel presence is often negligible. But in the GIWW, a loaded barge tow can occupy an additional 30 percent or more of the channel cross section at the GIWW floodgate sites.

Evaluation Methodology - In the adjacent eastern sections of the GIWW where storm surge and flood improvements are ongoing along the New Orleans Westbank Western Closure Complex (WCC), similar vessel presence velocity accelerations posed a problem for planned ship simulator evaluations. The method for defining vessel presence accelerations was developed in the 2 dimensional model work for the WCC. The largest of 3 design vessel configurations used at the WCC was used to define the velocity fields at the 2 GIWW gate sites, Larose and Houma. Figure 77 schematizes the flow block concept. Cross sectional area was calculated and applied as a full-depth input. The flow block cross sectional area was equivalent to the design vessel, but was narrower than the actual vessel. This adaptation was necessary because RMA2 cannot evaluate pressure flows under a vessel, thus the area adjustments were made as a full channel depth block.
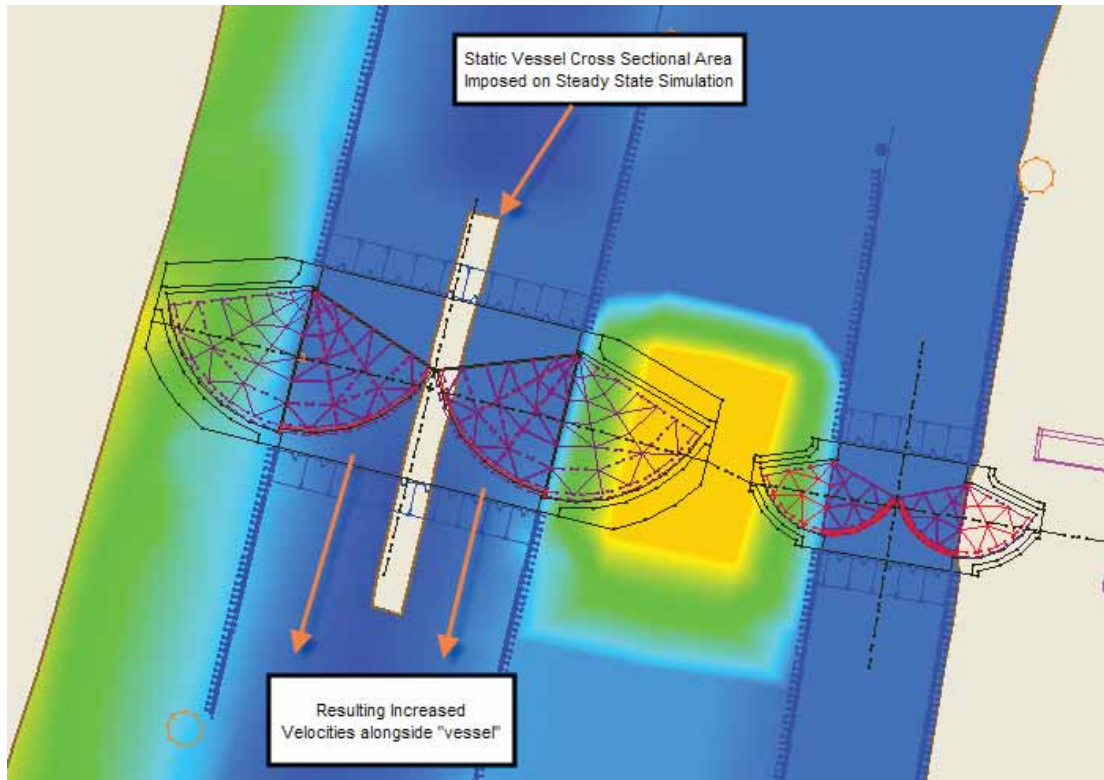
Figure 77 - Vessel Presence Velocity Acceleration schematic

Accuracy of Vessel Presence Velocity Accelerations - The WCC benefitted from a navigation physical model that re-produced certain numerical model simulations. The 2 dimensional flow block velocity strategy defined vessel presence increases, removed the flow block and ramped up discharge to match the flow block velocities. Flow velocities recorded along-side the model tow vessel in the physical model matched the 2 dimensional "flow block" velocities within 0.1 fps. Though no MTG physical model values are available for comparison, this strategy shows a reasonable estimate of the velocity increase in the GIWW as a vessel passes. Figure 78 and Figure 79 show the accelerations expected for 70 ft wide tow drafting 10 ft at the GIWW Houma and Larose gate sites. No significant water surface elevations were noted due to the proposed floodgate configurations. Figure 80 shows a typical minor local water surface elevation change at the GIWW Larose location. Table 69 displays a few selected water surface elevation changes at a few locations, with the greatest differentials reaching only a tenth of one foot.
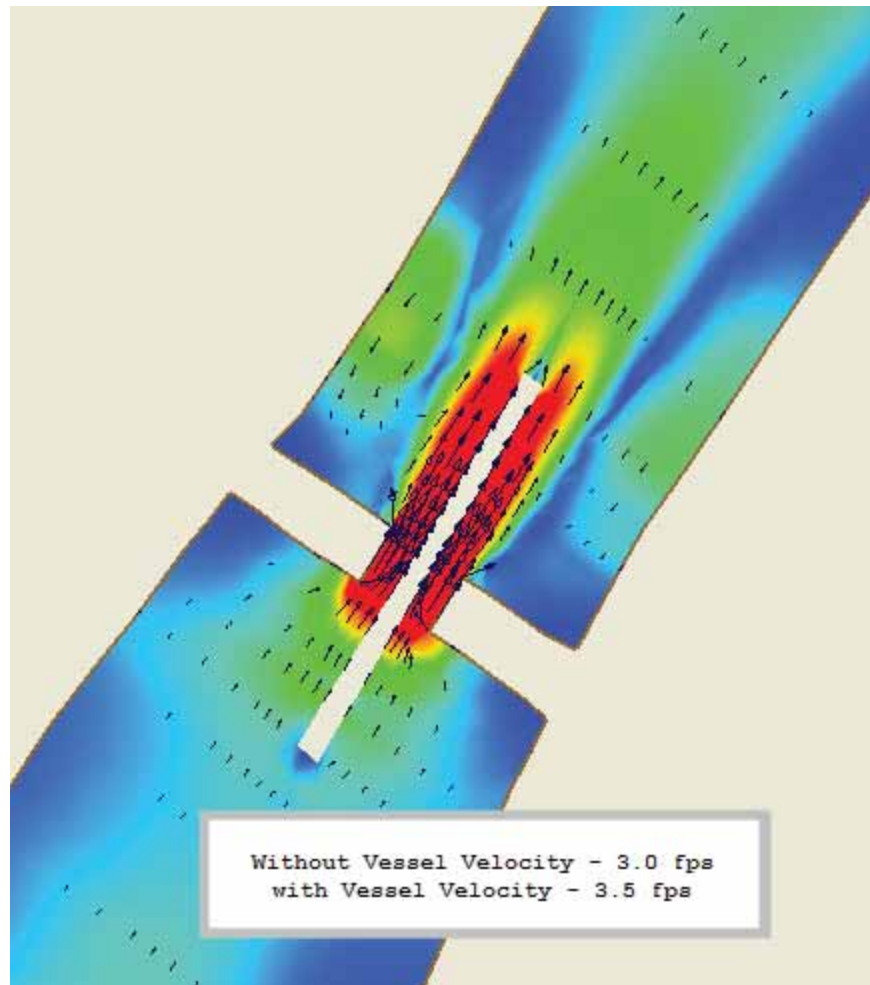
Figure 78 - GIWW at Larose Vessel Presence Velocity field
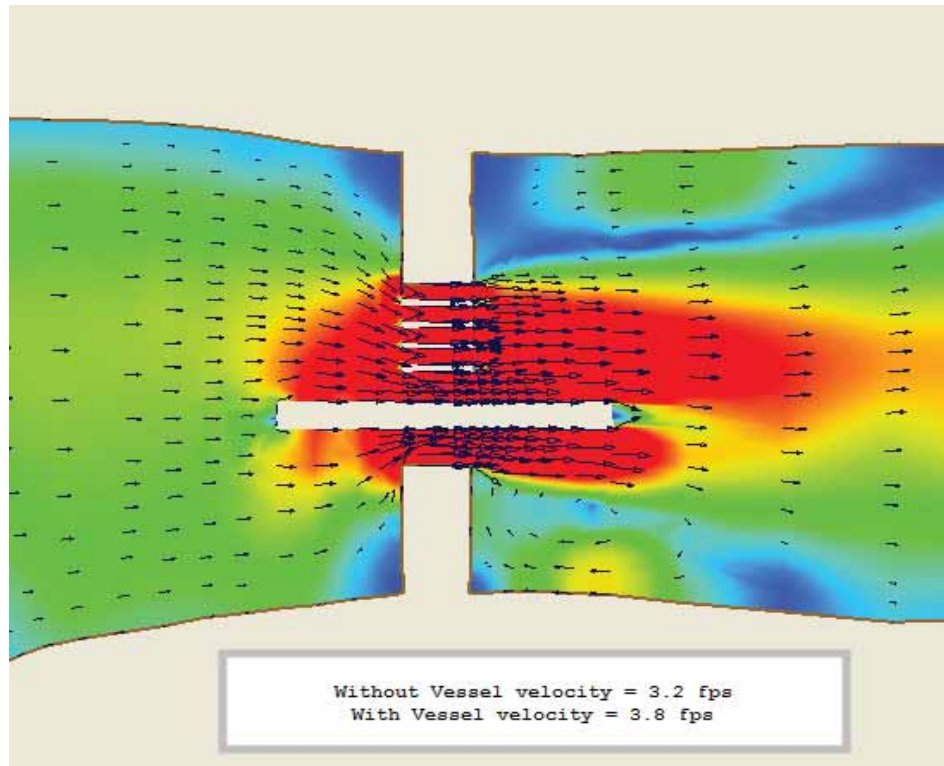
Figure 79 - Vessel Presence Velocity Field at GIWW Houma
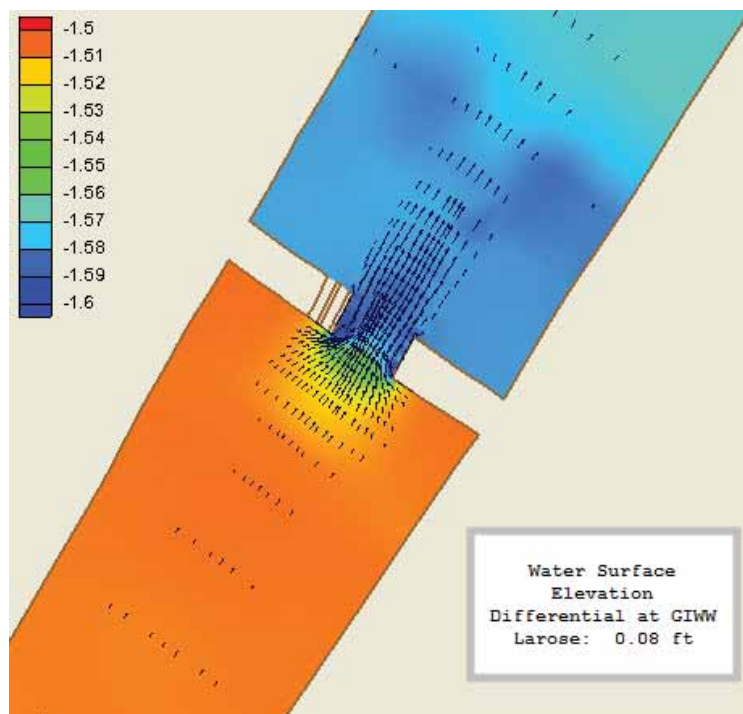
Figure 80 - Head Differential showing 0.08 ft at the GIWW Houma gate site

Table 69 - Water Surface Elevation Comparisons

Location	Existing	With Gates
Amelia	4.36	4.35
Larose	-0.16	-0.14
Dulac	1.14	1.02
Houma	1.79	1.64

Part IV - Conclusions and Recommendations.

This report contains an initial gate design for each of the 7 proposed floodgate sites. Some revisions to the design flows were completed during technical review and field data review that became available during the technical review schedule. Also, 175 ft wide configurations were evaluated for the GIWW floodgate sites at Houma and Larose. A range of magnitudes is evident between the design flows and velocities associated with the 7 floodgate sites in this study and comparisons to ERDC study simulations and field data. Some of the design values offer close agreement, some are farther apart. To explain, the Gulf, canal, and river system in the project areas is very complex, with certain floodgate sites being controlled more by high discharges, and others that are more sensitive to water surface elevation (tide, storm, and wind combinations). The model domains and boundary conditions used in this study and as reviewed from the ERDC studies are all considered accurate, and the range of magnitudes is a function of system complexity. The resulting range is presented as more accurate than a single study. The more significant design flow range occurs with the Mid-HNC gates, where the values range from <1.0 to < 2.0 fps. This low magnitude range offers confidence that even if the design values are more prone to follow the higher velocities, the floodgate designs will still offer a safe margin below the 4.4 fps criteria.

Maximum velocity values are listed below for each configuration at the proposed floodgate locations. Using the 3 mph (4.4fps) navigation criteria limit (as used at the Western Closure Complex), configurations showing values less than 4.4 fps can be considered acceptable in terms of velocity. No head differential problems were noted. The vessel presence velocity acceleration simulations on the GIWW at Houma and Larose did show a significant increase in velocity compared to the gate-only conditions, but the accelerations were still below the 4.4 fps criteria. The system model does not show any significant water level rises due to the presence of the proposed gates. The two configuration options that exceed the velocity criteria are highlighted. In terms of velocity, including the simulated presence of a design vessel between the gates, all but two options (the single 175 ft gate at Houma and the 56 ft without sluice gates at Falgout) are viable. The final structure design, including environmental flow control structures, is provided in Table 70.

GIWW at Houma-
Existing condition- 1.09 fps.
Dual 125 ft sector gate configuration- 2.5 fps
Vessel presence in dual 125 ft sector gate configuration- 3.2 fps
Single 175 ft Sector Navigation Gate + 4 sluice gates open- 3.2 fps

Single 175 ft Sector and Sluice gates open with vessel presence- 3.8 fps
Single 175 ft Sector Gate with Sluice Gates closed- 4.8 fps

GIWW at Larose-
Existing condition- 0.78 fps.
125 ft Gate configuration- 3.0 fps
Vessel presence in 125 ft gate configuration- 3.5fps
175 ft Gate configuration- 2.1 fps
175 ft Gate configuration + sluice gates- 1.8 fps
Minors Canal-
Existing condition- 0.30 fps.
Gate configuration- 0.5 fps
Grand Bayou-
Existing condition- 0.58 fps.
Gate configuration with sluice gates open- 0.5 fps
Gate configuration with sluice gates closed- 1.1 fps
Note- The lower velocity with the proposed gate in place is due to the gate sill elevation that is lower than the surrounding channel bottom elevation.

Falgout Canal-
Existing condition- 0.91 fps.
Gate configuration with sluice gates open- 1.8 fps

Gate configuration with sluice gates closed- 5.8 fps

Bayou Dularge-
Existing condition- 1.3 fps.
Gate configuration 2.0 fps
Bayou Grand Caillou-
Existing condition- 0.51 fps.
Gate configuration with sluice gates open- 0.41 fps
Gate configuration with sluice gates closed- 1.1 fps
Note- The lower velocity with the proposed gate and sluice gates open is due to the gate sill elevation that is lower than the surrounding channel bottom elevation, offering an increase in channel cross section and resulting slower velocities. Also, the ERDC field data set shows substantially higher flows in the Houma Navigation Canal near Bayou Grand Caillou. Values shown here compare well with long-term data as shown in USGS paper 1672.
Maintenance Concerns-
Bayou Grand Caillou and Grand Bayou show lower gate plan constriction velocities than their existing condition because the gate sill elevation was deeper than the surrounding channel bottom elevations. MVN has experienced sedimentation problems at other project locations where a floodgate sill elevations was lower than surrounding channel elevations. However, MVN Operations evaluations expect no problems with the MTG sites, because of very low sediment influx and cleanout action from passing vessel propwash.

Table 70 - Final Sizing Table for All Structures

Reach	Name	Sector Gate Width (FT)	Sluice Gates # & W(FT)	Env. Structure Sluice Gates # and Diameter (FT)	Invert (FT)	1% AEP LORR 2085 Elevation	1% AEP LORR 2035 Elevation	3% AEP LORR 2085 Elevation	3% AEP LORR 2035 Elevation	Latitude	Longitude
Barrier (N of GIWW)	Levee				-12	20	14	13	9	20°40'25.61"N	90°00'24.01"W
Barrier - Reach 4	Bayou Black	56	none		-4.5	23	--	15	--	29°40'21.22"N	91°00'12.82"W
Barrier - Reach 4	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°39'40.52"N	90°58'59.25"W
Barrier - Reach 4	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°38'48.20"N	90°58'18.06"W
Barrier - Reach 3	Shell Canal W	56	none		-10	23	--	16	--	29°37'38.42"N	90°56'46.97"W
Barrier - Reach 3	Shell Canal E	56	none		-12	23	--	16	--	29°37'26.51"N	90°55'57.84"W
Barrier - Reach 3	Elliot Jones Canal	20 Stop Log	none		-8	23	--	16	--	29°37'26.51"N	90°55'57.84"W
Barrier - Reach 3	Bayou Black Pump Station	FP & Butterfly Valve				23		16			
Barrier - Reach 3	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°36'37.32"N	90°53'58.12"W
Barrier - Reach 3	NAFTA	56	none		-12	23	--	16	--	29°35'56.09"N	90°52'32.23"W
Barrier - Reach 3	Hanson Canal Pump Station	FP & Butterfly Valve				23	--	16	--		
Barrier - Reach 2	Humphreys Canal	20 Stop Log	none		-8	23	--	16	--	29°35'58.50"N	90°51'37.20"W
Barrier - Reach 2	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°21'21.41"N	90°44'52.10"W
Barrier - Reach 2	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°34'57.17"N	90°49'32.18"W
Barrier - Reach 2	Environmental Control			6- 6x6	-4.5	23	--	16	--	29°34'10.65"N	90°48'53.20"W
A-3	GIWW at Houma	175	6 @ 16'		Sector Gate@-16'; sluice gates at-13	23	--	16	--	29°32'6.69"N	90°47'33.38"W
A-3 (N of GIWW)	Levee					19.5	14	13	9		
A-3 (S of GIWW)	Levee					20.5	17.5	13	9		
A-3	Minor's Canal	56	none		-9	23	--	16	--	29°33'04.20"N	90°47'51.69"W
A	Environmental Control			6 - 6x6	-4.5	23	--	16.0 or 16.5?	--	29°29'06.36"N	90°44'34.43"W
B	Levee					20.5	17.5	13.5	11		
B	Marmande Canal	30	none		-4	23	--	16.5	--	29°27'39.72"N	90°45'49.93"W
B	Falgout Canal	56	9 @ 16'		-9	23	--	16.5	--	29°24'55.77"N	90°47'14.29"W
E-2	Levee					23.5	21	15.5	14		
E-2	Highway 315 Road Gate	Road Gate				25.5	--	18	--	29°24'31.38"N	90°47'11.29"W
E-2	Bayou Du Large	56	none		-7	25.5	--	18	--	29°24'31.96"N	90°47'12.53"W
E-2	Environmental Control			9 - 6x6	-4.5	25.5	--	18	--	29°23'55.60"N	90°44'46.40"W
E-1	Levee					23.5	21	15.5	14		
E-1	Environmental Control			9 - 6x6	-4.5	25.5	--	18	--	29°24'26.31"N	90°46'11.40"W
F-2	Levee					23.5	21	15.5	14		
F-1	Levee					23.5	21	15.5	14		
F-1	Bayou Grand Caillou	56	9 @ 16'		-12	25.5	--	18	--	29°20'31.51"N	90°44'21.49"W
F-1/G-1	Houma Navigation Canal	250' gate, 110' lock			-18	30.5	--	22.5	--	29°19'46.84"N	90°43'48.06"W
G-1	Levee					24	22	17.5	16		
G-1	Four Point Bayou Road Gate	Road Gate				30.5	--	22.5	--	29°19'10.56"N	90°42'19.56"W
G-2	Levee					24	22	17.5	16		
G-2	Four Point Bayou	30	none		-6	30.5	--	22.5	--	29°19'10.28"N	90°42'18.04"W
G-2	Environmental Control			6 - 6x6	-4.5	30.5	--	22.5	--	29°19'02.84"N	90°41'54.90"W
G-2	Environmental Control			4 - 6x6	-4.5	30.5	--	22.5	--	29°18'58.09"N	90°41'38.44"W
G-3	Levee					24	22	17.5	16		
G-3	Environmental Control			4 - 6x6		24	22	17.5	16	29°18'46.71"N	90°40'54.83"W
H-1	Levee					24	22	17.5	16		
H-1	Environmental Control			1 - 6x6	-4.5	30.5	--	22.5	--	29°18'06.79"N	90°40'14.73"W

Table 70 - Final Sizing Table for All Structures

Reach	Name	Sector Gate Width (FT)	Sluice Gates # & W(FT)	Env. Structure Sluice Gates # and Diameter (FT)	Invert (FT)	1% AEP LORR 2085 Elevation	1% AEP LORR 2035 Elevation	3% AEP LORR 2085 Elevation	3% AEP LORR 2035 Elevation		
H-1	Environmental Control			6 - 6x6	-4.5	30.5	--	22.5	--	29°17'52.15"N	90°39'32.25"W
H-1	Highway 56 Road Gate	Road Gate				30.5	--	22.5	--	29°17'44.88"N	90°38'57.99"W
H-1/H-2	Bayou Petit Caillou	56	6 @ 16'		-8	30.5	--	22.5	--	29°17'44.43"N	90°38'54.62"W
H-2	Levee					25.5	23	19	16.5		90°38'40.49"W
H-2	Placid Canal	56	6 @ 16'		-8	31.5	--	24	--	29°18'31.63"N	
H-3	Levee					26.5	24	20	18		
H-3/I-1	Bush Canal	56	9 @ 16'		-12	33	--	25	--	29°22'5.39"N	90°36'7.40"W
I-1	Levee					26.5	24	20	18		
I-1/I-2	Bayou Terrebonne	56	none		-9	33	--	25	--	29°23'16.94"N	90°35'17.03"W
I-1/I-3	Highway 55 Road Gate	Road Gate				33	--	25	--	29°23'16.94"N	90°35'17.03"W
I-2	Levee					26.5	24	20	18		
I-2	Madison/Nettleton Pump Station	FP & Butterfly Valve				33	--	25	--		
I-3	Levee					26.5	24	20	18		
I-3/J-2	Humble Canal	56	none		-9	33	--	25	--	29°26'06.60"N	90°33'42.43"W
J-2	Levee					26.5	24	20	18		
J-2	Environmental Control			4 - 5x10	-3.5	33	--	25	--	29°26'12.21"N	90°32'22.13"W
J-2	Environmental Control			4 - 5x10	-3.5	33	--	25	--	29°26'33.45"N	90°31'15.87"W
J-2	Environmental Control			5 - 5x10	-3.5	33	--	25	--	29°27'20.46"N	90°29'59.10"W
J-1	Levee					26.5	24	20	18		
J-3	Levee					26.5	24	20	18		
J-3	Pointe-aux-Chenes Pump Station	FP & Butterfly Valve				33	--	25	--		
J-3	Highway 665 Road Gate	Road Gate				33	--	25	--	29°25'04.07"N	90°26'54.96"W
J-3/K	Bayou Pointe-aux-Chenes	56	none		-6	33	--	25	--	29°25'04.78"N	90°26'53.44"W
K	Levee					25.5	22.5	17.5	16		
K	Environmental Control			2 - 6x6	-4.5	29.5	--	21	--	29°26'47.15"N	90°26'49.85"W
K	Environmental Control			2 - 6x6	-4.5	29.5	--	21	--	29°27'56.00"N	90°26'24.52"W
L-3	Levee					25.5	22.5	17.5	16		
L	Environmental Control			6- 6x6	-4.5	29.5	--	21	--	29°30'11.73"N	90°25'9.78"W
L-3	Grand Bayou	56	9 @ 16'		-9	29.5	--	21	--	29°31'06.00"N	90°24'42.22"W
North of L	GIWW at Lafourche	175	3 @ 16'		Sector Gate @-16'; Sluice gates @-10'	17.5	--	15	--	29°35'17.12"N	90°22'21.06"W

2.6.5 Reverse Load Case

2.6.5.1 Background/Introduction

The proposed MTG navigational hydraulic structures will experience several different operational conditions and water levels during and after construction of the project. This document will address the methodology used to determine the reverse head load cases that will result from forces due to standing water on the protected side and flood side at each structure, which will produce a maximum differential head. The structures will be designed to handle these forces. Table 71 below provides the differential head for each of the 21 structures proposed along the levee alignment. An explanation is also included below the table, which summarizes the methodology used to determine the water surface elevations in this analysis and supporting documentation.

Table 71 - Differential Head for Maximum Reverse Head Loads from Hurricanes

	Levee Reach	Structure	Flood Side WSEL (ft)[*]	2085 3% AEP / 1% AEP Levee Protected Side WSEL (ft)[*]	2085 3% AEP / 1% AEP Levee Differential Head (ft)[*]
1	Reach A	GIWW at Houma	-1.25	3.04[1]	4.29
2		Minor's Canal	-1.25	2.54	3.79
3	Reach B	Falgout Canal	-0.74	3.06	3.80
4	Reach E	Bayou DuLarge	-0.74	3.06	3.80
5	Reach F	Bayou Grand Calliou	-0.74	3.11	3.85
6	Reach G	Houma Navigation Canal	-1.40	4.05[2]	5.45
7		Bayou Fourpoints	-2.76	3.30	6.06
8		Bayou Petite Calliou	-2.76	3.49	6.25
9	Reach H	Lapeyrouse Canal	-2.76	3.49	6.25
10		Placid Canal	-2.76	3.49	6.25
11		Bush Canal	-1.14	2.99	4.13
12	Reach I	Bayou Terrebonne	-1.64	3.10	4.74
13		Humble Canal	-1.64	1.19	2.83
14	Reach K	Bayou Pointe-aux-Chenes	-1.64	3.00	4.64
15		GIWW at Larose	-1.64	3.96[1]	5.60
16		Shell Canal E	-1.35	2.87	4.22
17	Barrier Plan Reach 3	Shell Canal W	-1.35	2.87	4.22
18		Elliot Jones Canal	-1.35	3.28	4.63
19		NAFTA Canal	-1.35	3.28	4.63
20		Humphreys Canal	-1.35	3.28	4.63
21		Black Bayou Canal	-0.87	4.58	5.45

[*]**Water surface elevations are referenced to NAVD88 datum, epoch 2004.65.**
[1] **An additional 0.50' was added to the protected side elevation for wind setup on major waterbodies.**
[2] **An additional 1.00' was added to the protected side elevation for wind setup on major waterbodies.**

2.6.5.2   Reverse Load Case Discussion and Supporting Documentation

A reverse load case will be experienced at each structure when a hurricane changes path abruptly due to a change in wind direction. The water surface elevation on the flood side is decreased to an extremely low water surface elevation and the protected side is at a higher water surface elevation due to an interior rainfall event. It is important to note that the reverse heads are generally unlikely to occur, because the structures could normally be opened to prevent water levels from reaching the elevations indicated on the protected side. The worst case scenario is assumed, which is that the protected side design stages are modeled such that the structures cannot be opened due to mechanical or electrical malfunctions. This will result in the structure experiencing a maximum head differential. This analysis only represents static loads from water levels, when the structure is in closed position and do not include any dynamic loads from waves, or boat impacts, forces acting on the hull of the boat.

The differential heads were calculated by obtaining gage data for the flood side elevation and extracting the protected side elevation from the interior model. An exhaustive search of gage records and associated gage adjustments within the study area were obtained from both MVN's and USGS's stream gage websites to establish the flood side elevation.

An analysis was completed to determine the closest gage to each structure. From the gage data, the record annual low was extracted and adjusted to the correct datum. The resulting elevation was used for the flood side elevation at each structure.

The protected side elevations were determined from the results of the unsteady interior drainage (UNET) model developed for this study. The peak water surfaces elevation was determined at each structure for the 10% annual chance storm event, future conditions, for both the 3% AEP and 1% AEP alternatives. This was used in conjunction with the protected side elevations to calculate the differential head at each structure. The interior model considered the intermediate case of eustatic sea level rise rates for all future analysis years. The model reported no difference in water surface elevations between the 3% AEP and 1% AEP levee models. The protected side water surface elevation was increased to account for wind setup. An additional 0.5 foot and 1.0 foot was added at structures located along major waterbodies, Houma Navigational Canal and GIWW, respectively. Similar guidance was used on Inner Harbor Navigational Canal Lock and West Closure Complex projects to account for strong winds that pushes the water surface in one direction and causes the water to pile up on one side of the basin (on the protected side) when the structure is in closed position. The two values, the exterior and interior water surface elevations, were used to calculate the head differential values in Table 71.

2.6.6 Rip Rap Design

2.6.6.1 Introduction

The Hydraulics and Hydrologic Branch of the Corps of Engineers, MVN, was tasked to determine the size and location of rip rap needed to protect 28 structures along the proposed MTG project alignment from scour. For this analysis, the following means of scour were identified and analyzed- tidal currents, ship wake, ship propeller (prop) wash. For tropical event risk reduction, a standard rip rap size and layout developed by the Hurricane Protection Office is proposed for each structure, and will not be detailed here. At a location where rock is required simultaneously for tropical events and the requirements of this analysis, the larger size rock shall govern.

The methodology described here consisted of superimposing all of the discrete velocity components and sizing and locating the rip rap at each structure based on the resultant velocities. The analysis and results thereof are explained in the following dialogue.

2.6.6.2 Analysis

2.6.6.2.1 Tidal Velocities

The US Army Engineer Research and Development Center developed an ADH (Adaptive Hydraulics) model to determine the maximum expected tidal velocities expected to occur through several structures. The resultant maximum tidal velocities are depicted in Figure 81. Because the configuration of the Barrier Alignment structures had not been completed by the time the modeling was done, the velocities developed by ERDC for similar structures and locations were adopted for this study.

It was quickly concluded that none of the environmental structures would need protection from tidal flows, since in addition to not having any navigational traffic; these structures would have to be sized so that the maximum velocity occurring through each is 2.6 ft/sec.
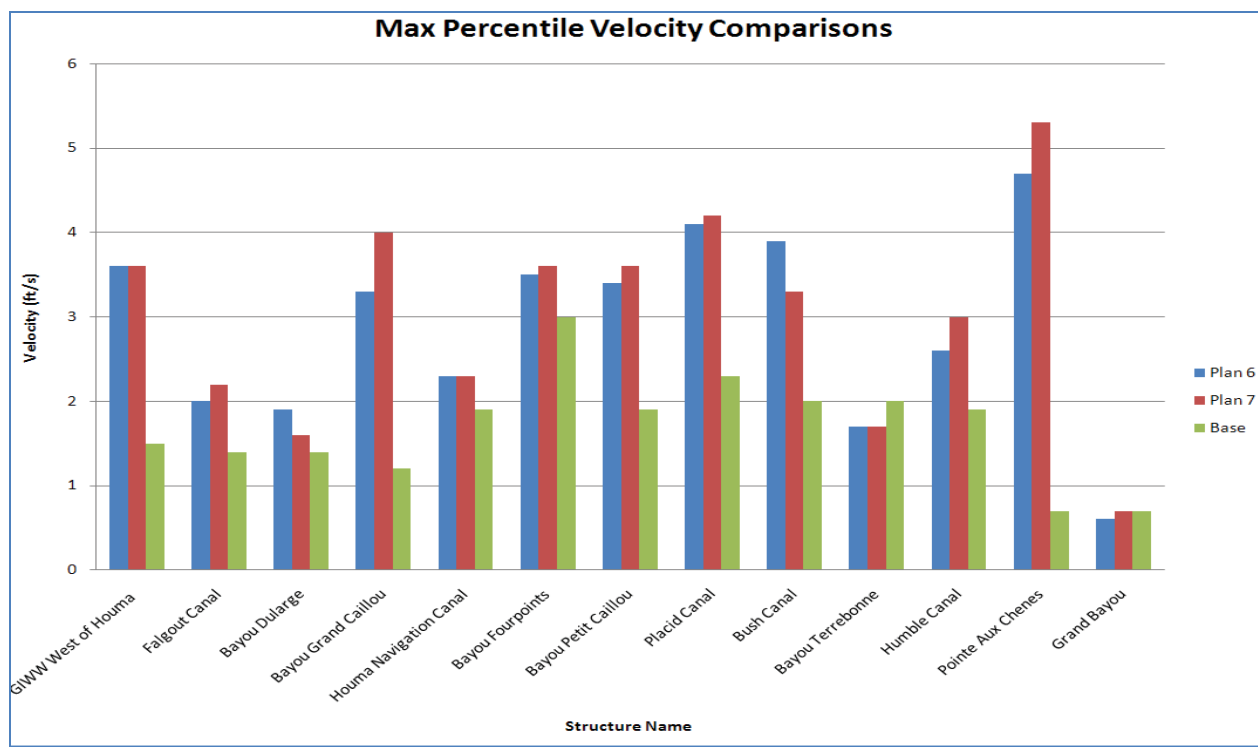
Figure 81 - Maximum tidal velocities

2.6.6.2.2 Passing Vessel – Resultant Velocities

The increase of stream velocities caused by the passing of a vessel is the resultant of two inputs- the velocity increase due to the vessel's propeller jet (prop wash) and the velocity increase due to the vessel's wake. The prop wash currents occurring at the bottom of the channel are dependent on such variables as vessel horsepower, prop diameter, speed of the vessel, and the vertical distance from the propeller to the channel bottom. The wake currents are a resultant of the bow wave velocity (a rapid decrease in velocity near the bow of the barges which is caused by the bow wave), displacement velocity (the rapid increase following the bow wave), and the return velocity. The point of interest along of the velocity distribution occurring during the vessel's passing is the maximum resultant velocity. Of note is the fact that the channel bottom is not subjected to the maximum wake velocity at the same time as the maximum propeller velocity. However, the results of applying this principle were negligible, and were not applied in this analysis.

The following steps summarize the process used to determine velocity increases, due to a passing vessel. Uncertainty is introduced for the structures in which the "large" vessel prototype was not used. For these structures, a smaller vessel with estimated design values (such as horsepower) was used. Also, the equations allow some uncertainty in their application for the Morganza structures (the equations were developed empirically during experimentation completed for the Illinois Water System). However, the equations were taken from the most in-depth and credible analysis completed (to-date) for the purpose of determining water bottom velocity influences along water bottoms

due to commercial tows.

Steps to obtain resultant bottom velocity-

1. Arrange Eq. (1), the main resultant velocity equation.

    a. $V_{vessel} = V_{prop} + V_{wake}$ (Eq. 1)

    b. Variable Definitions (for Part 1)

    V_{vessel} = Resultant max. bottom velocity increase due to passage of vessel

    V_{prop} = Maximum velocity increase due to propeller action

    V_{wake} = Velocity increase due to the wake of the ship

2. Determine "V_{prop}" for use in Eq. 1

$$V_{prop} = E\left(\frac{D_p}{H_p}\right) V_2 fn\left(\frac{V_a - V_g}{V_2}, \frac{D_p}{H_p}\right)$$ (Eq. 2)

    a. Determine $fn\left(\frac{V_a - V_g}{V_2}, \frac{D_p}{H_p}\right)$ for use in Eq. 2

$$fn\left(\frac{V_a}{V_2}, \frac{g}{H_p}, \frac{r}{H_p}\right) = 1 - cfunc\ abs\left(\frac{V_a - V_g}{V_2}\right)\left(\frac{H_p}{D_p}\right)^{1.5}$$ (Eq. 3)

    b. Determine "V_2" for use in Eq. 2 and 3

$$V_2 = \frac{1.13}{D_0}\sqrt{\frac{Thrust}{\rho}}$$ (Eq. 4)

    c. Determine "Thrust" for use in Eq. 4

$$EP_o = 23.57\ HP^{0.974} - 2.3(S)^2 (HP)^{0.5}$$ (Eq. 5)

    d. Variable Definitions (for Part 2)

.☐☐☐   Water Density

abs   = Absolute Value
cfunc = Coefficient, .50 for open wheel propellers
D_o = Jet diameter at the location of max. contraction of the jet, equal   to .71 D_p *for open wheel propellers*
D_p = Propeller diameter
fn   = Function
E   = Empirical coefficient, .43 for open wheel and .58

<div align="center">for Kort</div>
<div align="center">nozzles</div>

HP = Horsepower

H_p = Vertical distance from center of propeller to channel bottom

P_o = Thrust (lbs.)

S = Speed (m.p.h.)

V_2 = Velocity increase at propeller

V_a = Channel velocity

V_g = Vessel speed relative to ground

3. Determine V_{wake} for use in Eq. 1.

    a. Determine V_{wake} for use in Eq. 1

$$V_{wake} = -0.78\ (Draft/Depth)^{1.81}\ (V_a - V_g) \qquad \text{(Eq. 7)}$$

The equations were applied in Microsoft Excel spreadsheets to determine the resultant maximum velocities. Summary tables and detailed tables for each structure can be furnished upon request. For each structure, the resultant ship velocities were superimposed on the tidal velocities to obtain the resultant maximum velocities (Eq. 8 and 9).

$$V_{result} = V_{prop} + V_{wake} + V_{tidal} \qquad \text{(Eq. 8)}$$

$$= V_{vessel} + V_{tidal} \qquad \text{(Eq. 9)}$$

2.6.6.2.3 Rock Sizes

After the resultant velocities were obtained the rock sizes were determined using the Isbach equation (the basic equation for the movement of stones through flowing water). This equation was incorporated into the Excel spreadsheet where the velocities were computed.

The resultant gradations are contained in Table 72. Evident from the table is the fact that not all of the structures need rip rap channel protections. The criteria was that protection would be necessary once velocities surpassed 8.4 ft./sec. This velocity represents a minimal value on the Isbach table (corresponding to the equation).

2.6.6.3   Results

Since no geotechnical information was available concerning the failure plane of the structures, it is recommended, as a conservative approach (for Feasibility Study

purposes), to extend the rock out 150 feet upstream and downstream of each structure.

Regarding the top rock elevation in the channel, it was determined that the critical case for each structure is the reverse head scenario. This is because after an event causing reverse head and upon opening the gates to release interior drainage, a high flow of highly turbulent water would pass through the structure. In order to determine the upper extent of the rock along the channel cross section, a prior analysis (Alette, Washington, 2011) used to determine the reverse head load cases was used. In this analysis (2011), an unsteady interior drainage model was used to determine the maximum protected side water. The peak water surface elevations for the 10% annual chance storm event for year 2085 proposed conditions for the 1% and 3% AEP levee was extracted from the model and used to obtain the protected side elevations to calculate the differential head at each structure. The interior model considered the intermediate case of eustatic sea level rise rates for all future analysis years. The model reported no difference in water surface elevations between the 3% AEP and 1% AEP levee models. The protected side water surface elevation was increased to account for wind setup. An additional 0.5 foot and 1.0 foot was added at structures located along major waterbodies, Houma Navigational Canal and GIWW, respectively. Table 72 provides a summary of rip rap requirements. Additional data can be furnished upon request.

Table 72 - Summary of Rip Rap Requirements

Structure	Resultant bottom maximum velocity, ft/sec.	Rock required	W$_{50}$ weight, lbs.	Gradation - thickness (see Table 2)	Top rock extent elevation, NAVD 88
It is recommended as a conservative approach (for Feasibility Study purposes), to extend the rock 150 feet in each direction along the channel centerline up to the top rock extent elevation.					
Bayou Dularge	7.2	No			3.1
GIWW West of Houma	4.4	No			3.0
Falgout Canal	7.3	No			3.1
Bayou Grand Caillou	9	Yes	16.3	3	3.1
Bayou Petit Caillou	8.7	Yes	13.1	3	3.5
Placid Canal	8.9	Yes	15.9	3	3.5
Bush Canal	6.1	No			3.0
Bayou Terrebonne	8	No			3.1
Elliot Jones Canal	4.2	No			3.3
Humble Canal	9	Yes	16	3	1.2
Pointe-aux-Chenes	9.9	Yes	29.4	5	3.0
Grand Bayou	6.7	No			3.0*
GIWW N. of Bayou Lafourche	5.9	No			3.0*
Bayou Marmande	10.7	Yes	45.8	6	3.1*
Bayou Black	10.2	Yes	34.9	5	4.6
NAFTA	5.1	No			3.3
Shell East	5.1	No			2.9
Shell West	5	No			2.9
Humphreys Canal	8.6	Yes	12.5	3	3.3
Bayou Fourpoints	4.8	No			3.3
Barrier Alignment - Sluice-Gate/Culvert 1	Undetermined	Undetermined	20**	3	4.6*
Barrier Alignment - Sluice-Gate/Culvert 2	Undetermined	Undetermined	20**	3	4.6*
Barrier Alignment - Sluice-Gate/Culvert 3	Undetermined	Undetermined	20**	3	4.6*
Barrier Alignment - Sluice-Gate/Culvert 4	Undetermined	Undetermined	20**	3	2.9*
Barrier Alignment - Sluice-Gate/Culvert 5	Undetermined	Undetermined	20**	3	3.3*
Barrier Alignment - Sluice-Gate/Culvert 6	Undetermined	Undetermined	20**	3	3.3*
Barrier Alignment - Sluice-Gate/Culvert 7	Undetermined	Undetermined	20**	3	3.3*
Barrier Alignment - Sluice-Gate/Culvert 8	Undetermined	Undetermined	20**	3	3.3*

2.7 WATER QUALITY

2.7.1 System Wide Model for Salinity

TABS-MDS (TABS – Multi-Dimensional Sediment) is the ERDC version of RMA-10 with sediment transport. TABS-MDS performs one, two, and three-dimensional shallow water hydrodynamic calculations with salinity transport coupled to the hydrodynamics (King 1988). Due to the shallow nature of the bays and bayous in the system, along with the significant winds common for southern Louisiana, a high degree of mixing is prevalent in the system resulting in vertical homogeneity for the majority of the study area. Wang (1998) indicated that the Houma Navigational Canal could be partially to highly stratified, but the majority of the data shown in Wang (1998) and USGS (2008) indicate a partially stratified Houma Navigational Canal. Sensitivity model simulations were performed to determine the impact of performing 3D model simulations versus 2D model simulations for the Houma Navigational Canal. These sensitivities indicated that the stratification was limited in occurrence (temporally) and primarily located in the southern most extents of the Houma Navigational Canal. Therefore the 2D approach is adequate for the purposes of determining the impacts of the levee system on salinity. An existing mesh of the central southern Louisiana coast was provided to ERDC by the New Orleans district. This initial mesh was created for the Atchafalaya Bay Reevaluation study (Donnell, Letter, and Teeter, 1991) and later modified by Mr. David Elmore and again by Amena Henville (both of MVN) for the MTG project. This initial mesh provided by MVN, shown in Figure 82, extends from the Atchafalaya Bay (western boundary) to Port Fourchon (eastern boundary).
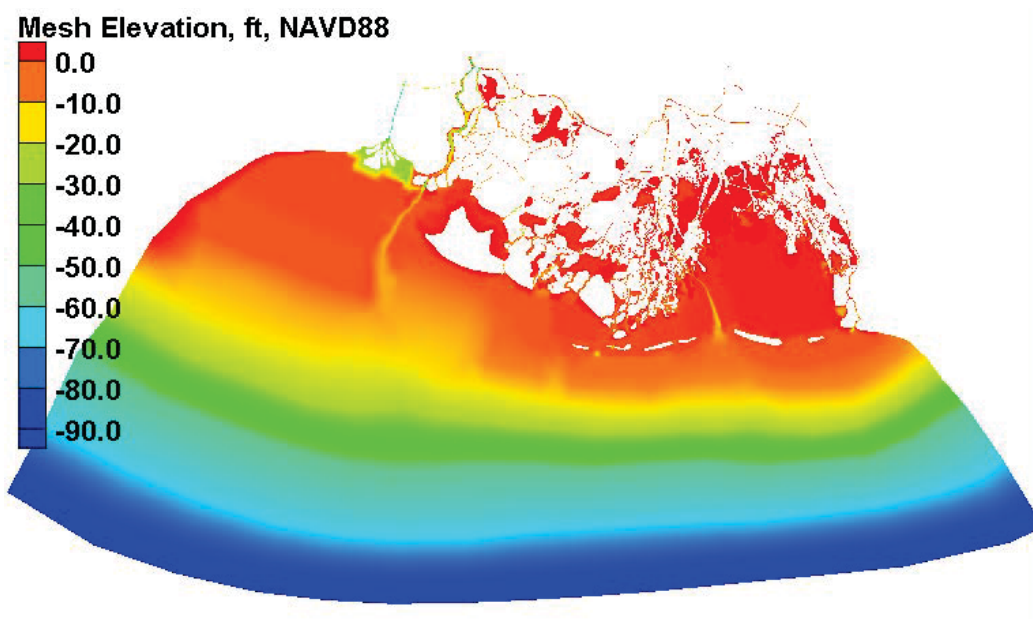
Figure 82 - Initial Mesh

The initial, schematized mesh required modification to properly model the salinity behavior of this very complex system. These mesh modifications resulted in a more detailed, accurate and stable numerical model, but at a significant cost in terms of computer run time as the new mesh contained a greatly increased spatial resolution. These modifications were essential, however, to achieve the salinity transport necessary to address the study goals. The western Vermilion Bays were also added to the model domain as they were deemed important as a storage mechanism for the large freshwater inflow from the Wax Lake Outlet and the Atchafalaya River. These grid modifications were performed in the Surface-water Modeling System, (SMS) (Brigham Young University, 2002), a graphical user interface developed by ERDC for use in setting up and running numerical models.

The existing conditions grid, Figure 83, used in the model simulations by ERDC contained 183,565 nodes and 52,968 elements. With the initiation of the Morganza project a comparison was performed between the Morganza mesh provided by MVN, the previous Atchafalaya River mesh (Atchafalaya Bay Reevaluation study), and the ADCIRC mesh (sl15v3_2007_r09a) used for hurricane surge modeling. The bathymetry used in the numerical model was a combination of these three sources. A detailed description of the bathymetry data used from each mesh to generate the Morganza bathymetry used in the model simulations can be furnished upon request.

FTN and CHT reviewed the numerical model mesh and made several recommendations for improvements. These consisted primarily of including/removing different connections throughout the model domain (e.g. channel A is should be connected to channel B, or channel A should not be connected to channel B). All of these recommendations were accepted and implemented in the model. Additional recommendations were made to add additional marsh coverage. Some strategically located marsh areas were added, but including all marsh areas was impractical. Adding all the recommended marsh areas would result in a mesh that would possess an extreme computational burden requiring an excessive amount of time to obtain model results. During the validation process, preliminary results were vetted to project stake holders, and their detailed knowledge of various locations in the study area was used to improve the model until a consensus was reached.

It should be noted that the intended purpose of this model is to model the hydrodynamic and salinity behavior of the system for normal tidal conditions (extreme surge conditions were considered in Cialone, et al. 2009) and therefore roads, levees, and other high elevation areas were not detailed in the model domain. During normal tidal periods, these areas are never inundated and therefore convey no flow making their inclusion in the model unnecessary.
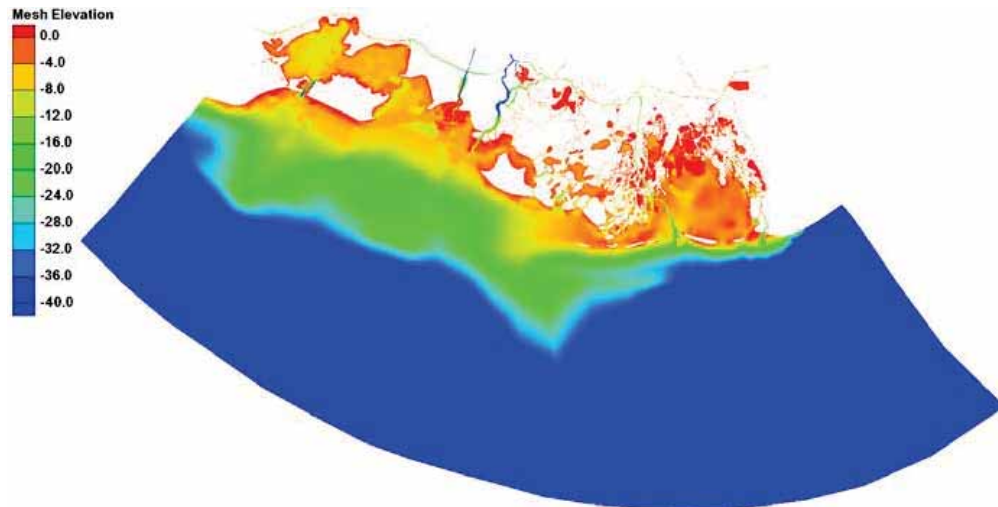
Figure 83 - Existing Conditions Mesh

2.7.1.1 Validation

The Engineering Research and Development Center (ERDC) produced a validated (hydrodynamics and salinity) numerical model that would be used to make base versus plan comparisons. The numerical model was then used to determine the resulting salinity and hydrodynamic circulation patterns due to the proposed MTG levee system. A summary of the results of the model validation are provided in this section and the full methodology and analysis can be furnished upon request.

2.7.1.1.1 Results

The salinity comparisons are satisfactory. As expected some gages compare better than others, but the basic behavior of the system is replicated very well. The exact values may not be exactly matched, but the trends tend to be simulated accurately. Some gages experience sudden large spikes in the salinity values and this behavior is also reproduced in the model results. Due to the large uncertainty in the model boundary conditions and the model bathymetry, these comparisons are as well as can be expected. For base versus plan relative comparisons, the numerical model is sufficiently validated.

2.7.1.1.2 Conclusions

Tide ranges in the northernmost sections of the model are over-estimated. Tide ranges in the remaining portions of the model are accurately replicated (with the exception of Lake Boudreaux, see below).

The tide range for Lake Boudreaux is underestimated by the model. Upon further analysis, it was observed that the field data indicated that the tide range in Lake Boudreaux was ~1.25 ft whereas the tide range in Bayou Petit Caillou was ~1.00 ft and

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the tide range at the Houma Navigation Canal gage was ~0.75 ft. Since these are the two primary means for flow to enter Lake Boudreaux, it is believed that the tide measurements in Lake Boudreaux should not be this high and therefore the gage is in error explaining the less than ideal comparisons at this location.

The numerical model does a good job of replicating the extreme events that occur during the first five months of 2004 (see Bayou Terrebonne, Houma Navigational Canal, Bayou Petit Caillou, etc comparison plots). The error metrics for the water surface elevation comparisons indicate an good replication of the field by the numerical model with average values of 0.87 for the correlation coefficient, 0.28 ft for the RMS error, and 0.92 for the Willmott coefficient.

The model does a good job replicating the mean flow measurement at Bayou Penchant, GIWW at Houma, Bayou Grand Caillou, GIWW at Bay Wallace, and GIWW at Larose. The flood flow is slightly under estimated at the Houma Navigation Canal location. As expected the error metrics for the discharge values are not as favorable as the water surface elevations, but are still acceptable with values of 0.63 for the correlation coefficient, 1761 cfs for the RMS error, and 0.72 for the Willmott coefficient.

The numerical model is slightly under estimating the salinity intrusion into Caillou Lake. The salinity comparisons for the Houma Navigation Canal show a good replication of the types of events (sudden spikes in salinity and quick returns to freshwater) that occur for this area. The magnitudes of the events are sometimes over/under estimated but the events occurring in the field are replicated to some degree in the model. The salinities for the eastern portion of the model (Pointe-aux-Chenes, Grand Bayou Canal) are over estimated in the model, but not by an unreasonable amount. The error metrics for some of the salinity gages are less than ideal. For gages with salinities residing mostly at/near 0 ppt, the correlation coefficient and Willmott coefficients are skewed toward lower values due to the relatively few non-zero model and field values.

While some of the previously shown comparisons could be improved, numerous factors indicate that significant improvements are unlikely. Those factors consist primarily of the vast domain of the model in conjunction with the numerous uncertainties present in the boundary condition specification. It is believed that the current validation is sufficient to make base versus plan comparisons. The numerical model should be very accurate in determining the salinity impact of proposed plan alterations in terms of resulting direction of change (i.e., expected to be very accurate in determining if the salinity in an area will increase or decrease). The numerical model will be less accurate determining the magnitude of change (base versus plan) for a given area. While it is believed that the magnitude of these changes in salinity will be reasonably accurate, a significant uncertainty is believed to exist in this regard. It should also be noted that the magnitude of the actual salinity value will have a significant amount of uncertainty and therefore should not be used to make any type of determinations.

2.7.1.2   Plan Comparison of Alternatives

2.7.1.2.1 Introduction

The validated model (McAlpin, 2011) was modified to include three proposed levee configurations. The validation period (2004 calendar year) from McAlpin, 2011 was used to compare the existing conditions and all three plan configurations. A comprehensive analysis was performed on the water surface elevations, discharges, and salinities to obtain an approximate indication of the resulting behavior of the system if the proposed changes were to occur.

The validated (hydrodynamics and salinity) TABS-MDS mesh was used as the starting point in generating the plan condition meshes (McAlpin, 2011). The existing conditions mesh (Figure 83) was modified to include the impermeable levee along with all proposed structures to be located along the levee. Plan 1 will be discussed in detail with plans 2 and 3 being discussed as slight modifications to plan 1. Full model development details can be furnished upon request.

2.7.1.2.2 Base Versus Plan Comparisons

Extensive base (existing conditions) versus plan comparisons were performed. These comparisons were separated into three categories to better facilitate the comparisons. The first category consists of comparison of the residual (average) water levels and salinities. The second comparison was done by looking at a time series of the water surface elevations and salinities at a number of discrete points throughout the study area. The third comparison consisted of comparing discharges between the base and plan conditions at a number of locations.

A comparison was performed on the average salinity values for the base and plan conditions. In an effort to observe any seasonal changes to the system, the average values were obtained for:

1. January to March for 2004
2. April to June for 2004
3. July to September for 2004
4. October to December for 2004

These comparisons show that the changes in the average salinity values due to the new levee system are minor (less than 1 ppth in most areas). The changes in average water surface elevations were determined to be minor as well (less than 0.1 ft in most areas). There are some localized areas that experience greater changes. It should be noted that the results from the January to March time period should be used with caution as this period is still being influenced by the initially specified salinity field. The results are provided here in Table 73 and Table 74 respectively. Discharge results are presented in Figure 84. Detailed comparison plots can be furnished upon request.

Table 73 - Minimum, Maximum, and Mean Water Surface Elevations for the Base and Plans 1 – 3 for the 2004 calendar year simulations

Point Number	Base Min (ft)	Base Max (ft)	Base Mean (ft)	Plan 1 Min (ft)	Plan 1 Max (ft)	Plan 1 Mean (ft)	Plan 2 Min (ft)	Plan 2 Max (ft)	Plan 2 Mean (ft)	Plan 3 Min (ft)	Plan 3 Max (ft)	Plan 3 Mean (ft)
1	-2.44	2.69	-0.16	-2.45	2.70	-0.16	-2.45	2.71	-0.16	-2.63	2.73	-0.19
2	-2.10	2.71	-0.13	-2.11	2.73	-0.14	-2.13	2.75	-0.14	-2.68	2.83	-0.18
3	-1.96	2.71	-0.11	-1.97	2.73	-0.11	-1.98	2.75	-0.11	-2.71	2.90	-0.16
4	-1.71	2.59	-0.05	-1.66	2.52	-0.05	-1.66	2.54	-0.05	-1.04	2.01	0.07
5	-1.12	2.42	0.11	-1.11	2.39	0.11	-1.10	2.40	0.11	-0.91	2.32	0.15
6	-1.11	2.60	0.13	-1.11	2.59	0.13	-1.10	2.58	0.13	-0.91	2.39	0.16
7	-1.11	2.62	0.10	-1.11	2.59	0.09	-1.10	2.60	0.10	-0.98	2.34	0.11
8	-0.99	2.63	0.23	-0.99	2.60	0.23	-0.98	2.60	0.23	-0.82	2.38	0.25
9	-0.81	2.61	0.39	-0.81	2.58	0.39	-0.80	2.59	0.40	-0.67	2.40	0.41
10	-0.55	2.61	0.65	-0.55	2.59	0.65	-0.55	2.60	0.65	-0.48	2.46	0.66
11	-2.56	2.68	-0.17	-2.56	2.68	-0.17	-2.56	2.69	-0.17	-2.66	2.70	-0.18
12	-2.35	2.80	-0.17	-2.35	2.80	-0.17	-2.35	2.80	-0.17	-2.37	2.81	-0.19
13	-2.24	2.81	-0.16	-2.24	2.81	-0.16	-2.24	2.82	-0.16	-2.23	2.85	-0.17
14	-1.93	2.84	-0.13	-1.91	2.85	-0.13	-1.91	2.86	-0.13	-1.50	2.84	-0.10
15	-2.06	2.81	-0.19	-2.05	2.81	-0.19	-2.05	2.82	-0.19	-1.97	2.82	-0.18
16	-1.85	2.85	-0.16	-1.82	2.86	-0.16	-1.84	2.86	-0.16	-1.63	2.85	-0.13
17	-2.13	2.76	-0.20	-2.13	2.76	-0.20	-2.13	2.76	-0.20	-2.08	2.77	-0.19
18	-1.83	2.76	-0.09	-1.77	2.74	-0.09	-1.79	2.75	-0.09	-1.18	2.59	0.00
19	-1.66	2.85	-0.14	-1.61	2.86	-0.13	-1.65	2.86	-0.14	-1.46	2.81	-0.08
20	-1.50	2.30	-0.12	-1.28	2.34	-0.04	-1.47	2.30	-0.12	-1.08	2.17	0.03
21	-1.86	2.88	-0.11	-1.78	2.84	-0.09	-1.76	2.85	-0.09	-1.62	2.82	-0.07
22	-2.06	2.93	-0.18	-2.06	2.93	-0.18	-2.06	2.93	-0.18	-2.03	2.94	-0.18
23	-1.82	3.12	-0.14	-1.82	3.12	-0.14	-1.82	3.12	-0.14	-1.81	3.13	-0.14
24	-0.79	2.15	0.19	-0.82	2.15	0.18	-0.80	2.14	0.19	-0.74	2.08	0.20
25	-0.83	2.33	0.23	-0.86	2.33	0.22	-0.84	2.32	0.22	-0.78	2.21	0.23
26	-0.79	2.46	0.33	-0.81	2.44	0.33	-0.80	2.44	0.33	-0.72	2.31	0.33
27	-1.13	2.16	0.09	-1.13	2.15	0.09	-1.10	2.15	0.10	-0.95	2.12	0.13
28	-1.24	2.26	0.04	-1.25	2.23	0.03	-1.22	2.25	0.04	-0.98	2.09	0.10
29	0.27	1.80	0.47	0.27	1.62	0.46	0.27	1.61	0.47	0.27	1.61	0.48
30	-2.45	2.81	-0.18	-2.47	2.83	-0.18	-2.47	2.83	-0.18	-2.49	2.83	-0.18
31	-2.40	2.77	-0.16	-2.39	2.75	-0.16	-2.39	2.75	-0.16	-2.50	2.76	-0.17
32	-1.04	2.82	-0.07	-1.03	2.84	-0.07	-1.08	2.87	-0.09	-1.04	2.93	-0.09

Point Number	Base Min (ft)	Base Max (ft)	Base Mean (ft)	Plan 1 Min (ft)	Plan 1 Max (ft)	Plan 1 Mean (ft)	Plan 2 Min (ft)	Plan 2 Max (ft)	Plan 2 Mean (ft)	Plan 3 Min (ft)	Plan 3 Max (ft)	Plan 3 Mean (ft)
33	-1.36	2.81	-0.07	-1.32	2.79	-0.07	-1.47	2.83	-0.10	-1.35	2.93	-0.09
34	-1.70	2.37	-0.08	-1.65	2.29	-0.08	-1.64	2.30	-0.08	-1.09	1.96	0.03
35	0.11	2.26	0.34	0.11	2.17	0.34	0.11	2.18	0.34	0.12	2.03	0.36
36	-1.48	2.90	-0.09	-1.45	2.62	-0.09	-0.68	1.80	-0.01	-1.50	2.73	-0.11
37	-1.49	2.06	-0.09	-1.43	1.98	-0.09	-1.43	1.98	-0.08	-1.18	1.91	-0.04
38	-1.92	2.46	-0.16	-1.62	2.13	-0.13	-1.62	2.13	-0.13	-1.40	2.08	-0.10
39	-2.02	2.87	-0.14	-2.06	2.92	-0.14	-2.07	2.93	-0.14	-1.97	2.92	-0.13
40	-2.00	3.01	-0.13	-2.02	3.05	-0.14	-2.03	3.06	-0.14	-1.93	3.05	-0.13
41	-2.02	2.98	-0.14	-2.04	3.00	-0.14	-2.05	3.01	-0.14	-1.95	3.00	-0.13
42	-1.81	2.52	-0.12	-1.73	2.33	-0.11	-1.73	2.33	-0.11	-1.47	2.27	-0.08
43	-1.96	2.97	-0.13	-1.92	2.79	-0.13	-1.93	2.79	-0.13	-1.76	2.77	-0.11
44	-1.65	2.24	-0.12	-1.59	2.16	-0.12	-1.58	2.16	-0.11	-1.42	2.06	-0.07
45	-1.88	3.23	-0.15	-1.84	3.28	-0.15	-1.87	3.29	-0.15	-1.82	3.27	-0.15
46	-1.68	3.10	-0.12	-1.62	3.27	-0.12	-1.65	3.33	-0.13	-1.57	3.27	-0.11
47	-1.66	3.08	-0.11	-1.60	3.13	-0.11	-1.63	3.16	-0.11	-1.53	3.11	-0.10
48	0.44	3.09	0.56	0.44	2.03	0.55	N/A	N/A	N/A	0.44	2.02	0.55
49	-1.72	3.06	-0.14	-1.67	3.27	-0.14	-1.72	3.38	-0.14	-1.64	3.27	-0.13
50	-1.74	3.11	-0.15	-1.82	3.32	-0.15	-1.86	3.42	-0.15	-1.79	3.31	-0.14
51	0.16	3.17	0.36	0.16	2.12	0.35	N/A	N/A	N/A	0.16	2.11	0.35
52	-1.57	2.89	-0.09	-1.52	2.92	-0.09	-1.55	2.95	-0.09	-1.44	2.86	-0.08
53	0.18	2.29	0.36	0.18	2.06	0.36	N/A	N/A	N/A	0.18	2.05	0.36
54	-1.56	2.26	-0.16	-1.51	2.07	-0.16	-1.53	2.21	-0.17	-1.49	2.06	-0.16
55	-1.15	2.22	-0.06	-1.77	2.25	-0.12	-0.96	2.20	-0.02	-1.75	2.20	-0.12
56	-2.33	3.03	-0.18	-2.28	3.00	-0.18	-2.12	3.02	-0.18	-2.27	3.00	-0.18
57	-2.33	2.98	-0.19	-2.32	2.99	-0.19	-2.34	3.02	-0.19	-2.31	2.99	-0.19
58	-2.33	3.15	-0.19	-2.32	3.16	-0.19	-2.34	3.17	-0.19	-2.31	3.16	-0.19
59	-2.36	3.19	-0.20	-2.35	3.20	-0.20	-2.37	3.21	-0.20	-2.35	3.20	-0.20
60	-2.23	2.88	-0.20	-2.22	2.89	-0.20	-2.23	2.89	-0.20	-2.22	2.89	-0.19
61	-2.28	2.92	-0.19	-2.27	2.93	-0.19	-2.28	2.94	-0.19	-2.27	2.93	-0.19
62	-2.25	2.61	-0.18	-2.24	2.64	-0.18	-2.26	2.68	-0.19	-2.24	2.63	-0.18
63	-2.10	2.58	-0.17	-2.09	2.62	-0.17	-2.11	2.66	-0.17	-2.09	2.61	-0.17
64	-2.34	2.83	-0.18	-2.32	2.86	-0.18	-2.35	2.91	-0.18	-2.31	2.86	-0.18
65	-0.42	1.94	0.06	-0.53	2.04	0.01	-0.35	1.74	0.10	-0.53	2.02	0.01
66	-0.60	2.46	0.00	-0.95	2.20	-0.10	-0.01	1.60	0.40	-0.94	2.19	-0.10
67	-2.34	2.68	-0.18	-2.31	2.73	-0.18	-2.35	2.80	-0.18	-2.30	2.73	-0.18
68	-2.14	2.66	-0.19	-2.13	2.70	-0.19	-2.15	2.75	-0.19	-2.12	2.69	-0.19

Point Number	Base Min (ft)	Base Max (ft)	Base Mean (ft)	Plan 1 Min (ft)	Plan 1 Max (ft)	Plan 1 Mean (ft)	Plan 2 Min (ft)	Plan 2 Max (ft)	Plan 2 Mean (ft)	Plan 3 Min (ft)	Plan 3 Max (ft)	Plan 3 Mean (ft)
69	-1.04	2.73	-0.14	-1.04	2.77	-0.14	-1.04	2.82	-0.14	-1.04	2.76	-0.14
70	-2.34	2.81	-0.20	-2.33	2.85	-0.20	-2.35	2.89	-0.20	-2.32	2.84	-0.20
71	-1.88	2.62	-0.16	-1.84	2.66	-0.16	-1.93	2.76	-0.17	-1.81	2.65	-0.15
72	-1.01	2.57	-0.05	-1.76	2.27	-0.15	-0.16	1.61	0.37	-1.74	2.25	-0.14
73	-1.77	2.66	-0.16	-1.71	2.68	-0.15	-1.79	2.74	-0.16	-1.65	2.66	-0.15
74	-1.58	2.61	-0.13	-1.58	2.58	-0.14	-1.64	2.62	-0.14	-1.52	2.54	-0.13
75	-0.96	2.60	-0.09	-0.98	2.53	-0.10	-0.92	2.55	-0.09	-0.96	2.50	-0.09
76	-1.53	2.54	-0.13	-1.52	2.52	-0.13	-1.57	2.55	-0.13	-1.47	2.47	-0.13
77	-1.28	1.93	-0.11	-1.28	1.92	-0.11	-1.28	1.92	-0.11	-1.22	1.86	-0.11
78	-1.26	2.12	-0.09	-1.26	2.12	-0.09	-1.26	2.12	-0.09	-1.19	2.02	-0.09

Table 74 - Minimum, Maximum, and Mean Salinities for the Base and Plans 1 – 3 for the 2004 calendar year simulations

Point Number	Base Min (ppt)	Base Max (ppt)	Base Mean (ppt)	Plan 1 Min (ppt)	Plan 1 Max (ppt)	Plan 1 Mean (ppt)	Plan 2 Min (ppt)	Plan 2 Max (ppt)	Plan 2 Mean (ppt)	Plan 3 Min (ppt)	Plan 3 Max (ppt)	Plan 3 Mean (ppt)
1	0.03	28.19	15.84	0.03	28.16	15.93	0.02	28.17	15.85	0.64	28.37	20.81
2	0.01	27.49	11.86	0.01	27.39	11.93	0.01	27.40	11.83	0.36	27.75	18.67
3	0.00	27.08	8.35	0.00	26.96	8.46	0.00	26.97	8.31	0.01	27.40	14.87
4	0.00	26.84	4.53	0.00	26.47	4.66	0.00	26.51	4.39	0.00	23.23	1.45
5	0.00	20.94	0.83	0.00	20.25	0.78	0.00	20.78	0.80	0.00	1.34	0.04
6	0.00	11.31	0.24	0.00	10.25	0.21	0.00	10.57	0.22	0.00	1.91	0.02
7	0.00	13.28	0.12	0.00	13.33	0.11	0.00	13.27	0.12	0.00	12.07	0.12
8	0.00	7.28	0.08	0.00	5.47	0.05	0.00	6.40	0.06	0.00	3.32	0.02
9	0.00	3.29	0.02	0.00	2.28	0.01	0.00	2.84	0.02	0.00	0.86	0.00
10	0.00	1.77	0.01	0.00	1.14	0.01	0.00	1.47	0.01	0.00	0.15	0.00
11	1.15	27.87	17.00	1.09	27.83	17.09	1.11	27.85	16.98	3.01	28.18	21.17
12	0.46	27.14	12.63	0.40	27.10	12.68	0.45	27.12	12.59	0.30	27.60	16.46
13	0.19	26.76	9.12	0.11	26.70	9.13	0.19	26.72	9.07	0.01	27.18	12.02
14	0.01	26.12	5.58	0.00	25.91	5.50	0.02	26.21	5.63	0.00	26.13	4.05
15	0.19	27.75	11.11	0.18	27.69	10.92	0.19	27.70	11.10	0.08	27.91	10.89
16	0.06	23.97	5.49	0.03	22.65	4.83	0.06	23.95	5.54	0.00	17.12	2.12
17	0.03	27.84	8.61	0.03	27.83	8.48	0.03	27.83	8.60	0.02	27.86	8.11
18	0.00	26.72	5.31	0.00	26.54	5.23	0.00	26.69	5.29	0.00	24.79	2.73

Point Number	Base			Plan 1			Plan 2			Plan 3		
	Min (ppt)	Max (ppt)	Mean (ppt)	Min (ppt)	Max (ppt)	Mean (ppt)	Min (ppt)	Max (ppt)	Mean (ppt)	Min (ppt)	Max (ppt)	Mean (ppt)
19	0.11	23.037	5.28	0.01	21.49	4.53	0.12	22.96	5.37	0.00	13.87	1.54
20	0.18	17.77	5.05	0.00	14.57	3.21	0.18	17.82	5.16	0.00	4.61	0.81
21	0.00	24.74	2.09	0.00	23.84	2.14	0.00	24.55	2.04	0.00	17.97	0.61
22	0.00	24.77	3.57	0.00	24.75	3.58	0.00	24.78	3.58	0.00	23.83	3.28
23	0.00	14.81	1.28	0.00	14.79	1.28	0.00	14.83	1.28	0.00	13.50	1.19
24	0.00	6.38	0.81	0.00	6.14	0.78	0.00	6.38	0.81	0.00	2.59	0.24
25	0.00	1.39	0.06	0.00	1.10	0.05	0.00	1.20	0.06	0.00	0.34	0.01
26	0.00	2.74	0.03	0.00	1.95	0.02	0.00	2.43	0.02	0.00	0.81	0.00
27	0.00	25.63	2.27	0.00	25.12	2.31	0.00	25.58	2.21	0.00	14.16	0.57
28	0.00	26.63	2.96	0.00	26.31	3.01	0.00	26.31	2.87	0.00	16.70	0.70
29	0.00	25.58	3.50	0.00	25.25	3.75	0.00	25.40	3.45	0.00	14.87	1.03
30	1.94	25.92	13.55	2.02	25.92	13.88	2.02	25.90	13.76	0.83	26.08	12.79
31	0.05	26.29	11.29	0.05	26.01	11.11	0.04	26.10	11.01	0.19	27.01	12.08
32	0.16	25.68	10.68	0.21	25.22	10.48	0.10	25.61	10.57	1.49	26.26	16.71
33	0.08	25.80	8.15	0.13	24.83	8.07	0.03	25.42	7.78	1.05	26.58	15.43
34	0.00	26.77	5.06	0.00	26.26	5.18	0.00	26.26	4.95	0.00	22.66	2.08
35	0.00	25.87	1.59	0.00	25.70	1.60	0.00	25.70	1.57	0.00	17.93	1.23
36	0.17	23.52	8.10	0.35	21.93	7.75	0.40	12.46	4.74	0.40	24.65	14.77
37	1.28	20.64	7.61	1.31	20.35	7.68	1.33	20.26	7.55	0.15	21.42	6.06
38	0.75	24.08	9.58	0.11	25.69	9.93	0.10	25.79	9.83	0.17	26.82	9.96
39	1.36	23.54	9.91	1.45	23.42	10.12	1.43	23.38	9.99	0.18	23.56	8.73
40	1.95	23.13	10.68	1.96	22.90	10.80	1.96	22.79	10.64	1.42	23.50	9.50
41	0.84	22.12	8.30	0.95	22.04	8.59	0.98	21.93	8.45	0.20	22.68	7.29
42	1.26	21.72	7.97	1.31	21.85	8.28	1.31	21.76	8.15	0.17	22.63	6.94
43	0.80	21.34	7.06	0.86	21.38	7.58	0.83	21.26	7.45	0.13	22.05	6.46
44	0.73	16.98	5.19	0.73	17.01	5.38	0.73	16.93	5.27	0.21	17.77	3.82
45	0.27	20.92	6.33	0.26	20.69	6.19	0.21	20.44	5.98	0.28	21.25	5.79
46	0.00	20.12	3.74	0.00	19.84	3.56	0.00	19.49	3.31	0.00	20.41	3.63
47	0.00	20.72	3.07	0.00	20.23	2.84	0.00	19.92	2.67	0.00	20.85	3.07
48	0.16	18.84	3.06	0.23	13.94	2.94	N/A	N/A	N/A	0.23	14.74	3.10
49	0.22	19.15	4.57	0.22	18.74	4.14	0.21	18.24	3.89	0.22	19.34	4.12
50	0.30	15.06	4.77	0.30	15.10	4.31	0.30	14.04	4.26	0.30	15.82	4.28
51	0.34	14.27	3.90	0.33	12.51	3.63	N/A	N/A	N/A	0.33	13.36	3.70
52	0.00	20.23	2.11	0.00	19.72	1.98	0.00	19.43	1.91	0.00	20.43	2.35
53	0.84	13.41	5.87	0.83	12.70	5.50	N/A	N/A	N/A	0.83	13.49	5.38
54	0.71	13.64	5.87	0.75	13.00	5.61	0.75	12.72	5.58	0.76	13.82	5.49

Point Number	Base Min (ppt)	Base Max (ppt)	Base Mean (ppt)	Plan 1 Min (ppt)	Plan 1 Max (ppt)	Plan 1 Mean (ppt)	Plan 2 Min (ppt)	Plan 2 Max (ppt)	Plan 2 Mean (ppt)	Plan 3 Min (ppt)	Plan 3 Max (ppt)	Plan 3 Mean (ppt)
55	0.00	20.52	3.18	0.00	21.03	4.13	0.00	21.07	2.43	0.00	21.56	4.08
56	0.28	21.66	6.48	0.22	21.85	8.25	0.17	21.80	6.02	0.19	22.32	8.01
57	0.62	22.12	10.67	0.68	22.13	10.96	0.56	21.98	10.42	0.67	22.57	10.59
58	2.06	23.50	13.17	1.94	23.51	13.21	2.14	23.40	13.21	1.93	23.92	12.78
59	2.39	24.73	15.24	2.39	24.74	15.24	2.40	24.72	15.34	2.39	24.98	14.85
60	0.44	23.37	12.69	0.45	23.41	12.65	0.44	23.41	12.79	0.45	23.65	12.34
61	0.36	20.42	11.35	0.36	20.42	11.31	0.36	20.49	11.46	0.36	20.74	11.00
62	0.26	21.02	8.33	0.26	21.03	8.35	0.25	21.03	8.40	0.26	21.35	8.10
63	0.44	20.36	8.04	0.44	20.42	8.09	0.44	20.30	8.22	0.44	20.93	7.83
64	0.23	21.71	9.40	0.41	21.75	9.52	0.22	21.64	9.53	0.41	22.20	9.20
65	0.90	7.40	3.16	0.90	16.21	7.65	0.84	6.60	2.47	0.90	16.79	7.44
66	0.13	19.79	3.74	0.10	19.56	3.72	0.15	1.45	0.41	0.08	20.16	3.66
67	0.03	20.45	4.81	0.03	20.42	4.96	0.00	20.19	4.93	0.03	20.96	4.83
68	0.25	12.95	4.91	0.25	12.80	4.94	0.25	13.47	5.14	0.25	13.51	4.81
69	0.15	14.40	3.95	0.15	14.39	3.93	0.15	14.84	4.06	0.15	15.11	3.84
70	0.06	13.16	3.34	0.06	13.12	3.31	0.06	13.44	3.41	0.06	13.81	3.24
71	0.00	17.29	2.40	0.00	16.75	2.38	0.00	17.15	2.50	0.00	17.35	2.41
72	0.00	16.82	2.41	0.00	16.61	2.36	0.15	3.44	0.46	0.00	17.26	2.40
73	0.00	14.18	1.69	0.00	13.87	1.65	0.00	13.93	1.78	0.00	17.01	1.73
74	0.00	14.31	1.40	0.00	14.02	1.37	0.00	13.71	1.47	0.00	17.17	1.49
75	0.00	10.93	1.39	0.00	10.90	1.25	0.00	12.70	1.01	0.00	15.06	1.37
76	0.00	14.39	1.22	0.00	14.10	1.21	0.00	13.84	1.29	0.00	17.28	1.35
77	0.00	17.67	0.65	0.00	17.64	0.65	0.00	17.55	0.65	0.00	20.79	0.80
78	0.00	13.26	0.58	0.00	13.05	0.57	0.00	12.84	0.58	0.00	16.60	0.72
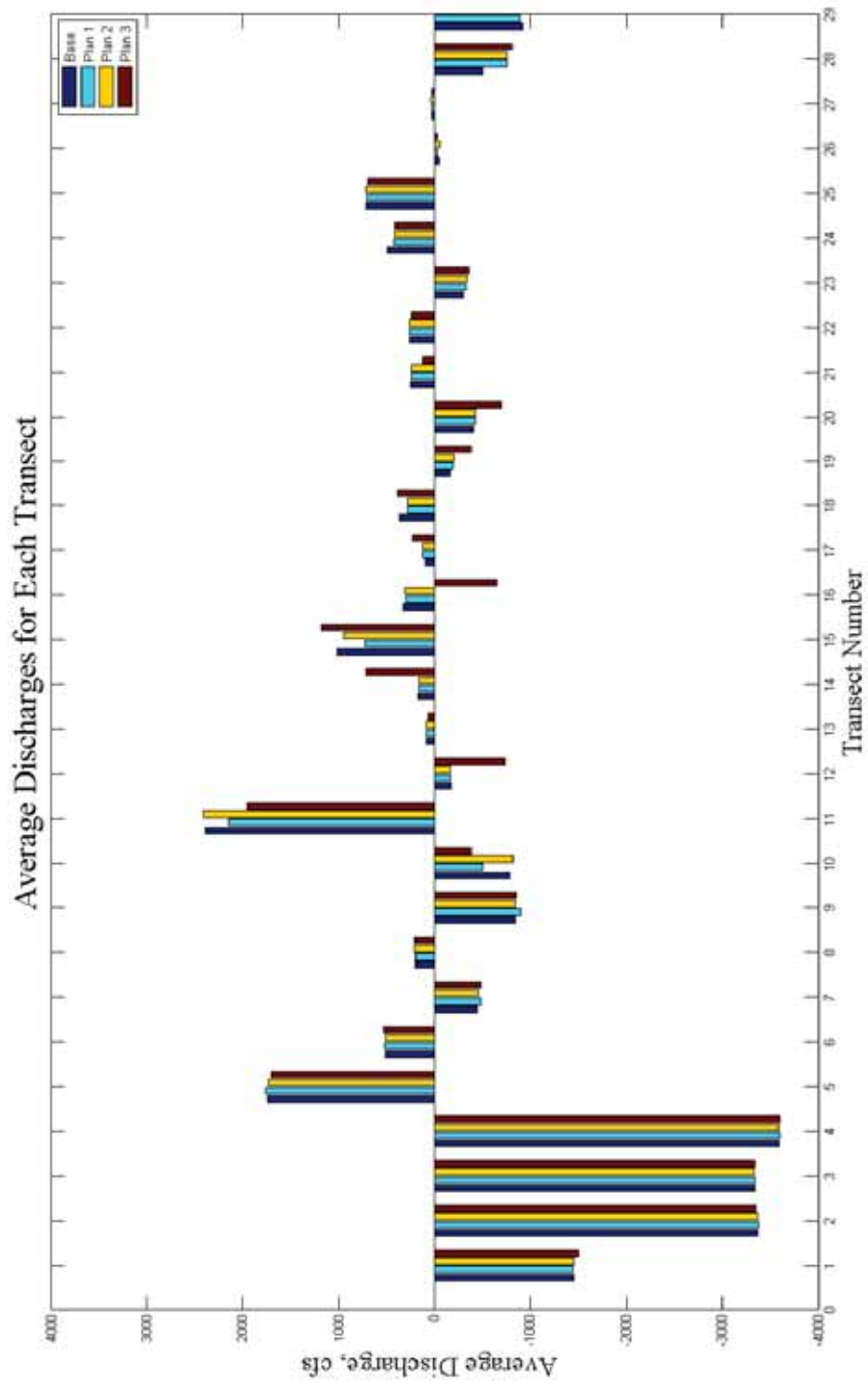
Figure 84 - Average discharges for all 29 transects for 2004

2.7.1.2.3 Summary

2.7.1.2.3.1 Water Surface Elevation Changes

There was minimal or no change in the water surface elevations between the base and the plans for points: 1, 2, 11, 12, 13, 15, 16, 17, 21, 22, 23, 30, 31, 32, 39, 40, 41, 43, 45, 46, 47, 49, 50, 52, 56, 57, 58, 59, 60, 61, 62, 63, 64, 67, 68, 69, 70, 71, 73, 74, 75, 76, 77 and 78.

 Plan 3 has a higher tide range for points 3, 65, 66, and 72 and a lower tide range for points 4, 5, 6, 7, 8, 9, 10, 14, 18, 19, 24, 25, 26, 27, 28, 29, 34, 35, 37, 42 and 44 with minimal change in tide range for plans 1 and 2
.
Plans 1 has an increased tide range and higher mean water level for point 20 (south of Falgout Canal) while plan 3 has an increased mean water level and a slightly reduced tide range. Plan 2 is similar to the base in mean water level and tide range. Point 33 has a slightly increased tide range for all plan conditions with the largest increase occurring with plan 2. Point 36 has a reduced tide range for all plans with a significant reduction occurring in plan 2 along with an increase in the mean water level.
Point 38 and 54 has a reduced tide range for all plans.

North of the proposed Houma Navigational Canal structure and lock, plan 3 has a significantly decreased tide range compared to the base and plans 1 and 2 (see point 4). The percentage of tide range reduction is increased for points farther north along the Houma Navigational Canal (HNC) as additional channels compensate for the elimination of flow up the HNC.

Points 48, 51 and 53 have reduced tide ranges for plans 1 and 3 with plan 2 having no tidal signal due to the ponding occurring in these areas for this configuration. Plans 1 and 3 have an increased tide range for points 55 and 65 with plan 2 having a similar tide range but a higher mean water level.

Points 66 and 72 have a higher tide range for plans 1 and 3 with a very minimal tide for plan 2. Plan 2 does have an increased mean water level. The mean water level for point 20 is increased for plans 1 and 3 with the plan 2 mean water level unchanged. Plans 1 and 3 have decreased mean water levels for points 55, 65, 66, and 72 while plan 2 has increased mean water levels.

Plan 3 has decreased mean water levels for points 2 and 3 and increased mean water levels for points 4, 5, 6, 18, 19, 21, 27, 28, 34, 37, 38, 42 and 44.

2.7.1.2.3.2   **Salinity Changes**

Points 7, 8, 9, 10, 17, 23, 25, 26, 46, 47 and 76 have minimal differences in salinity. Plan 1 had salinity increases for points 38, 41, 42, 43, 55, 56, 57 and 65 with salinity decreases for points 16, 19, 20, 31, 48, 49, 50, 51, 53, 54 and 75. The remaining points had minimal differences in the salinity for plan 1. The bold points are locations with average salinity changes of greater than 0.5 ppt for plan 1.

Plan 2 had salinity increases for points 38, 41, 42, 43, 64, 67, 68, 69, 70, 71, 73, and 74 with salinity decreases for points 31, 33, 36, 45, 49, 50, 52, 54, 55, 56, 57, 65, 66, 72 and 75. The remaining points had minimal differences in the salinity for plan 2. The bold points are locations with average salinity changes of greater than 0.5 ppt for plan 2.

Plan 3 had salinity increases for points 1, 2, 3, 11, 12, 13, 31, 32, 33, 36, 38, 52, 55, 56, 65, 77 and 78 with salinity decreases for points 4, 5, 6, 14, 15, 16, 18, 19, 20, 21, 22, 24, 27, 28, 29, 30, 34, 35, 37, 39, 40, 41, 42, 43, 44, 45, 49, 50, 53, 54, 58, 59, 60, 61, 62, 63 and 64. The remaining points had minimal differences in the salinity for plan 3. The bold points are locations with average salinity changes of greater than 0.5 ppt for plan 3.

2.7.1.2.3.3   **Discharge Changes**

Transects 6, 8, 22, 23, 25 and 26 show minimal differences between the existing conditions and all plan configurations.

Plan 1 discharge changes (compared to the base) are primarily in the vicinity of Falgout Canal. For Falgout Canal, the environmental structures connecting to the south provide an additional avenue for water to leave the system and therefore draw water from adjacent areas.

All plan configurations have slightly higher discharges ranges for Transects 17, 18 and 19.

Plan 3 has significant, wide-ranging changes to the flow of water occurring in the system. Due to the closing of the Houma Navigational Canal (HNC) lock and gate structure, flow is diverted to areas to the east and west of the HNC. This includes increased flows to the west (along Falgout Canal) and to the east (into Lake Boudreaux).

2.7.1.2.4 Conclusions

From the base versus plan comparisons discussed previously, some conclusions can be drawn. Through proper management of the planned structures a number of different salinity results, both beneficial and not, can be accomplished. From looking at the combination of results, it can be determined that through proper operation of the structures the proposed levee system will have a minimal effect on the global salinity values.

Plan 1 possesses minimal global salinity changes with the largest changes occurring in the marsh area south of Falgout Canal. This area is newly connected to Falgout Canal allowing for a new freshwater inflow to this area which in turn reduces the salinities (~3 ppt) with the largest benefit occurring during the winter months and minimal benefit occurring during the summer months. Globally the salinities changes tend to be less than 1 ppt with some larger localized salinity changes.

Plan 2 has minimal global salinity changes (less than 2 ppt) with some increased salinities possible in localized areas newly cutoff by the proposed levee system. Plan 2 has some areas that possess no connection to the remainder of the domain (due to closed environmental structures) and therefore will remain stagnant with constant water levels and salinities.

Plan 3 has noticeable salinity changes along the Houma Navigational Canal. The salinities are increased along the southern portion (~5 ppt) and lowered north of the Houma Navigational Canal structure. The Falgout Canal and Lake Boudreaux areas are freshened as the closed HNC structure forces the freshwater flow to divert along other avenues thereby freshening the surrounding areas.

Sensitivity simulations (not included in this report) demonstrated the importance of the two GIWW structures. Reducing the size of the western structure reduces the freshwater inflow able to enter the Morganza levee system and thereby increases the salinities in the study area. Conversely reducing the size of the eastern GIWW at Larose structure reduces the amount of freshwater able to leave the system and therefore decreases the salinities in the study area. While navigational concerns require certain structure sizes for these two areas, those simulations exhibit the type of control the new levee system will provide the operators. The complete plan comparison report can be furnished upon request.

The systemwide model did not include sea level rise. The systemwide model is currently being converted from TABS-MDS to ADH to investigate all RSLR cases.

2.7.2   Environmental Water Quality Assessment

2.7.2.1   Affected Environment

2.7.2.1.1 Introduction

This resource is institutionally significant because of the Clean Water Act, as amended, the Pollution Prevention Act, the Safe Drinking Water Act, and the Water Resources Planning Act. This resource is technically significant to restore and maintain the chemical, physical, and biological integrity of the Nation's waters. This resource is publicly significant because of the desire for clean water and water related activities such as boating, swimming, fishing, and as a source of potable water for human and animal consumption.

2.7.2.1.1.1   **Study Area Description**

The study area (Figure 85), located approximately 60 miles southwest of New Orleans in southeast Louisiana, covers approximately 1,891 square miles and includes portions of Lafourche and Terrebonne Parish. It is bounded by the Louisiana State Highway 311 and Bayou du Large to the west, Bayou Lafourche to the north and east, and the Gulf of Mexico to the south. The area includes the lower reach of the Terrebonne Basin, and is included in the abandoned Lafourche delta lobe. It includes barrier islands, open water, marsh, cypress and tupelo swamp, bottomland hardwood forest, farmland, and industry, residential, and other developed areas. There is very little topographical relief in the area. Like much of southeast Louisiana, the area contains many abandoned distributaries of the Mississippi River and their associated natural levees. In addition, the study area includes some of the most heavily used navigation waterways in the state of Louisiana (the Houma Navigation Canal and the Gulf Intracoastal Waterway).

Figure 85 - Study area and project features

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DRAFT ENGINEERING APPENDIX**

Development within the study area occurs along alluvial ridges, with the most significant development occurring in the cities of Houma and Thibodeaux. Undeveloped areas reveal characteristics of a transgressive delta lobe- wetlands which transition from swamp to marsh with increasing salinity gradient with proximity to the Gulf of Mexico, below which are several narrow, low profile barrier islands which are separated from the mainland marshes by a growing expanse of shallow coastal lakes and bays.

2.7.2.1.1.2 Project Description

The proposed project includes the construction of 77 miles of earthen levees, 21 floodgate structures proposed for the navigable waterways, 21 environmental water control structures, and a lock on the Houma Navigational Canal near Dulac (Figure 85). The proposed sector gates and tidal exchange structures will help reduce saltwater intrusion, reduce flood damage, minimize adverse impacts on commercial navigation, and allow water to flow between the interior and exterior marshes of the system. Approximately 61 of the 77 miles of the proposed levee alignment will follow existing hydraulic barriers such as natural ridges, roadbeds, and levees, so as to avoid generating unnecessary impacts to the ecosystem within the study area from the construction of hurricane risk reduction features.

For levee construction, adjacent borrow material will be used during the first of 3 to 4 levee lifts for levee reaches B, E, F, G, H, I, J, and K. Borrow material will be excavated using a bucket dredge, with the top 5 feet of organic material unsuitable for use as levee fill being placed on the flood side of the levee for marsh construction, and the underlying 15 ft of material being stockpiled for drying, and then placed according to Hurricane Storm Damage Risk Reduction System compaction criteria. Borrow material for subsequent lifts would be hauled in from offsite borrow sources.

2.7.2.1.1.3 Study Area Water Quality Influences

Water quality in the study area is a factor of area topography/bathymetry, water budget, coastal processes, local climate, tropical activity, and human activities. Study area elevation is predominated by a gradual downward slope with proximity to the Gulf of Mexico. Prominent topographic/bathymetric features include several abandoned Mississippi River distributaries and associated ridges, oil exploration and navigation canals and associated spoil banks, coastal lakes and bays, coastal wetlands, open water areas associated with deteriorated wetlands, barrier islands, and developed upland terrain.

Major water sources for the study area include the Gulf of Mexico, the Atchafalaya and

Mississippi Rivers, and precipitation. Major conduits for water exchange between the Gulf of Mexico and the study area include Falgout Canal, Bayou Grand Caillou, Houma Navigation Canal, Bayou Petite Caillou, and Bayou Terrebonne. The Gulf Intracoastal Waterway traverses the study area, and is a conduit for Atchafalaya River water (USGS 2003). Normal annual precipitation in the vicinity of the study area is 62 inches per year, with monthly averages ranging between 3.6 inches in April to 7.9 inches in July (NOAA 2012). Tropical storms and hurricanes produce significant time condensed rainfall totals that can cause major flooding and disrupt trends in water quality for affected areas.

Point and nonpoint water quality inputs to the basin include those occurring in the study area and in the Mississippi and Atchafalaya River watersheds, as the study area receives water input from these rivers. Primary point sources originating within the study area include package plant or other permitted small flows discharges, onsite treatment systems, total retention domestic sewage lagoons, municipal point source discharges, marina/boating sanitary on vessel discharges, industrial point source discharges, petroleum/natural gas activities, and sanitary sewer overflows (LDEQ 2011). Primary nonpoint sources originating within the study area include urban runoff/storm sewers, managed pasture grazing, non irrigated crop production, rangeland grazing, and municipal nonpoint sources (urbanized high density areas). The most significant nonpoint pollution sources for the Mississippi and Atchafalaya rivers include pesticide and fertilizer application, while primary point sources include industrial and municipal point source discharges.

Coastal processes and activities also affect study area water quality. Coastal wetlands have a significant effect on study area water quality, and are affected by these processes and activities. Wetlands have the ability to remove constituents such as nutrients, suspended sediments, organic matter, and metals from the water column. Wetlands have the ability to act as a permanent sink for these constituents through burial into substrate or release into the atmosphere. They also serve as a beneficial or detrimental source for constituents, and can change from a sink to a source for items such as nutrients or organic carbon through alterations to nutrient loading, hydrologic regime, burning, and vegetation change (Johnson 2004). The amount of time a wetland has been subjected to chemical loading can also influence whether it is a source or sink for certain constituents. For example, chronic loadings of elevated concentrations can result in saturation for a particular chemical after a number of years (Mitsch and Gosselink 2000). Wetlands modification of water column constituent concentrations varies seasonally with changes in wetland plant metabolism, species distribution, and density.

Wetland area within the study area has been decreasing in part as a result of the abandonment of the Lafourche delta lobe. Coastal processes such as erosion (day to day and storm induced) and subsidence are natural processes which promote wetland

loss in a decaying delta. Wetland area in the study area has been declining at an increased rate over the past century, which is likely a result of anthropogenic factors such as hydrologic modification, sea level rise, freshwater and sediment deprivation, shoreline erosion, herbivory, localized salinity intrusion/salinity stress, and hydrocarbon extraction (Steyer et. al 2008; Couvillion et. al 2011).

2.7.2.1.2 Methods, Criteria, and Guidelines for Evaluation of Sediment and Water Quality

2.7.2.1.2.1   Water Quality

2.7.2.1.2.2   Louisiana Water Quality Inventory

The Clean Water Act (CWA) established a process for states to develop information on the quality of their water resources. Section 305(b) requires that each state develop a program to monitor the quality of its surface and groundwater and prepare a report describing the status of its water quality. Section 303(d) requires states to list impaired waterbodies where water quality standards are not met and designated uses are not fully supported, and to develop a Total Maximum Daily Load (TMDL) for those waterbodies. The Louisiana Water Quality Inventory Report- Integrated Report, prepared by the Louisiana Department of Environmental Quality (LDEQ), is the current form of biennial reporting of the status of Louisiana waters in accordance with CWA sections 305(b) and 303(d).

For the purpose of water quality monitoring and assessment and development of TMDLs, Louisiana is divided into twelve major watershed basins, and each basin is further divided into waterbody subsegments. This subsegment approach divides the state's waters into discrete hydrologic units. The waterbody subsegment system within each watershed basin provides a workable framework to evaluate the State's waters. Subsegments are periodically added or removed as water quality standards related to a subsegment or group of subsegments are revised.

Section 305(b) of the Clean Water Act requires, among other items, a water quality assessment for each subsegment, which includes a description of each subsegment and the extent to which their waters provide for the protection and propagation of fish and wildlife and allow for recreational activities in and on the water (USEPA 2011). All assessments are prepared using existing and readily available water quality data and information in order to comply with rules and regulations under Section 305(b) of the Clean Water Act.

Subsequently, Section 303(d) of the Clean Water Act requires the identification, listing, and ranking for development of Total Maximum Daily Loads (TMDLs) for waters that do not meet applicable water quality standards after implementation of technology based controls. By definition, a TMDL is the maximum amount of a pollutant that can be discharged into a water body from all sources (both point and non point) and still maintain water quality standards.

Louisiana Water Quality Standards (LAC 33-IX.1123) define eight designated uses for surface waters, including- primary contact recreation; secondary contact recreation; fish and wildlife propagation; drinking water supply; oyster propagation; agriculture; outstanding natural resource; and limited aquatic life and wildlife use. Principal designated uses for Louisiana waterbodies include primary contact recreation, secondary contact recreation, and fish and wildlife propagation. The definitions for these primary uses are-

Primary Contact Recreation—any recreational or other water contact activity involving prolonged or regular full body contact with the water and in which the probability of ingesting appreciable amounts of water is considerable. Examples of this type of water use include swimming, skiing, and diving.

Secondary Contact Recreation—any recreational or other water contact activity in which prolonged or regular full body contact with the water is either incidental or accidental, and the probability of ingesting appreciable amounts of water is minimal. Examples of this type of water use include fishing, wading, and boating.

Fish and Wildlife Propagation—the use of water for aquatic habitat, food, resting, reproduction, cover, and/or travel corridors for any indigenous wildlife and aquatic life species associated with the aquatic environment. This use also includes the maintenance of water quality at a level that prevents damage to indigenous wildlife and aquatic life species associated with the aquatic environment and contamination of aquatic biota consumed by humans. The use subcategory of limited aquatic life and wildlife recognizes the natural variability of aquatic habitats, community requirements, and local environmental conditions. Limited aquatic life and wildlife use may be designated for water bodies having habitat that is uniform in structure and morphology, with most of the regionally expected aquatic species absent, low species diversity and richness, and/or a severely imbalanced trophic structure. Aquatic life able to survive and/or propagate in such water bodies includes species tolerant of severe or variable environmental conditions. Water bodies that might qualify for the limited aquatic life and wildlife use subcategory include intermittent streams, and naturally dystrophic and man made water bodies with characteristics including, but not limited to, irreversible hydrologic modification, anthropogenically and irreversibly degraded water quality, uniform channel morphology, lack of channel structure, uniform substrate, lack of

riparian structure, and similar characteristics making the available habitat for aquatic life and wildlife suboptimal.

Designated uses and criteria for each water body subsegment are listed in the Louisiana Water Quality Standards. Designated uses have a specific suite of ambient water quality parameters used to assess their support. Data and information collected from within or immediately downstream of a waterbody subsegment are used to evaluate each subsegment's designated uses. Where more than one parameter and criterion define a designated use, support for each use is defined by the designated use's poorest performing parameter (most severely impaired). Likewise, where data from more than one sample station are available, the most severely impaired station is used to make the assessment.

Following statistical determination of a water body's designated use support, along with a determination of the chemical parameters in the water body subsegment which might be impaired, a determination is then made as to which Integrated Report Category (IRC) the suspected water body impairment combination (WIC) should be placed in. A WIC is simply one impairment affecting one waterbody subsegment. Based on the IR Category, it is possible that either a TMDL is required, or has been completed, for a particular subsegment.

In addition to use of numerical data, LDEQ regional staff members are asked for input regarding significant suspected sources of impairment, or whether impairment due solely to natural sources is occurring. Numerical data alone can suggest impairment for some Louisiana water bodies when in fact there is no impairment or the impairment is due exclusively to natural causes. Using best professional judgment, regional staff familiar with the area suggest one or more suspected sources for a waterbody subsegment's impairment.

Total maximum daily loads (TMDLs) indicate that the majority of the pollutant load entering state waters comes from nonpoint sources of pollution; therefore, LDEQ is implementing a watershed based approach to reducing those loads in the water bodies where TMDLs have been completed. Presently, LDEQ utilizes both regulatory and non regulatory mechanisms to control nonpoint sources of pollution. Urban storm water for cities with populations of 50,000 or greater and construction sites of one acre or more are regulated through the LPDES permit program. Home sewage treatment systems are regulated through the LDHH. LDEQ's Water Quality Assessment Division (WQAD) currently houses the state's Nonpoint Source Management Program, which has been successful in implementing voluntary programs for forestry and agricultural sources of pollution. This has been done through coordination with other concerned agencies, such as the Louisiana Department of Agriculture and Forestry (LDAF), the U.S. Natural Resource Conservation Service (NRCS), and the Louisiana State University (LSU) AgCenter. LDEQ will continue to monitor state waters through the four year cyclic

process to determine whether the current implementation strategy is successful in restoring and maintaining water quality and the designated uses within Louisiana.

2.7.2.1.2.3   Louisiana Pollutant Discharge Elimination System (LPDES)

Louisiana's water quality regulations require permits for the discharge of pollutants from any point source into waters of the state of Louisiana. This surface water discharge permitting system is administered under the Louisiana Pollutant Discharge Elimination System (LPDES) program.

LPDES permits are official authorization developed and promulgated by the Office of Environmental Services of LDEQ. The LPDES permit establishes the wasteload content of wastewaters discharged into waters of the state. The permitting process allows the state to control the amounts and types of wastewaters discharged into its surface waters, in order to meet water quality standards. In 1996 LDEQ assumed responsibility for administering the permitting, compliance, and enforcement activities of the National Pollutant Discharge Elimination System (NPDES) from the U.S. Environmental Protection Agency (USEPA).

2.7.2.1.2.4   Louisiana Nonpoint Source Management Plan

Nonpoint source pollution is a type of pollution which is generated during rainfall events, and includes, among other things, agricultural and urban runoff. Section 319 of the Clean Water Act requires that state develop a nonpoint source management plan to reduce and control nonpoint sources of pollution from the various types of land uses that contribute to water quality problems across the United States. Louisiana has determined that agriculture, forestry, urban runoff, home sewage systems, sand and gravel mining, construction, and hydromodification all contribute to nonpoint source pollution problems across the state. Nonpoint source pollution is the largest remaining type of water pollution that needs to be addressed within Louisiana, and across the nation, in order to restore full support for designated uses of impaired waterbodies.

Louisiana's Nonpoint Source Program is managed by the LDEQ, and the goal of the program is to provide education regarding nonpoint source pollution and nonpoint source pollution prevention. The state of Louisiana has applied for and received Section 319 funds to implement both statewide and watershed projects to address nonpoint source pollution.

2.7.2.1.2.5 **Water Quality Criteria**

Water quality criteria are elements of state water quality standards expressed as constituent concentrations, levels, or narrative statements representing the quality of water supporting a particular designated use. When criteria are met, water quality will protect the designated use. Louisiana has both general and numeric criteria in LAC 33-IX.1113. General criteria are expressed in a narrative form (in concise statements) and include aesthetics, color, suspended solids, taste and odor, toxic substances (in general), oil and grease, foam, nutrients, turbidity, flow, radioactive materials, and biological and aquatic community integrity. Numeric criteria are generally expressed as concentrations (e.g., weight measured per liter) or scientific units and include pH, chlorides, sulfates, total dissolved solids, dissolved oxygen, temperature, bacteria, and specific toxic substances.

The USEPA has published national criteria recommendations for a number of substances, and states may incorporate these without modifications into their water quality standards. However, while states generally use USEPA guidance and recommendations in developing and adopting their own criteria, they are allowed the flexibility to develop their own methodology as well. USEPA guidance is under continuous development and revision. States review and incorporate these developments and revisions into their water quality standards as appropriate.

Human health criteria provide guidelines that specify the potential risk of adverse effects to humans due to substances in the water. Factors considered include body weight, risk level, fish consumption, drinking water intake, and incidental ingestion while swimming. Categories of criteria are then developed for each toxic substance for public drinking water supply, non drinking water (swimming), and non swimming water.

Aquatic life criteria are designed to protect all aquatic life, including plants and animals. There are two types of criteria- acute, for short term exposures (e.g., spills); and chronic for long term or permanent exposures. One or both of the acute and chronic criteria may be related to other water quality characteristics, such as pH, temperature, or hardness. Separate criteria are also developed for fresh and salt waters. The federal water quality standards regulations allow states to develop numerical criteria or modify USEPA's recommended criteria to account for site specific or other scientifically defensible factors.

2.7.2.1.2.6 **Sediment Quality**

The National Oceanic and Atmospheric Administration (NOAA) has developed Screening Quick Reference Tables, or SQuiRTs, to help evaluate potential risks from contaminated water, sediment, or soil (NOAA 2008). The suite of sediment screening benchmarks provided in these tables are for different biological and toxicological

endpoints, some of which (such as PEL, T50, and ERM screening values) represent levels at which a high probability of adverse effects or toxicity for benthic organisms exposed to whole sediment are expected. Further definitions of screening benchmarks are available at:
http://response.restoration.noaa.gov/faq_topic.php?faq_topic_id=6#tecpec.

Although they are not regulated, these benchmarks provide a useful means of assessing sediment chemistry to determine whether or not acute or chronic exposure of benthic organisms to whole sediments may result in adverse biological effects or mortality.

2.7.2.1.3 Study Area Historical Water Quality

2.7.2.1.3.1 Louisiana Water Quality Inventory

In order to assess historical water quality of the study area, a review of historical water quality inventories for waterbody subsegments within the study area was conducted. Table 75 and Figure 86 depict all subsegments included in the study area.

Table 75 - Study area subsegments

Subsegment	Subsegment Description
120202	Bayou Black-From ICWW to Houma
120203	Bayou Boeuf-From Lake Palourde to ICWW
120207	Thibodaux Swamp-Forested wetland located in Lafourche and Terrebonne Parishes, 6.2 miles southwest of Thibodaux, east of Terrebonne-Lafourche Drainage Canal, and north of Southern Pacific Railroad; also called Pointe Au Chene Swamp
120301	Bayou Terrebonne-From Thibodaux to ICWW in Houma
120302	Bayou Folse-From headwaters to Company Canal
120303	Bayou L'Eau Bleu-From Company Canal to ICWW
120304	Intracoastal Waterway-From Houma to LaRose
120403	Intracoastal Waterway-From Bayou Boeuf Locks to Bayou Black in Houma; includes segments of Bayous Boeuf, Black and Chene
120405	Lake Hache and Lake Theriot
120406	Lake de Cade
120501	Bayou Grand Caillou-From Houma to Bayou Pelton
120502	Bayou Grand Caillou-From Bayou Pelton to Houma Navigation Canal (Estuarine)
120503	Bayou Petit Caillou-From Bayou Terrebonne to LA-24 bridge
120504	Bayou Petit Caillou-From LA-24 bridge to Boudreaux Canal (Estuarine)
120505	Bayou Du Large-From Houma to Marmande Canal
120506	Bayou Du Large-From Marmande Canal to 1/2 mile north of St. Andrews Mission (Estuarine)
120507	Bayou Chauvin-From Ashland Canal to Lake Boudreaux (Estuarine)
120508	Houma Navigation Canal-From Bayou Pelton to 1 mile south of Bayou Grand Caillou (Estuarine)
120509	Houma Navigation Canal-From Houma to Bayou Pelton
120601	Bayou Terrebonne-From Houma to Company Canal (Estuarine)
120602	Bayou Terrebonne-From Company Canal to Humble Canal (Estuarine)
120603	Company Canal-From ICWW to Bayou Terrebonne
120604	Bayou Blue-From ICWW to Grand Bayou Canal
120605	Bayou Pointe au Chien-From headwaters to St. Louis Canal
120606	Bayou Blue-From Grand Bayou Canal to Bully Camp Canal (Estuarine)
120701	Bayou Grand Caillou-From Houma Navigation Canal to Caillou Bay (Estuarine)
120702	Bayou Petite Caillou-From Boudreaux Canal to Houma Navigation Canal (Estuarine)
120704	Bayou Terrebonne-From Humble Canal to Lake Barre (Estuarine)
120705	Houma Navigation Canal-From 1/2 mile south of Bayou Grand Caillou to Terrebonne Bay (Estuarine)
120706	Bayou Blue-From Bully Camp Canal to Lake Raccourci (Estuarine)
120707	Lake Boudreaux
120709	Bayou Petite Caillou-From Houma Navigation Canal to Terrebonne Bay
120801	Caillou Bay
120802	Terrebonne Bay
120803	Timbalier Bay
120804	Lake Barre
120805	Lake Pelto

Clean Water Act Section 305(b) listings of study area subsegments, from 1996 to 2010, were included in the review. For each subsegment, an average designated use support value was calculated. The calculated average support values were a function of designated use and level of support. Support levels for each combination of subsegment, year, and designated use were as follows-

0- subsegment not supporting designated use
1- subsegment fully supporting designated use

The average support value calculated for each subsegment serves as a simplistic representation for waterbody subsegment health with respect to designated uses (with

zero being the least healthy value possible, and one being the most). In order to develop a visual representation of the long term health of each subsegment with respect to designated uses, the average support values for subsegments were color coded in 20th percentile increments. Table 76 and Figure 86 illustrate the average support values for each subsegment. For Table 76, subsegments were grouped based on their predominant orientation with respect to the proposed MTG project levee alignment (flood or protected side) to illustrate the overall disparity in waterbody health with respect to designated uses. A significant portion (15 of 16) of the subsegments in the lowest 40th percentile are predominantly located within the proposed levee alignment. In general, this disparity is present because the portion of the study area north of the proposed levee alignment is more developed (and therefore subject to a higher volume of point and nonpoint sources of pollution), while the portion south of the proposed levee alignment is more tidally influenced (and therefore frequently flushed with ocean water).

Table 76 - 1996-2010 average support values

Flood or Protected Side	Subsegment	Average of Support, 1996-2010
Flood	120203	0.957
	120403	0.818
	120405	0.722
	120406	0.957
	120506	0.522
	120701	0.806
	120702	0.680
	120704	0.720
	120705	0.920
	120706	0.920
	120709	0.840
	120801	0.913
	120802	0.957
	120803	0.913
	120804	0.913
	120805	0.870
Protected	120202	0.548
	120207	1
	120301	0.208
	120302	0.553
	120303	0.667
	120304	0.789
	120501	0.444
	120502	0.548
	120503	0.522
	120504	0.387
	120505	0.556
	120507	0.333
	120508	0.742
	120509	0.839
	120601	0.500
	120602	0.387
	120603	0.944
	120604	0.556
	120605	0.444
	120606	0.556
	120707	0.739

= 0-20th Percentile
= 20th-40th Percentile
= 40th-60th Percentile
= 60th-80th Percentile
= 80th-100th Percentile
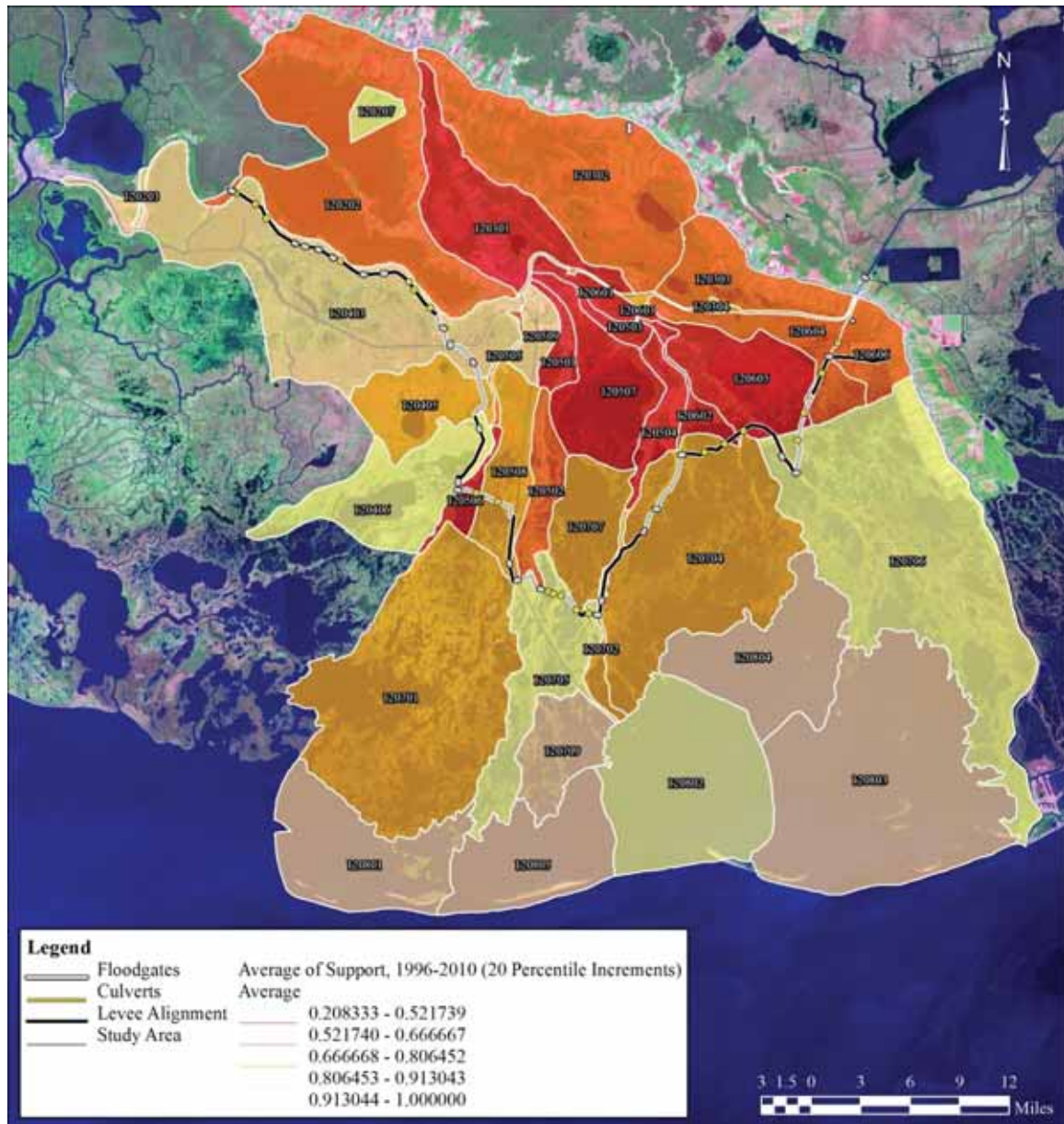
Figure 86 - Map depicting average support values superimposed by project features

In order to both determine the most prevalent water quality issues present in the study area and determine which water quality parameters should be summarized for the depiction of historical water quality for the study area, the same collection of historical Clean Water Act Section 305(b) listings was reviewed to determine the most prevalent historical causes and sources of subsegment impairment (Table 77 and Table 78). Between 1996 and 2010, the most common suspected cause of impairment was low dissolved oxygen, followed by Fecal Coliform, non native aquatic plants, total phosphorus, nitrate plus nitrite nitrogen, and nutrients, while the most common

suspected source of impairment was package plant of other permitted small flows discharges, followed by introduction of non native organisms, on site treatment systems, total retention domestic sewage lagoons, unknown sources, and natural sources. When subsegments are grouped based on their predominant orientation with respect to the proposed level alignment, leading suspected causes between flood and protected side remain similar, while sources differ (for example, introduction of non native organisms is a more significant suspected source north of the proposed levee alignment, which petroleum/natural gas activities is a more significant suspected source of impairment south of the proposed levee alignment).

Table 77 - Frequency of suspected causes of impairment, 1996-2010

Suspected Cause of Impairment	Count
Dissolved Oxygen	124
Fecal Coliform	114
Non-Native Aquatic Plants	80
Total Phosphorus	47
Nitrate/Nitrite	47
Nutrients	37
Chloride	31
Oil and Grease	31
Turbidity	23
Mercury	23
Priority Organics	20
Metals	19
Total Dissolved Solids	15
Sulfates	11
Total Suspended Solids	10
Radiation	9
Flow Alteration	7
Pesticides	5
Taste and Odor	5
Siltation	5
Habitat Alteration	5
pH	4
Color	2
Unknown	2
Arsenic	2
Nonpriority Organics	2
Total Toxics	1

Flood or Protected Side	Suspected Cause of Impairment	Count
Flood	Fecal Coliform	37
	Dissolved Oxygen	21
	Nutrients	16
	Priority Organics	14
	Oil and Grease	14
	Non-Native Aquatic Plants	13
	Metals	10
	Mercury	9
	Radiation	9
	Chloride	8
	Turbidity	7
	Habitat Alteration	3
	pH	3
	Total Suspended Solids	2
	Total Phosphorus	2
	Nitrate/Nitrite	2
	Taste and Odor	2
	Flow Alteration	1
	Siltation	1
Protected	Dissolved Oxygen	103
	Fecal Coliform	77
	Non-Native Aquatic Plants	67
	Nitrate/Nitrite	45
	Total Phosphorus	45
	Chloride	23
	Nutrients	21
	Oil and Grease	17
	Turbidity	16
	Total Dissolved Solids	15
	Mercury	14
	Sulfates	11
	Metals	9
	Total Suspended Solids	8
	Flow Alteration	6
	Priority Organics	6
	Pesticides	5
	Siltation	4
	Taste and Odor	3
	Arsenic	2
	Habitat Alteration	2
	Unknown	2
	Nonpriority Organics	2
	Color	2
	Total Toxics	1
	pH	1

Table 78 - Frequency of suspected sources of impairment, 1996-2010

Suspected Source of Impairment	Count
Package Plant or Other Permitted Small Flows Discharges	121
Introduction of Non-native Organisms (Accidental or Intentional)	85
On-site Treatment Systems (Septic Systems and Similar Decentralized Systems)	80
Total Retention Domestic Sewage Lagoons	67
Source Unknown	64
Natural Sources	54
Municipal Point Source Discharge	50
Marina/Boating Sanitary On-vessel Discharges	43
Industrial Point Source Discharge	42
Petroleum/Natural Gas Activities	40
Urban Runoff/Storm Sewers	32
Other Spill Related Impacts	30
Managed Pasture Grazing	29
Sanitary Sewer Overflows (Collection System Failures)	27
Non-irrigated Crop Production	16
Forced Drainage Pumping	16
Channelization	15
Upstream Source	15
Natural Conditions - Water Quality Standards Use Attainability Analyses Needed	15
Drought-Related Impacts	12
Rangeland Grazing	12
Unspecified Land Disturbance	11
Flow Alterations from Water Diversions	11
Municipal (Urbanized High Density Area)	10
Wildlife Other than Waterfowl	9
Unspecified Domestic Waste	9
Contaminated Sediments	9
Combined Sewer Overflows	8
Dredging	8
Sewage Discharges in Unsewered Areas	8
Agriculture	8
Atmospheric Deposition - Toxics	8
Site Clearance (Land Development or Redevelopment)	7
Changes in Tidal Circulation/Flushing	7
Unspecified Urban Stormwater	6
Industrial/Commercial Site Stormwater Discharge (Permitted)	6
Streambank Modifications/Destabilization	5
Waterfowl	4
Sources Outside State Jurisdiction or Borders	4
Non-Point Source	4
Unpermitted Discharge (Domestic Wastes)	3
Sediment Resuspension (Clean Sediment)	3
Seafood Processing Operations	2
Hazardous Waste	2
Internal Nutrient Recycling	2
Hydromodification	2
Changes in Ordinary Stratification and Bottom Water Hypoxia/Anoxia	2
Aquaculture	1
Drainage/Filling/Loss of Wetlands	1

Flood or Protected Side	Suspected Source of Impairment	Count
Flood	Package Plant or Other Permitted Small Flows Discharges	29
	Petroleum/Natural Gas Activities	24
	Source Unknown	24
	Marina/Boating Sanitary On-vessel Discharges	19
	Industrial Point Source Discharge	16
	Managed Pasture Grazing	15
	Total Retention Domestic Sewage Lagoons	15
	Other Spill Related Impacts	15
	On-site Treatment Systems (Septic Systems and Similar Decentralized Systems)	15
	Natural Sources	15
	Introduction of Non-native Organisms (Accidental or Intentional)	13
	Municipal Point Source Discharge	9
	Atmospheric Deposition - Toxics	7
	Urban Runoff/Storm Sewers	7
	Upstream Source	7
	Channelization	6
	Rangeland Grazing	6
	Non-irrigated Crop Production	6
	Sewage Discharges in Unsewered Areas	5
	Agriculture	5
	Dredging	5
	Natural Conditions - Water Quality Standards Use Attainability Analyses Needed	4
	Flow Alterations from Water Diversions	4
	Industrial/Commercial Site Stormwater Discharge (Permitted)	4
	Contaminated Sediments	4
	Combined Sewer Overflows	3
	Waterfowl	3
	Sources Outside State Jurisdiction or Borders	3
	Internal Nutrient Recycling	3
	Changes in Ordinary Stratification and Bottom Water Hypoxia/Anoxia	2
	Site Clearance (Land Development or Redevelopment)	2
	Streambank Modifications/Destabilization	2
	Wildlife Other than Waterfowl	2
	Unspecified Domestic Waste	2
	Unspecified Urban Stormwater	2
	Hazardous Waste	1
	Non-Point Source	1
Protected	Package Plant or Other Permitted Small Flows Discharges	92
	Introduction of Non-native Organisms (Accidental or Intentional)	72
	On-site Treatment Systems (Septic Systems and Similar Decentralized Systems)	65
	Total Retention Domestic Sewage Lagoons	52
	Municipal Point Source Discharge	41
	Source Unknown	40
	Natural Sources	39
	Sanitary Sewer Overflows (Collection System Failures)	27
	Industrial Point Source Discharge	26
	Urban Runoff/Storm Sewers	25
	Marina/Boating Sanitary On-vessel Discharges	24
	Forced Drainage Pumping	16
	Petroleum/Natural Gas Activities	16
	Other Spill Related Impacts	15
	Managed Pasture Grazing	14
	Drought-Related Impacts	12
	Unspecified Land Disturbance	11
	Natural Conditions - Water Quality Standards Use Attainability Analyses Needed	11
	Municipal (Urbanized High Density Area)	10
	Non-irrigated Crop Production	10
	Channelization	9
	Upstream Source	8
	Unspecified Domestic Waste	7
	Flow Alterations from Water Diversions	7
	Changes in Tidal Circulation/Flushing	7
	Wildlife Other than Waterfowl	7
	Rangeland Grazing	6
	Contaminated Sediments	5
	Combined Sewer Overflows	5
	Site Clearance (Land Development or Redevelopment)	5
	Unspecified Urban Stormwater	4
	Streambank Modifications/Destabilization	3
	Agriculture	3
	Sediment Resuspension (Clean Sediment)	3
	Non-Point Source	3
	Unpermitted Discharge (Domestic Wastes)	3
	Dredging	3
	Sewage Discharges in Unsewered Areas	3
	Hydromodification	2
	Seafood Processing Operations	2
	Industrial/Commercial Site Stormwater Discharge (Permitted)	2
	Drainage/Filling/Loss of Wetlands	1
	Sources Outside State Jurisdiction or Borders	1
	Waterfowl	1
	Hazardous Waste	1
	Aquaculture	1
	Atmospheric Deposition - Toxics	1

2.7.2.1.3.2   **Historical Water Quality Monitoring**

In addition to review and summary of the historical Clean Water Act Section 305(b) listings for the study area, historical water quality monitoring data was reviewed and summarized to determine water quality trends in the study area.

Four (4) LDEQ long term water quality monitoring stations exist in the study area, as depicted in Table 79 and Figure 95. Stations are increasing in salinity gradient from station 37, 113, 110, to 114, with station 37 being the only station on the flood side of the proposed levee alignment. In general, water quality monitoring data for these stations was collected monthly until 1998, and thereafter was collected monthly for one out of every four years.

Table 79 - LDEQ long term water quality monitoring station descriptions

Station ID	Latitude	Longitude	Station Description	Monitoring Interval Begin	End
37	29.233329	-90.666674	Houma Navigation Canal south of Cocodrie, Louisiana	3/6/1978	12/10/1990
110	29.598606	-90.718614	Bayou Terrebonne at Houma, Louisiana	6/1/1958	8/4/2008
113	29.382771	-90.715279	Bayou Grand Caillou at Dulac, Louisiana	3/6/1978	8/3/2011
114	29.684717	-90.998618	Bayou Black at Gibson, Louisiana	6/1/1958	2/1/2005

Figure 87 through Figure 92 depict long term water quality monitoring results and trends for these stations, for the parameters most frequently cited as suspected causes of impairment for subsegments included in the study area.

For dissolved oxygen (Figure 87), trendlines for all stations indicate that dissolved oxygen levels have improved for all sites (although, as with this and most other parameters, r-squared values for all stations are generally less than .1). Trendlines for Fecal Coliform levels (Figure 88 and Figure 89) for all stations except 114 indicate decreasing trends in Fecal Coliform levels. Station 114 had several high measurements of Fecal Coliform density between 1993 and 1998, which might explain the positive slope of the trendline for this station. Overall, mildly decreasing trends are observed for total phosphorus, for all stations (Figure 90). Kjeldahl nitrogen trendlines are consistently decreasing, and the trendline for station 110 has a relatively high r-squared value of .125 (Figure 92). Nitrate plus nitrite trendlines suggest very little change in long term nitrate plus nitrogen concentrations has occurred over the past thirty years at these stations (Figure 91). Overall, these figures suggest that dissolved oxygen and fecal coliform levels have improved within the past thirty years.
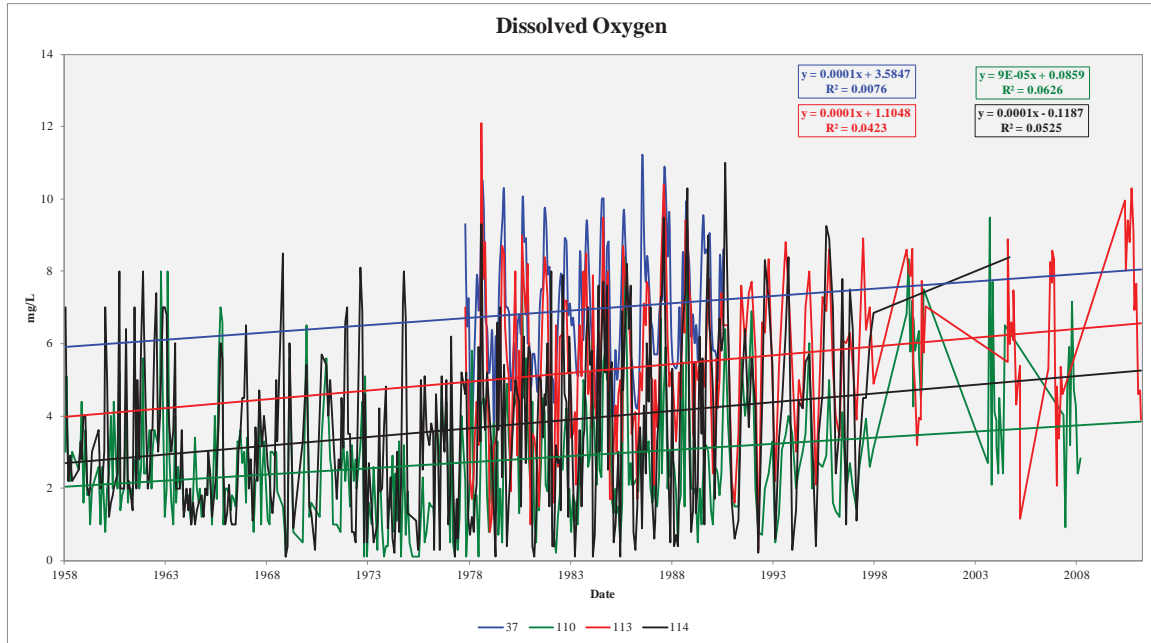
Figure 87 - Study area long term dissolved oxygen monitoring data plots and trends
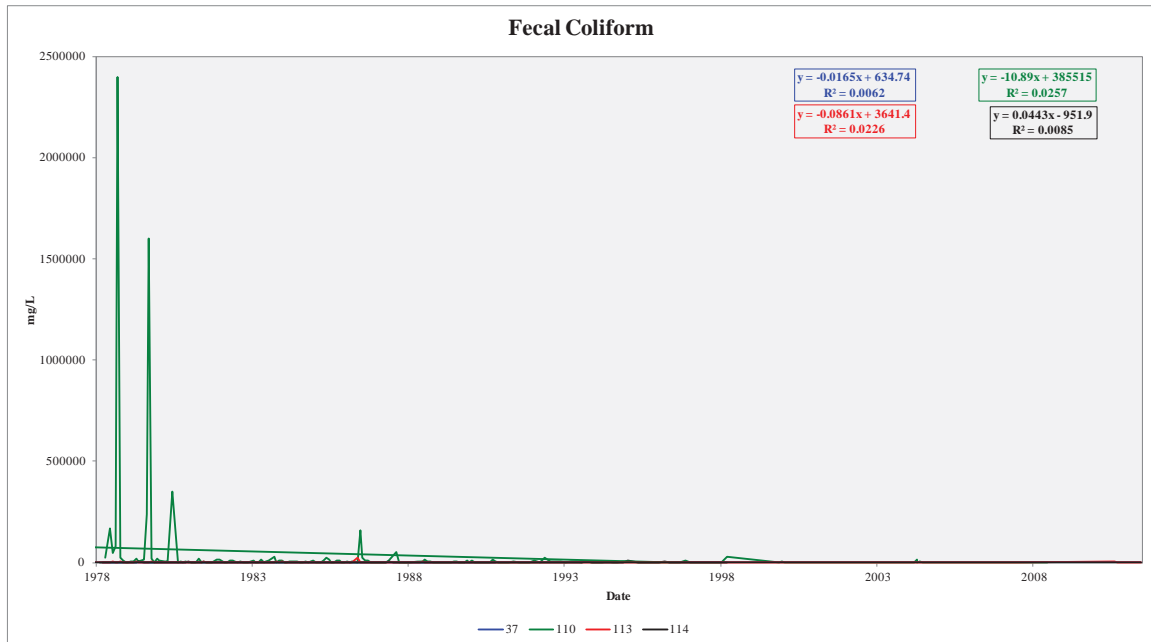
Figure 88 - Study area long term Fecal Coliform monitoring data plots and trends
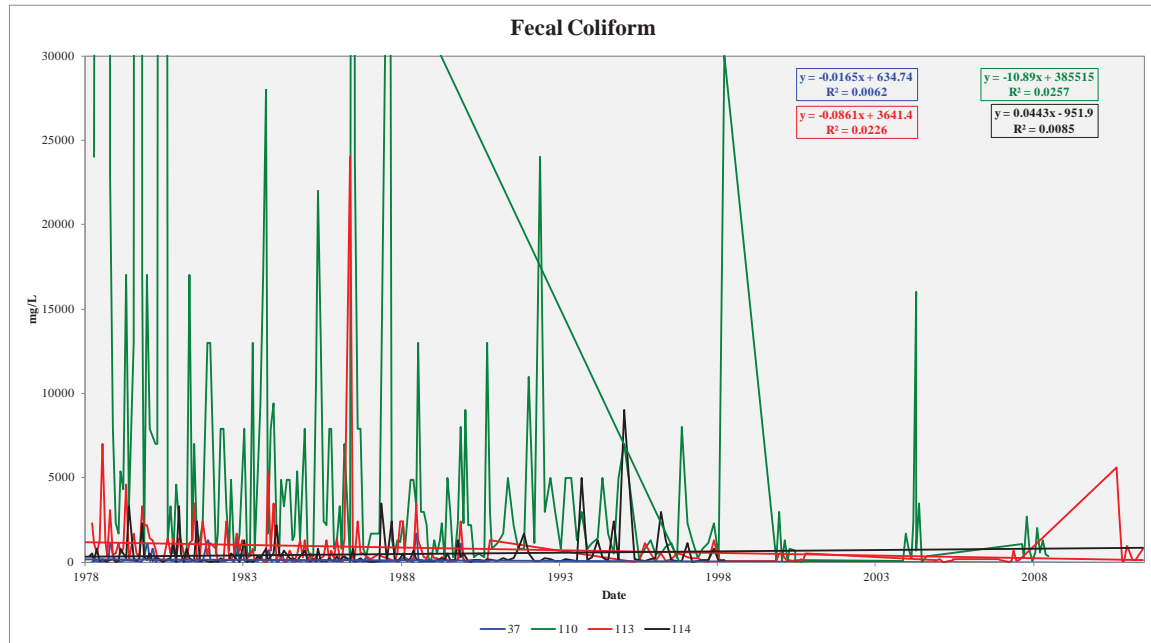
Figure 89 - Study area long term Fecal Coliform monitoring data plots and trends (reduced y-axis scale)
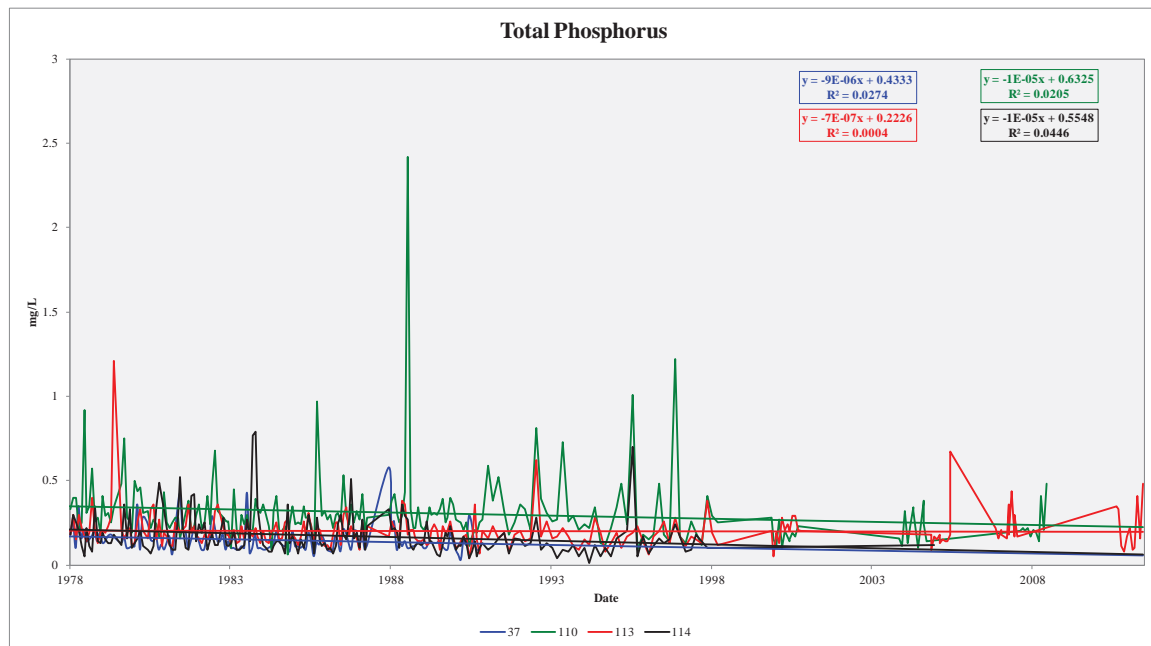
Figure 90 - Study area long term total phosphorus monitoring data plots and trends
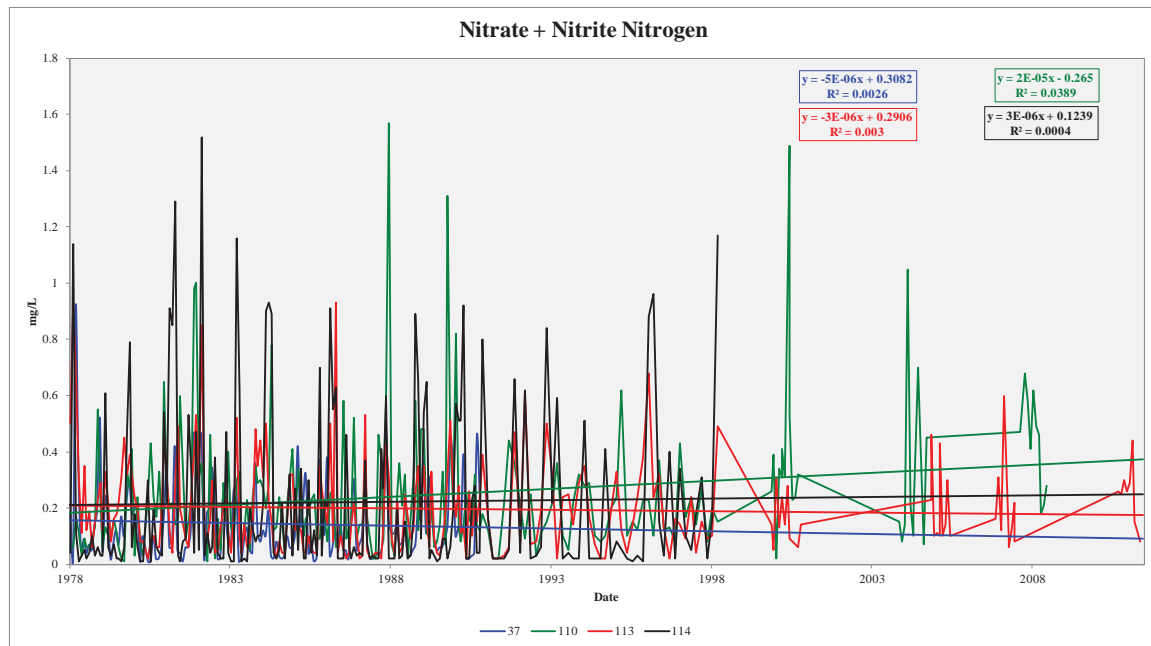
Figure 91 - Study area long term nitrate plus nitrite monitoring data plots and trends
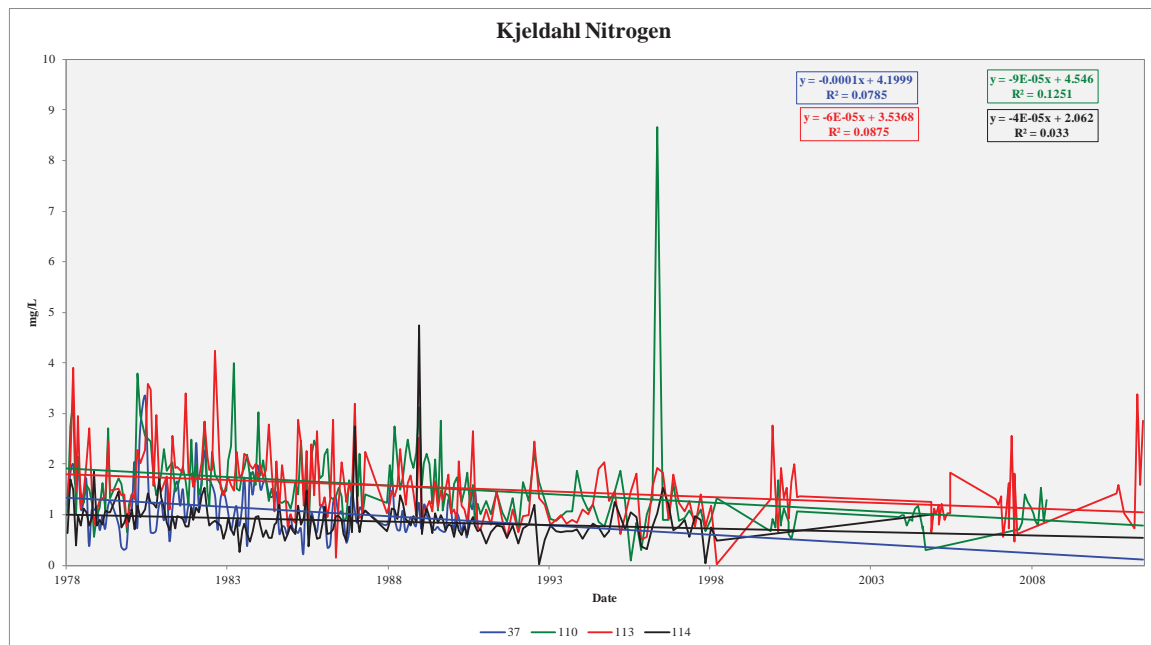
Figure 92 - Study area long term Kjeldahl nitrogen monitoring data plots and trends

2.7.2.1.4 Study Area Existing Water and Sediment Quality

2.7.2.1.4.1   Louisiana Water Quality Inventory

Figure 93 depicts overall subsegment support for the 2010 Louisiana Water Quality Inventory, for subsegments included in the study area. Subsegments shaded green are fully supporting all designated uses, while subsegments shaded orange are fully supporting some, but not all, designated uses. Consistent with long term trends, most subsegments fully supporting all designated uses are located on the flood side of the proposed levee alignment.
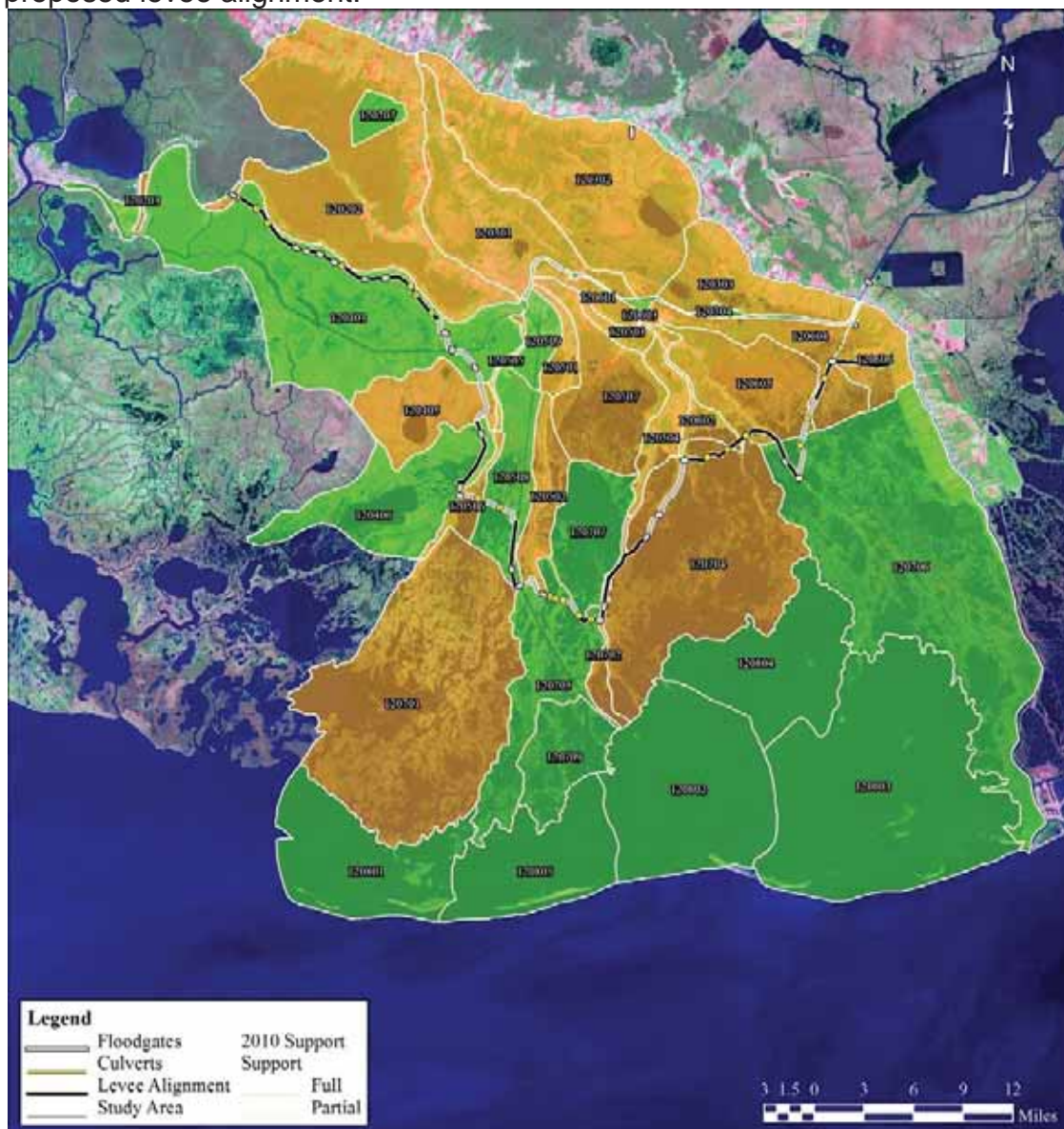
Figure 93 - 2010 subsegment designated use support

2.7.2.1.4.2   **LPDES Permitted Discharges**

Figure 94 depicts the locations of point source discharges permitted under the LPDES. Clearly, the majority of these permitted discharges occur on the protected side of the proposed levee alignment, in developed areas. Over 70% of these permitted discharges occur in the subsegments in the lowest 40th percentile with respect to average subsegment support; approximately 88% of the permitted discharges occur on the protected side of the proposed levee alignment.

Figure 94 - Locations of LPDES permitted discharges within the study area

2.7.2.1.4.3   Project specific Water and Sediment Sampling

In addition to the review of Clean Water Act Section 305(b) listings and historical water quality monitoring data, project specific sediment, water, and elutriate chemistry data was collected. Water and sediment samples were collected from a total of twelve (12) sites between January 31st and February 2nd, 2011 (see Table 80 and Figure 95).

Table 80 - Project specific water and sediment sampling sites

Station ID	Latitude	Longitude	Station Description	Sampling Date
Site 1	29.650000	-90.872500	Munson's World Famous Swamp Tours, north of Barrier Alignment	1/31/2011
Site 2	29.548056	-90.791111	Near canal with bridge crossing, 1/2 miles east of Minors Canal	1/31/2011
Site 3	29.417500	-90.784722	Canal by upper Bayou du Large pump station	1/31/2011
Site 4	29.335556	-90.843333	Floodgate near end of Bayou Dularge Road	2/1/2011
Site 5	29.389739	-90.733056	South of east end of Falgout Canal	2/1/2011
Site 6	29.384444	-90.729167	Houma Navigation Canal and Falgout Road	2/1/2011
Site 7	29.302222	-90.670000	Highway 57 northwest of Rabbit Bayou – location of proposed culvert with sluice gates	2/1/2011
Site 8	29.387500	-90.587778	Flood side of Mason Canal Road at proposed Bayou Terrebonne floodgate	2/1/2011
Site 9	29.437836	-90.565075	Near dock at Humble Canal, west of Humble Canal floodgate	2/1/2011
Site 10	29.430833	-90.587778	Pump station, Oak Point Road off of Highway 65	2/2/2011
Site 11	29.474122	-90.435028	Shoreline of Grand Bayou Canal at proposed Grand Bayou floodgate	2/2/2011
Site 12	29.543889	-90.402778	Off Highway 24 across from shipyard in GIWW, at proposed Grand Bayou floodgate	2/2/2011

Figure 95 - Project specific water and sediment sampling sites and LDEQ long term monitoring stations

The purpose of data collection was to ensure proposed dredged material disposal activities associated with the proposed project do not have adverse environmental effects on the receiving aquatic environment. Disposal of dredged material should not exceed State or Federal water quality criteria outside of the established mixing zone in order to comply with the section 404(b)(1) guidelines and in order to ensure 401 water quality certification. Evaluation of sediment chemistry was performed to determine whether sediment has the potential to result in mortality of mobile benthic organisms. Evaluation of water and elutriate chemistry is typically performed to determine whether the proposed discharge of dredged material effluent exceeds State and/or Federal water quality criteria outside of the State enforced mixing zone, and therefore may result in toxicity to water column organisms. Sample preparation and testing is performed in accordance with the Inland and/or Upland Testing Manual (USEPA 1998; USACE 2003), depending on the proposed dredged material disposal method.

Table 81 displays the chemical classes included in the analysis of sediment, water, and elutriates, the latter of which is a mixture of dredging site water and sediment at proportions intended to replicate those of hydraulic dredging. Up to five (5) herbicides, fourteen (14) inorganic/general chemistry parameters, twenty one (21) metals, twenty four (24) pesticides, seven (7) PCB congeners, nine (9) PAHs, fifty eight (58) semi volatile organic compounds, fifty four (54) volatile organic compounds, and total petroleum hydrocarbons were included in the analyses. As a disclaimer, analysis of elutriates for project specific sampling and analysis does not suggest adjacent borrow would be hydraulically placed for levee construction; in contrast, and as described in section 2.7.1.1.2, material would be mechanically excavated and dewatered prior to placement. Therefore, elutriate test results have little bearing on predicted water column impacts during placement of adjacent borrow for levee fill. In addition, the type of elutriate test conducted (modified elutriate or standard elutriate) was not specified in the laboratory report. In summary, the purpose and type of elutriate testing conducted for this project was not specified, however results of testing is being provided herein.

Table 81 - Chemical classes included in sediment, water, and elutriate analysis

Chemical Class	Sediment	Water	Elutriate
Herbicides	X	X	X
Inorganic/General Chemistry	X	X	X
Metals	X	X	X
Pesticides	X	X	X
Polychlorinated Biphenyls	X	X	X
Polycyclic Aromatic Hydrocarbons	X	X	X
Semi-Volatile Organic Compounds	X	X	X
Total Petroleum Hydrocarbons	X	X	X
Volatile Organic Compounds	X	X	

2.7.2.1.4.4 **Water and Elutriate Quality**

Water and elutriate chemistry data was compared with applicable State and Federal water quality criteria to determine whether results exceeded these criteria. Salinity data from LDEQ water quality monitoring stations in proximity to project specific sampling sites was used to estimate the salinity regime of these sites, in order to determine applicable water quality criteria (LDEQ water quality criteria exists for freshwater, brackish, and marine waters, while EPA water quality criteria exists for freshwater and marine waters). Table 82 through Table 85 below display exceedances of water quality criteria for water and elutriates. In most cases, values exceeding criteria are not measured values, but are instead estimates, as results were below the laboratory reporting limit (in other words, the concentration was below that which the laboratory could quantify with confidence).

For freshwater sites (Table 82 and Table 83), the only exceedances for measured values are for copper (Site 1 elutriate), iron (Site 1 elutriate, Site 2 water, Site 12 elutriate and water), lead (Site 1 elutriate, site 12 elutriate), and mercury (site 1 elutriate). These measured elutriate concentrations, which are for exceedances of chronic water quality criteria, are within one order of magnitude of criteria.

Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded acute criteria for cadmium, p,p'-DDD, and toxaphene, for all freshwater sites and both analytical mediums (water and elutriates), and chronic criteria for cadmium, mercury, p,p'-DDD, p,p'-DDT, endrin, heptachlor, heptachlor epoxide, methoxyclor, toxaphene, and hexachlorobutadiene for all freshwater sites and both analytical mediums.

Table 82 - Exceedances of water quality criteria for freshwater sites (excludes State hardness dependent metals criteria)

Chemical Class	Parameter	Units	Freshwater Site 1 Elutriate	Site 1 Water	Site 2 Elutriate	Site 2 Water	Site 12 Elutriate	Site 12 Water	LDEQ Freshwater Acute	LDEQ Freshwater Chronic	EPA Freshwater Acute	EPA Freshwater Chronic
Metals	Cadmium	µg/L	2.50	2.50	2.50	2.50	2.50	2.50			2	0.25
	Copper	µg/L	17.0	5.00	5.00	5.00	5.00	5.00				9
	Iron	µg/L	1,700	930	220	1,100	4,000	2,800				1,000
	Lead	µg/L	14.0	1.50	1.50	1.50	4.80	1.50			65	2.5
	Mercury	µg/L	0.220	0.100	0.100	0.100	0.100	0.100	2.04	0.012	1.4	0.77
Pesticides	DDD, p,p'-	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.03	0.006		
	DDT, p,p'-	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	1.1	0.001	1.1	0.001
	Endrin	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.086	0.0375	0.086	0.036
	Heptachlor	µg/L	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.52	0.0038	0.52	0.0038
	Heptachlor Epoxide	µg/L	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250			0.52	0.0038
	Methoxychlor	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500				0.03
	Toxaphene	µg/L	0.850	0.850	0.850	0.850	0.850	0.850	0.73	0.0002	0.73	0.0002
Semi-Volatile Organic Compounds	Hexachlorobutadiene	µg/L	5.00	3.75	5.00	3.75	5.00	3.75	5.1	1.02		

Table 83 - Exceedances of State hardness dependent metals criteria

Param...		Calcium		Copper		Lead		Nickel		Zinc		LDEQ Water Quality Criteria for Metals (Harness-Dependent for Freshwater/Brackish Criteria)										
		µg/L		µg/L		µg/L		µg/L		µg/L		Cadmium		Copper		Lead		Nickel		Zinc	
Salinity Regime	Station ID	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Acute	Chronic	Acute	Chronic	Acute	Chronic	Acute	Chronic	Acute	Chronic
Freshwater	Site 1	2.5	2.5	17	5	14	1.5	16	7.5	35	6	22.1	0.944	14.7	10.0	29.5	2.02	1,155	128	93.4	85.3

For brackish sites (Table 84 and Table 85), the only measured concentration exceeding criteria was for ammonia (Site 5, elutriate).

Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded acute criteria for copper, silver, p,p'-DDD, beta-endosulfan, endrin, toxaphene, and hexachlorobutadiene for all brackish sites and both analytical mediums, and chronic criteria for copper, mercury, silver, p,p'-DDD, p,p'-DDT, dieldrin, alpha-endosulfan, beta-endosulfan, endrin, heptachlor, heptachlor epoxide, methoxychlor, toxaphene, and hexachlorobutadiene for all brackish sites and both analytical mediums.

Table 84 - Exceedances of water quality criteria for brackish sites

| | | | Brackish | | | | | | | | | | | | | | | | | Water Quality Criteria | | | |
| | | | Site 3 | | Site 4 | | Site 5 | | Site 6 | | Site 8 | | Site 9 | | Site 10 | | | LDEQ Brackish | | EPA Marine | |
Chemical Class	Parameter	Units	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Elutriate	Water	Acute	Chronic	Acute	Chronic
Metals	Copper	µg/L	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00			4.3	3.1
	Mercury	µg/L	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	2.00	0.012	1.8	0.94
	Silver	µg/L	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50	2.50			1.9	
Pesticides	DDD, p,p'-	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.03	0.006		
	DDT, p,p'-	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.13	0.001	0.13	0.001
	Dieldrin	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.237	0.0019	0.71	0.0019
	Endosulfan, alpha-	µg/L	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.034	0.0087	0.034	0.0087
	Endosulfan, beta-	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500			0.034	0.0087
	Endrin	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.037	0.0023	0.037	0.0023
	Heptachlor	µg/L	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.063	0.0036	0.053	0.0036
	Heptachlor Epoxide	µg/L	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250	0.0250			0.053	0.0036
	Methoxychlor	µg/L	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500	0.0500				0.03
	Toxaphene	µg/L	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.21	0.0002	0.21	0.0002
Semi Volatile Organic Compounds	Hexachlorobutadiene	µg/L	5.00	3.75	5.00	3.75	5.00	3.75	5.00	3.75	5.00	3.75	5.00	3.75	5.00	3.75	1.6	0.32		

Table 85 - Exceedances of Federal criteria for ammonia

		Nitrogen, Ammonia			
		mg/L		Marine	
Salinity Regime	Station ID	Elutriate	Water	Acute	Chronic
Brackish	Site 5	3.90	0.066	25	3.7

For marine sites (Table 86), no exceedances of measured values were reported. Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded acute criteria for silver, beta-endosulfan, endrin, toxaphene, and hexachlorobutadiene for all marine sites and both mediums, and chronic criteria for mercury, silver, p,p'-DDT, dieldrin, alpha-endosulfan, beta-endosulfan, endrin, heptachlor, heptachlor epoxide, methoxychlor, toxaphene, and hexachlorobutadiene for all marine sites and both mediums.

Table 86 - Exceedances of water quality criteria for marine sites

Chemical Class	Parameter	Units	Marine Site 7 Elutriate	Water	Marine Site 11 Elutriate	Water	LDEQ Marine Acute	Chronic	EPA Marine Acute	Chronic
Metals	Mercury	µg/L	0.100	0.260	0.100	0.100	2	0.025	1.8	0.94
	Silver	µg/L	2.50	2.50	2.50	2.50			1.9	
Pesticides	DDT, p,p'-	µg/L	0.0500	0.0500	0.0500	0.0500	0.13	0.001	0.13	0.001
	Dieldrin	µg/L	0.0500	0.0500	0.0500	0.0500	0.71	0.0019	0.71	0.0019
	Endosulfan, alpha-	µg/L	0.0250	0.0250	0.0250	0.0250	0.034	0.0087	0.034	0.0087
	Endosulfan, beta-	µg/L	0.0500	0.0500	0.0500	0.0500			0.034	0.0087
	Endrin	µg/L	0.0500	0.0500	0.0500	0.0500	0.037	0.0023	0.037	0.0023
	Heptachlor	µg/L	0.0250	0.0250	0.0250	0.0250	0.053	0.0036	0.053	0.0036
	Heptachlor Epoxide	µg/L	0.0250	0.0250	0.0250	0.0250			0.053	0.0036
	Methoxychlor	µg/L	0.0500	0.0500	0.0500	0.0500				0.03
	Toxaphene	µg/L	0.850	0.850	0.850	0.850	0.21	0.0002	0.21	0.0002
Semi-Volatile Organic Compounds	Hexachlorobutadiene	µg/L	5.00	3.75	5.00	3.75	1.6	0.32		

2.7.2.1.4.5   Sediment Quality

Table 87 through Table 89 displays exceedances of NOAA sediment screening values. In most cases, values exceeding screening values are not measured values, but are instead estimates, as results were below the laboratory reporting limit.

For freshwater sites (Table 87), the measured concentrations for arsenic, copper, nickel, and zinc exceeded freshwater Lowest Effect Level (LEL) screening values at all freshwater sites, while the measured value for mercury at Site 12 exceeded the freshwater LEL screening value.

Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded sediment screening values at all freshwater sites for the following parameters- antimony, mercury, silver, aldrin, gamma-BHC, p,p'-DDD, p,p'-DDE, p,p'-DDT, dieldrin, endrin, heptachlor epoxide, toxaphene acenaphthene, acenaphthalene, anthracene, benzo(a)anthracene, benzo(a)pyrene, benzo(g,h,i)perylene, benzo(k)fluoranthene, phenanthrene, chrysene, dibenzo(a,h)anthracene, fluoranthene, fluorene, and naphthalene.

Table 87 - Exceedances of sediment screening values for freshwater sites

| Chemical Class | Parameter | Units | Freshwater | | | NOAA Sediment Screening Values for Freshwater Sediment | | | | | | | |
			Site 1	Site 2	Site 12	ARCS Hyalella TEL	TEL	TEC	LEL	PEL	PEC	SEL	UET
Metals	Antimony	µg/kg	1,950	2,800	4,850								3,000 M
	Arsenic	µg/kg	6,600	4,400	6,400	10,798	5,900	9,790	6,000	17,000	33,000	33,000	17,000 I
	Copper	µg/kg	21,100	16,300	31,700	28,012	35,700	31,600	16,000	197,000	149,000	110,000	86,000 I
	Mercury	µg/kg	55.0	75.0	255		174	180	200	486	1,060	2,000	560 M
	Nickel	µg/kg	19,600	16,000	25,600	19,514	18,000	22,700	16,000	36,000	48,600	75,000	43,000 H
	Silver	µg/kg	325	465	800					500			4,500 H
	Zinc	µg/kg	81,500	61,100	152,000	98,000	123,000	121,000	120,000	315,000	459,000	820,000	520,000 M
Pesticides	Aldrin	µg/kg	1.15	1.60	2.75				2.00			80.0	40.0 I
	BHC, gamma-	µg/kg	1.15	1.60	2.75		0.940	2.37	3.00	1.38	4.99	10.0	9.00 I
	DDD, p,p'-	µg/kg	2.25	3.10	5.35		3.54	4.88	8.00	8.51	28.0	60.0	60.0 I
	DDE, p,p'-	µg/kg	2.25	3.10	5.35		1.42	3.16	5.00	6.75	31.3	190	50.0 I
	DDT, p,p'-	µg/kg	2.25	3.10	5.35		1.19 c	4.16	8.00	4.77 c	62.9	710	50.0 I
	Dieldrin	µg/kg	2.25	3.10	5.35		2.85	1.90	2.00	6.67	61.8	910	300 I
	Endrin	µg/kg	2.25	3.10	5.35		2.67	2.22	3.00	62.4	207	1,300	500 I
	Heptachlor Epoxide	µg/kg	1.15	1.60	2.75		0.600	2.47	5.00	2.74	16.0	50.0	30.0 I
	Toxaphene	µg/kg	22.4	31.2	53.5		0.100 c						
Polycyclic Aromatic Hydrocarbons	Acenaphthene	µg/kg	224	312	535		6.71 c			88.9 c			290 M
	Acenaphthylene	µg/kg	224	312	535		5.87 c			128 c			
	Anthracene	µg/kg	224	312	535	10.0	46.9 c	57.2	220	245 c	845	3,700	260 M
	Benzo(a)anthracene	µg/kg	224	312	535	15.7	31.7	108	320	385	1,050	14,800	500 I
	Benzo(a)pyrene	µg/kg	224	312	535	32.4	31.9	150	370	782	1,450	14,400	700 I
	Benzo(g,h,i)perylene	µg/kg	224	312	535				170			3,200	300 M
	Benzo(k)fluoranthene	µg/kg	224	312	535	27.2			240			13,400	13,400 B
	Phenanthrene	µg/kg	224	312	535	18.7	41.9	204	560	515	1,170	9,500	800 I
Semi-Volatile Organic Compounds	Chrysene	µg/kg	224	312	535	26.8	57.1	166	340	862	1,290	4,600	800 I
	Dibenzo(a,h)anthracene	µg/kg	224	312	535	10.0	6.22 c	33.0	60.0	135 c		1,300	100 M
	Fluoranthene	µg/kg	224	312	535	31.5	111	423	750	2,355	2,230	10,200	1,500 M
	Fluorene	µg/kg	224	312	535	10.0	21.2 c	77.4	190	144 c	536	1,600	300 M
	Naphthalene	µg/kg	113	157	271	14.7	34.6 c	176		391 c	561		600 I

For brackish sites (Table 88), sediment screening values were exceeded for measured or estimated (j-flagged, not below the laboratory reporting limit) concentrations of aluminum (AET at all sites), antimony (T20 at sites 8, 9, and 10; T50 at sites 3, 4, and 5), arsenic (ERL at Site 9), barium (TEL at sites 3, 4, 5, 6, 9, and 10), cobalt (AET at Site 9), copper (TEL at sites 3, 4, and 5; ERL at Site 6), manganese (AET at sites 3, 4, 6, 8, 9, and 10), nickel (TEL at sites 3, 4, 8, and 10; ERL at sites 5, 6, and 9), zinc (T20 at sites 4, 5, and 6; TEL at Site 3), benzo(a)anthracene (ERL at Site 3), benzo(a)pyrene (T50 at Site 3), benzo(b)fluoranthene (T20 at sites 4 and 8; T50 at Site 3), benzo(g,h,i)perylene (T20 at Site 3), phenanthrene (PEL at Site 5), chrysene (ERL at Site 3), fluoranthene (TEL at sites 3 and 6), pyrene (ERL at Site 3; TEL at Site 6), and Indeno(1,2,3-cd)pyrene (T20 at Site 3). With the exception of the measured phenanthrene concentration for Site 5, no measured values exceeded PEL or ERM screening values.

Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded sediment screening values at all brackish sites for the following parameters- mercury, silver, gamma-BHC, p,p'-DDD, p,p'-DDE, p,p'-DDT, dieldrin, heptachlor, heptachlor epoxide, toxaphene, acenaphthene, acenaphthylene, anthracene, benzo(k)fluoranthene, benzoic acid, benzyl alcohol, bis(2-ethylhexyl) phthalate, butyl benzyl phthalate, o-cresol, p-cresol, dibenzo(a,h)anthracene, dibenzofuran, 2,4-dimethylphenol, fluorine, hexachlorobenzene, 2-methylnaphthalene, naphthalene, nitrobenzene, and n-nitrosodiphenylamine. For benzo(a)anthracene, benzo(a)pyrene, benzo(b)fluoranthene, benzo(g,h,i)perylene, phenanthrene, chrysene, fluoranthene, pyrene, and indeno(1,2,3-cd)pyrene, estimated concentrations for sites

with results below the laboratory reporting limit also exceeded sediment screening values.

Table 88 - Exceedances of sediment screening values for brackish sites

Chemical Class	Parameter	Units	Site 3	Site 4	Site 5	Site 6	Site 8	Site 9	Site 10	T20	TEL	ERL	T50	PEL	ERM	AET	Eco Tox EqP
Metals	Aluminum	µg/kg	11,400,000	12,100,000	16,200,000	12,800,000	6,580,000	12,000,000	9,020,000							0.0180 N	
	Antimony	µg/kg	2,850	3,200	5,250	2,600	1,800	1,900	1,900	630			2,400			9,300 E	
	Arsenic	µg/kg	5,500	3,300	4,500	6,800	4,000	16,200	6,300	7,400	7,240	8,200	20,000	41,600	70,000	35,000 B	
	Barium	µg/kg	523,000	180,000	212,000	375,000	106,000	359,000	322,000		130,100 #						
	Cobalt	µg/kg	8,300	5,400	8,900	9,000	6,500	21,600	6,700							10,000 N	
	Copper	µg/kg	22,400	21,400	21,600	38,700	10,300	16,600	16,100	32,000	18,700	34,000	94,000	108,000	270,000	390,000 MO	
	Manganese	µg/kg	395,000	383,000	253,000	678,200	319,000	1,100,000	369,000							260,000 N	
	Mercury	µg/kg	85.0	90.0	140	75.0	55.0	60.0	55.0	140	130	150	480	700	710	410 M	
	Nickel	µg/kg	17,600	17,100	23,000	21,900	15,900	25,000	17,200	15,000	15,900	20,900	47,000	42,800	51,600	130,000 EL	
	Silver	µg/kg	472	530	800	430	300	320	315	230	700	1,000	1,100	1,770	3,700	3,100 B	
	Zinc	µg/kg	128,000	108,000	128,000	113,000	70,300	71,400	76,600	94,000	124,000	150,000	245,000	271,000	410,000	430,000 I	
Pesticides	BHC, gamma-	µg/kg	1.80	1.95	2.95	1.60	1.15	1.25	1.20		0.320			0.990		4.80 N	3.70
	DDD, p,p'-	µg/kg	3.55	3.80	5.75	3.05	2.25	2.40	2.30		1.22	2.00		7.81	20.0	16.01	
	DDE, p,p'-	µg/kg	3.55	3.80	5.75	3.05	2.25	2.40	2.30		2.07	2.20		374	27.0	9.001	
	DDT, p,p'-	µg/kg	3.55	3.80	5.75	3.05	2.25	2.40	2.30		1.19	1.00		4.77	7.00	12.0 E	
	Dieldrin	µg/kg	3.55	3.80	5.75	3.05	2.25	2.40	2.30	0.830	0.720	0.0200	2.90	4.30	8.00	1.90 E	
	Heptachlor	µg/kg	1.80	1.95	2.95	1.60	1.15	1.25	1.20							0.300 B	
	Heptachlor Epoxide	µg/kg	1.80	1.95	2.95	1.60	1.15	1.25	1.20	0.600 c				2.74 c			
	Toxaphene	µg/kg	35.5	38.0	57.5	30.6	22.5	24.1	22.9		0.100 c						28.0
Polycyclic Aromatic Hydrocarbons	Acenaphthene	µg/kg	353	380	575	306	225	240	229	19.0	6.71	16.0	116	88.9	500	130 E	
	Acenaphthylene	µg/kg	353	380	575	306	225	240	229	14.0	5.87	44.0	128	640		71.0 E	
	Anthracene	µg/kg	353	380	575	306	225	240	229	36.0	46.9	853	290	245	1,300	280 E	
	Benzo(a)anthracene	µg/kg	288	380	575	306	225	240	229	61.0	74.8	261	466	693	1,600	960 E	
	Benzo(a)pyrene	µg/kg	637	380	575	306	225	240	229	69.0	88.8	430	520	763	1,600	1,100 E	
	Benzo(b)fluoranthene	µg/kg	1,130	182	575	306	185	240	229	130			1,107			1,800 EL	
	Benzo(g,h,i)perylene	µg/kg	292	380	575	306	225	240	229	67.0			497			670 M	
	Benzo(k)fluoranthene	µg/kg	366	380	575	306	225	240	229	70.0			537			1,800 EL	
	Phenanthrene	µg/kg	142	380	575	306	225	240	229	68.0	86.7	240	455	544	1,500	660 E	
Semi-Volatile Organic Compounds	Benzoic Acid	µg/kg	353	380	575	306	225	240	229							65.0 O	
	Benzyl Alcohol	µg/kg	353	380	575	306	225	240	229							52.0 B	
	Bis(2-Ethylhexyl) Phthalate	µg/kg	353	380	575	306	225	240	229		182			2,647		1,300 I	
	Butyl Benzyl Phthalate	µg/kg	353	380	575	306	225	240	229							63.0 M	1,100
	Chrysene	µg/kg	476	380	575	306	225	240	229	82.0	108	384	652	846	2,800	950 E	
	Cresol, o-	µg/kg	353	380	575	306	225	240	229							8.00 B	
	Cresol, p-	µg/kg	353	380	575	306	225	240	229							100 B	
	Dibenzo(a,h)anthracene	µg/kg	353	380	575	306	225	240	229	19.0	6.22	63.4	113	135	260	230 OM	
	Dibenzofuran	µg/kg	353	380	575	306	225	240	229							110 E	2,000
	Dimethylphenol, 2,4-	µg/kg	353	380	575	306	225	240	229							18.0 N	
	Fluoranthene	µg/kg	391	380	575	124	225	240	229	119	113	600	1,034	1,494	5,100	1,300 E	
	Fluorene	µg/kg	353	380	575	306	225	240	229	19.0	21.2	19.0	114	144	540	120 E	540
	Hexachlorobenzene	µg/kg	353	380	575	306	225	240	229							6.00 B	
	Methylnaphthalene, 2-	µg/kg	353	380	575	306	225	240	229	21.0	20.2	70.0	128	201	670	64.0 E	
	Naphthalene	µg/kg	179	193	292	155	114	122	116	30.0	34.6	160	217	391	2,100	230 E	480
	Nitrobenzene	µg/kg	353	380	575	306	225	240	229							21.0 N	
	N-Nitrosodiphenylamine	µg/kg	353	380	575	306	225	240	229							28.0 I	
	Pyrene	µg/kg	796	380	575	198	225	240	229	125	153	665	932	1,398	2,600	2,400 E	
	Pyrene, Indeno (1,2,3-cd)	µg/kg	345	380	575	306	225	240	229	68.0		488				600 M	

For marine sites (Table 89), sediment screening values were exceeded for measured concentrations of aluminum (T20 at Site 11; T50 at Site 7), arsenic (ERL at Site 11), barium (TEL at Site 11), cobalt (AET at Site 11), copper (TEL at both sites), manganese (AET at both sites), and nickel (ERL at both sites), and for the estimated (j-flagged, not below the laboratory reporting limit) concentration of butyl benzyl phthalate at Site 7. No measured values exceeded PEL or ERM screening values.

Results below the laboratory reporting limit, when estimated as one half of the laboratory reporting limit, exceeded sediment screening values at both marine sites for the following parameters- silver, gamma-BHC, p,p'-DDD, p,p'-DDE, p,p'-DDT, dieldrin, heptachlor, heptachlor epoxide, toxaphene, acenaphthene, acenaphthylene, anthracene, benzo(a)anthracene, benzo(a)pyrene, benzo(b)fluoranthene, benzo(g,h,i)perylene, benzo(k)fluoranthene, phenanthrene, benzoic acid, benzyl alcohol, bis(2-ethylhexyl) phthalate, butyl benzyl phthalate, o-cresol, p-cresol, dibenzo(a,h)anthracene, dibenzofuran, 2,4-dimethylphenol, fluoranthene, fluorene, hexachlorobenzene, 2-methylnaphthalene, naphthalene, nitrobenzene, n-nitrosodiphenylamine, pyrene, and indeno(1,2,3-cd)pyrene. The concentration for butyl benzyl phthalate at Site 11, when estimated as one-half of the laboratory reporting limit,

also exceeded the AET screening value.

Table 89 - Exceedances of sediment screening values for marine sites

Chemical Class	Parameter	Units	Marine Site 7	Site 11	T20	TEL	ERL	T50	PEL	ERM	AET	Eco Tox EqP
Metals	Aluminum	µg/kg	8,770,000	14,100,000							0.0180 N	
	Antimony	µg/kg	4,100	2,200	630			2,400			9,300 E	
	Arsenic	µg/kg	4,500	10,300	7,400	7,240	8,200	20,000	41,600	70,000	35,000 B	
	Barium	µg/kg	95,500	201,000		130,100 #						
	Cobalt	µg/kg	7,200	10,900							10,000 N	
	Copper	µg/kg	20,600	22,200	32,000	18,700	34,000	94,000	108,000	270,000	390,000 MO	
	Manganese	µg/kg	307,000	630,000							260,000 N	
	Nickel	µg/kg	35,200	26,400	15,000	15,900	20,900	47,000	42,800	51,600	110,000 EL	
	Silver	µg/kg	700	365	230	730	1,000	1,100	1,770	3,700	3,100 B	
Pesticides	BHC, gamma-	µg/kg	2.30	1.25		0.320			0.990		4.80 N	3.70
	DDD, p,p'-	µg/kg	4.50	2.40		1.22	2.00		7.81	20.0	16.0 I	
	DDE, p,p'-	µg/kg	4.50	2.40		2.07	2.20		374	27.0	9.00 I	
	DDT, p,p'-	µg/kg	4.50	2.40		1.19	1.00		4.77	7.00	12.0 E	
	Dieldrin	µg/kg	4.50	2.40	0.830	0.720	0.0200	2.90	4.30	8.00	1.90 E	
	Heptachlor	µg/kg	2.30	1.25							0.300 B	
	Heptachlor Epoxide	µg/kg	2.30	1.25	0.600 c				2.74 c			
	Toxaphene	µg/kg	45.1	24.1		0.100 c						28.0
Polycyclic Aromatic Hydrocarbons	Acenaphthene	µg/kg	451	241	19.0	6.71	16.0	116	88.9	500	130 E	
	Acenaphthylene	µg/kg	451	241	14.0	5.87	44.0	140	128	640	71.0 E	
	Anthracene	µg/kg	451	241	34.0	46.9	85.3	290	245	1,100	280 E	
	Benzo(a)anthracene	µg/kg	451	241	61.0	74.8	261	466	693	1,600	960 E	
	Benzo(a)pyrene	µg/kg	451	241	69.0	88.8	430	520	763	1,600	1,100 E	
	Benzo(b)fluoranthene	µg/kg	451	241	130			1,107			1,800 EI	
	Benzo(g,h,i)perylene	µg/kg	451	241	67.0			497			670 M	
	Benzo(k)fluoranthene	µg/kg	451	241	70.0			537			1,800 EI	
	Phenanthrene	µg/kg	451	241	68.0	86.7	240	455	544	1,500	660 E	
Semi-Volatile Organic Compounds	Benzoic Acid	µg/kg	451	241							65.0 O	
	Benzyl Alcohol	µg/kg	451	241							52.0 B	
	Bis(2-Ethylhexyl) Phthalate	µg/kg	382	144		182			2,647		1,300 I	
	Butyl Benzyl Phthalate	µg/kg	551	241							63.0 M	1,100
	Chrysene	µg/kg	451	241	82.0	108	384	650	846	2,800	950 E	
	Cresol, o-	µg/kg	451	241							8.00 B	
	Cresol, p-	µg/kg	451	241							100 B	
	Dibenzo(a,h)anthracene	µg/kg	451	241	19.0	6.22	63.4	113	135	260	230 OM	
	Dibenzofuran	µg/kg	451	241							110 E	2,000
	Dimethylphenol, 2,4-	µg/kg	451	241							18.0 N	
	Fluoranthene	µg/kg	451	241	119	113	600	1,034	1,494	5,100	1,300 E	
	Fluorene	µg/kg	451	241	19.0	21.2	19.0	114	144	540	120 E	540
	Hexachlorobenzene	µg/kg	451	241							6.00 B	
	Methylnaphthalene, 2-	µg/kg	451	241	21.0	20.2	70.0	128	201	670	64.0 E	
	Naphthalene	µg/kg	228	122	30.0	34.6	160	217	391	2,100	230 E	480
	Nitrobenzene	µg/kg	451	241							21.0 N	
	N-Nitrosodiphenylamine	µg/kg	451	241							28.0 I	
	Pyrene	µg/kg	451	241	125	153	665	932	1,398	2,600	2,400 E	
	Pyrene, Indeno (1,2,3-cd)	µg/kg	451	241	68.0			488			600 M	

2.7.2.2   Environmental Consequences

2.7.2.2.1 Future without Project Conditions

Without the proposed MTG hurricane risk reduction project, the study area would still be affected by natural and man induced activities that would have beneficial and detrimental impacts to water quality. Some of these activities include: other Federal, state, local, and private restoration efforts such as CWPPRA, USACE ecosystem restoration projects, various NRCS programs (e.g., Coastal Wetlands Restoration Program), and LDNR projects; state and local water quality management programs; national level programs to address hypoxia in the northern Gulf of Mexico; the continued

erosion/subsidence of the coast; oil and gas development; industrial, commercial, and residential development; and Federal, State, and municipal navigation and flood damage reduction projects.

2.7.2.2.2 Future with Project Conditions

2.7.2.2.2.1   Direct Impacts

As the proposed project entails construction of approximately 93 miles of levee, the footprint for 78 miles of which includes existing hydraulic features within the study area, and the remaining 15 miles of which includes wetlands and open water, it would have significant direct impacts for area within the proposed footprint which currently consists of wetlands and open water. These areas would be converted into upland habitat, and would no longer provide for water quality. As coastal wetlands are known to benefit water quality, for example, as a source or sink for constituents, these benefits would no longer exist within the proposed levee footprint.

In addition, direct impacts as a result of construction activities are anticipated. The excavation and placement of borrow material for levee fill, as well as dredging and dredged material placement activities associated with flotation access channel construction, would result in localized increases in turbidity and suspended solids, at both the dredging and placement sites. Sediment chemistry for sample sites representative of adjacent borrow indicate the presence of low level contamination in some sediments proposed for use as levee fill. Because the method of excavation and placement (mechanical dredging) minimizes water column impacts from placement activities, and includes dewatering, it is not anticipated that the use of adjacent borrow for levee fill would have significant impacts on the receiving aquatic environment. In addition, because adjacent borrow material is expected to have characteristics similar to sediments present at the proposed placement sites, no significant changes in sediment quality at the placement sites are anticipated.

Construction of structures (i.e., floodgates, tidal exchange structures, and the locks) would result in localized increases in turbidity associated with runoff of construction materials. To minimize construction related impacts, it is anticipated that a Stormwater Pollution Prevention Plan (SWPPP) shall be implemented for construction activities. SWPPPs shall be prepared in accordance with good engineering practices emphasizing storm water Best Management Practices and complying with Best Available Technology Economically Achievable and Best Conventional Pollutant Control Technology. The SWPPP shall identify potential sources of pollution, which may reasonably be expected to affect storm water discharges associated with the construction activity. In addition, the SWPPP shall describe and ensure the implementation of practices which are to be

used to reduce pollutants in storm water discharges associated with the construction activity and to assure compliance with the terms and conditions of this permit (USEPA 2012).

2.7.2.2.2.2   Indirect Impacts

The proposed hurricane risk reduction project could have significant indirect impacts on study area water quality, the extent to which is largely unknown. Based on historical water quality information for the study area, it is clear that a majority of the water quality problems within the study area occur on the protected side of the proposed levee alignment. Although McAlpin et. al (2010) suggests that proper management of tidal exchange structures can minimize changes in flow and water level between the flood and protected side of the proposed levee alignment , it is a legitimate concern that the proposed alignment will cause significant alteration of hydrology and hydraulics in the study area, such that water exchange between the protected and flood sides of the proposed levee alignment is significantly inhibited, and that localized areas of stagnation behind the levee alignment may occur. If these conditions present themselves, the levee alignment would serve as a barrier between relatively free of contamination Gulf of Mexico waters and impaired waters, further exacerbating water quality conditions on the protected side of the alignment. Moreover, the potential expansion of developed areas as a result of the project could lead to additional point and nonpoint discharges within the hurricane risk reduction system, which would further degrade water quality on the protected side of the propose alignment. Also, as sea level rise increases water levels in the study area, the frequency with which tidal structures are closed would be expected to increase, causing further stagnation for waters on the protected side of the proposed levee alignment.

The proposed project could also prevent the introduction of mineral sediments from the flood side to the protected side. Mineral sediments are known to stimulate the growth of marsh vegetation, and input of mineral sediments associated with tropical activity can raise ground elevations, helping marshes to keep pace vertically with sea level rise. A lack of sediment input to the protected side of the proposed levee system could lead to the conversion of marsh substrate to predominantly organic substrate, creating a situation similar to that which occurs in areas subject to river water influx without mineral sediment input, such as portions of the Penchant Basin which receives Atchafalaya River water input and the marsh area beyond Big Mar which receives river water input via the Caernarvon Freshwater Diversion (Swarzenski et. al 2008). This could make marshes more vulnerable to erosional forces, leading to a further reduction in water quality on the protected side of the proposed levee alignment.

A major potential benefit of the project is that it would provide for the protection of marshes on the flood side of the proposed levee alignment, potentially extending the

lifespan of these marshes. However, the marshes just outside of the hurricane risk reduction system are expected to be subjected to an increase in wave energy as a result of the proposed project, which could lead to the accelerated loss of unprotected marsh vegetation. This detracts from rationale for utilizing the topmost organic sediment layer of adjacent levee borrow areas for marsh construction on the flood side of the proposed levee alignment. All of these impacts to wetlands habitat would affect water quality, for reasons stated in section 2.7.1.1.3.

2.7.2.2.2.3 Cumulative Impacts

The proposed project, combined with other coastal activities (such as those included in the discussion of future without project conditions) would cumulatively impact study area water quality. In addition, it is foreseeable that the proposed project may impact the attainment of state water quality standards in the study area, leading to changes in regulation of point and nonpoint source discharges within the area, particularly on the protected side of the proposed hurricane risk reduction alignment.

2.8 Other Studies – Eastern Tie-In Levee and C-North Levee Section

2.8.1 Background

Initially, it was proposed for levee Reach L of the authorized Morganza to the Gulf alignment to terminate at the connection to an existing levee of an adjacent study- the Larose to Golden Meadow Project. However, our most recent findings from the current MTG surge analysis results, which were completed after the 2002 report, indicates the existing levees in the LGM system will be flanked by surge, if the LGM levees are not raised to the same level of protection that will be provided by the construction of the MTG levees. Similar to MTG, the LGM project is currently completing a post authorization change report which is scheduled to be completed after the MTG report. At this time, the future levee elevations have not been finalized. The LGM project is not authorized to be raised to the same level of protection of the MTG study.
In order to resolve this issue, refinements have been made to the MTG alignment to prevent overtopping of the LGM levee and to also provide additional protection from storm surge from the Barataria Basin. Reach L of the MTG study has been extended northwest along the existing LGM study alignment (referred to as "Larose C-North Reach 1") and further north to Lockport (referred to as the "Lockport to Larose Reach"). The extended alignment is shown on Figure 96. The Lockport to Larose Reach follows an alignment formerly proposed under the Donaldsonville to the Gulf Feasibility Study which was determined to be economically feasible. The Lockport reach will protect the MTG study area from surges from the Barataria Basin but could be overtopped in future due to relative sea level rise. The following section details the design for the two levee reaches required to provide a continuous enclosed barrier to protect the MTG study

area from hurricane surge.

Figure 96 – Morganza to the Gulf Levee Reaches

2.8.2 Methodology

The Larose C-North reach was subdivided into two hydraulic reaches. The western portion of the reach remained Larose C-North and the northern portion of the reach was renamed the GIWW reach. The Larose to Lockport reach was subdivided in two hydraulic reaches as well. The new reaches were renamed to Lockport to Larose (a) and Lockport to Larose (b). The new reach designations are shown in Figure 97.
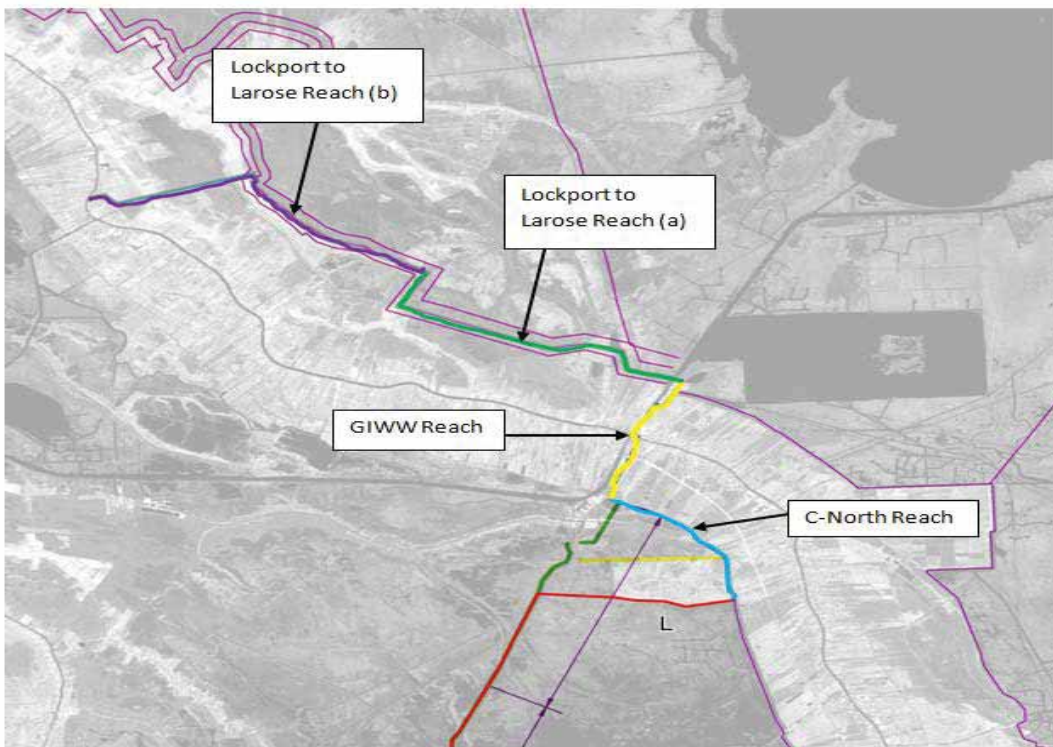
Figure 97– Hydraulic Reaches

Using the 2010 Morganza to the Gulf ADCIRC base hydraulic boundary conditions (for the Larose C-North reach) and the 2010 West Shore Lake Pontchartrain ADCIRC base hydraulic boundary conditions (for the GIWW, Lockport to Larose (a), and Lockport to Larose (b) reaches) along with the intermediate sea level rise rate of 0.03 ft/yr (eustatic + subsidence) previously used for the other Morganza to the Gulf levee reaches, the 2035 and 2085 hydraulic boundary conditions were then determined.

First, the 2010 base hydraulic boundary conditions results from the models were found.

Table 90 - ADCIRC Base Still Water Levels from model results

**2010 ADCIRC Base Model Results
Mean Still Water Levels At Varying Return Periods**

Levee Reach	2010 ADCIRC Point ID	10% (10 Year) Mean Still Water Level (ft)	Std. Dev.	2% (50 Year) Mean Still Water Level (ft)	Std. Dev.	1% (100 Year) Mean Still Water Level (ft)	Std. Dev.	0.2% (500 Year) Mean Still Water Level (ft)	Std. Dev.
GIWW	3	4.2	0.1	7	0.35	8.4	0.5	10.9	0.9
C-North	171	6.4	0.4	9.5	1.2	13.2	1.5	18	2.3
Lockport to Larose (a)	1	4.3	0.1	6.6	0.35	7.9	0.5	9.8	0.9
Lockport to Larose (b)	2	4	0.1	5.7	0.35	6.7	0.5	8.2	0.9

Table 91 – ADCIRC Base Wave Heights from model results

**2010 ADCIRC Base Model Results
Wave Heights At Varying Return Periods**

Levee Reach	2010 ADCIRC Point ID	10% (10 Year) Wave Height (ft)	Std. Dev.	2% (50 Year) Wave Height (ft)	Std. Dev.	1% (100 Year) Wave Height (ft)	Std. Dev.	0.2% (500 Year) Wave Height (ft)	Std. Dev.
GIWW	3	0.7	0.07	1	0.10	1.5	0.15	2.3	0.23
C-North	171	1	0.10	1.2	0.12	2.3	0.23	4.1	0.41
Lockport to Larose (a)	1	2.1	0.21	3	0.30	4	0.40	5.4	0.54
Lockport to Larose (b)	2	1.3	0.13	1.8	0.18	2.5	0.25	3.7	0.37

Table 92 – ADCIRC Base Wave Periods from model results

Levee Reach	2010 ADCIRC Point ID	2010 ADCIRC Base Model Results Wave Periods At Varying Return Periods									
		10% (10 Year) Wave Period (s)	Std. Dev.	2% (50 Year) Wave Period (s)	Std. Dev.	1% (100 Year) Wave Period (s)	Std. Dev.	0.2% (500 Year) Wave Period(s)	Std. Dev.
GIWW	3	0.9	0.18	1.3	0.26	3	0.60	4.8	0.96
C-North	171	5.1	1.02	5.2	1.04	5.5	1.10	5.8	1.16
Lockport to Larose (a)	1	2.9	0.58	4.1	0.82	4.8	0.96	5.6	1.12
Lockport to Larose (b)	2	2.9	0.58	4.2	0.84	5	1.00	6.1	1.22

The base hydraulic boundary conditions were then plotted to find the base hydraulic boundary condition for the 35 year return period as shown in Figure 98,Figure 99 and Figure 100 with return periods in years on the x-axis.
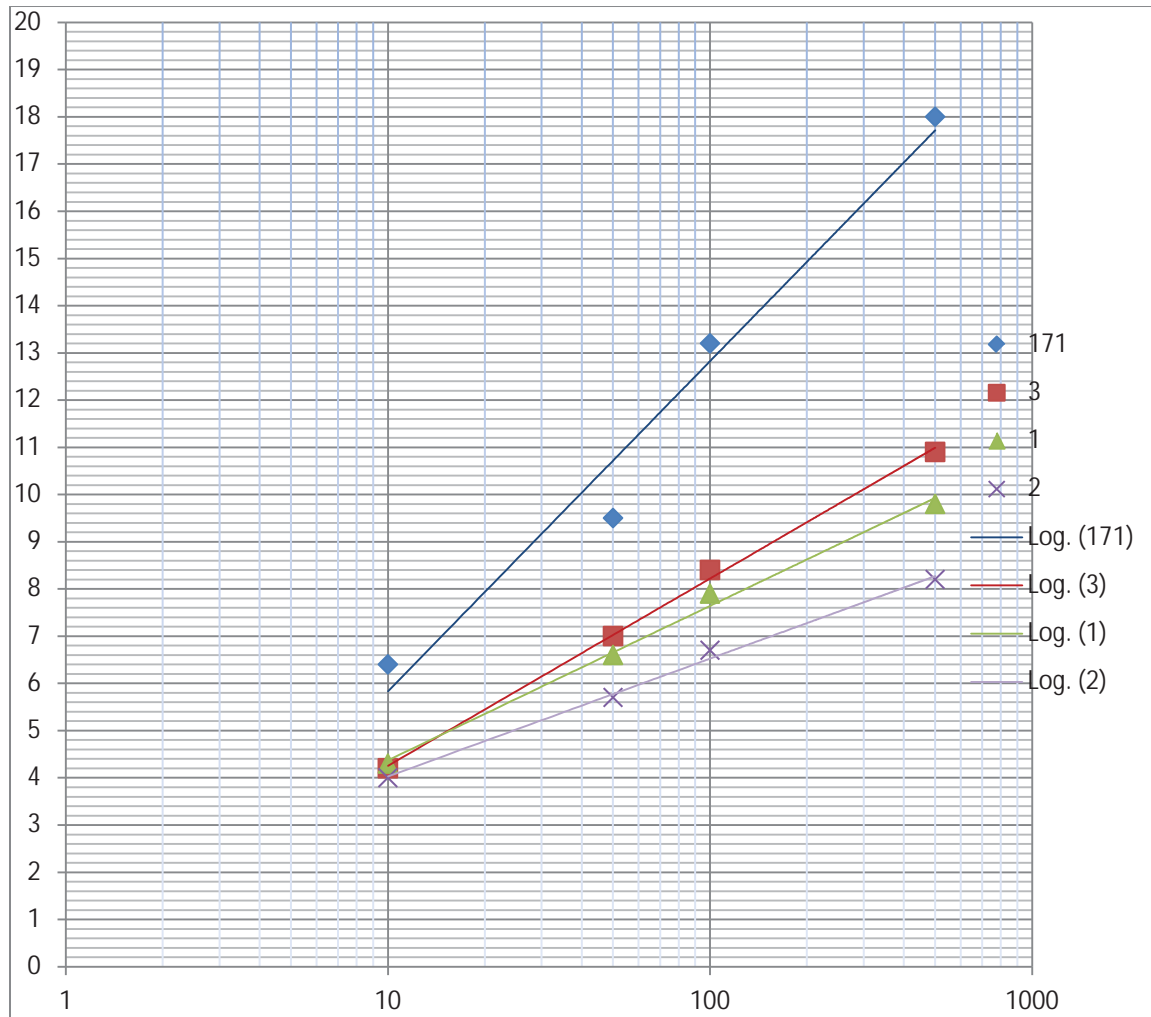
Figure 98 – Return Period (years) vs. Still Water Elevation (ft)
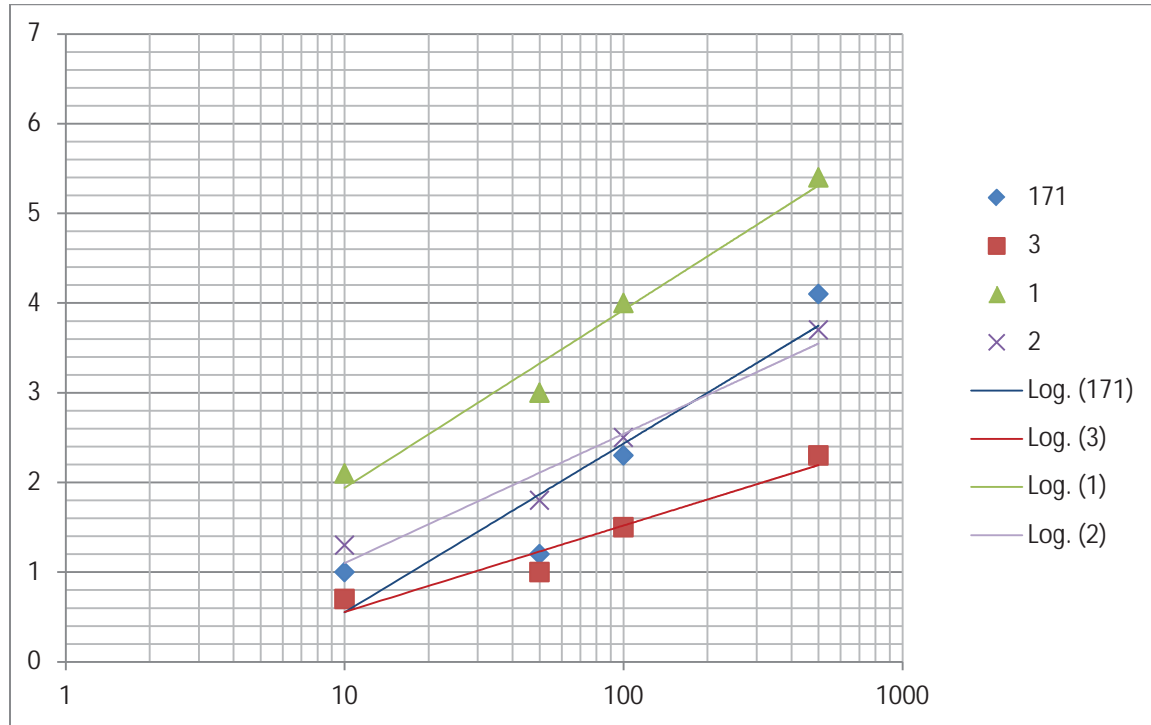
Figure 99 – Return Period (years) vs. Wave Height (ft)
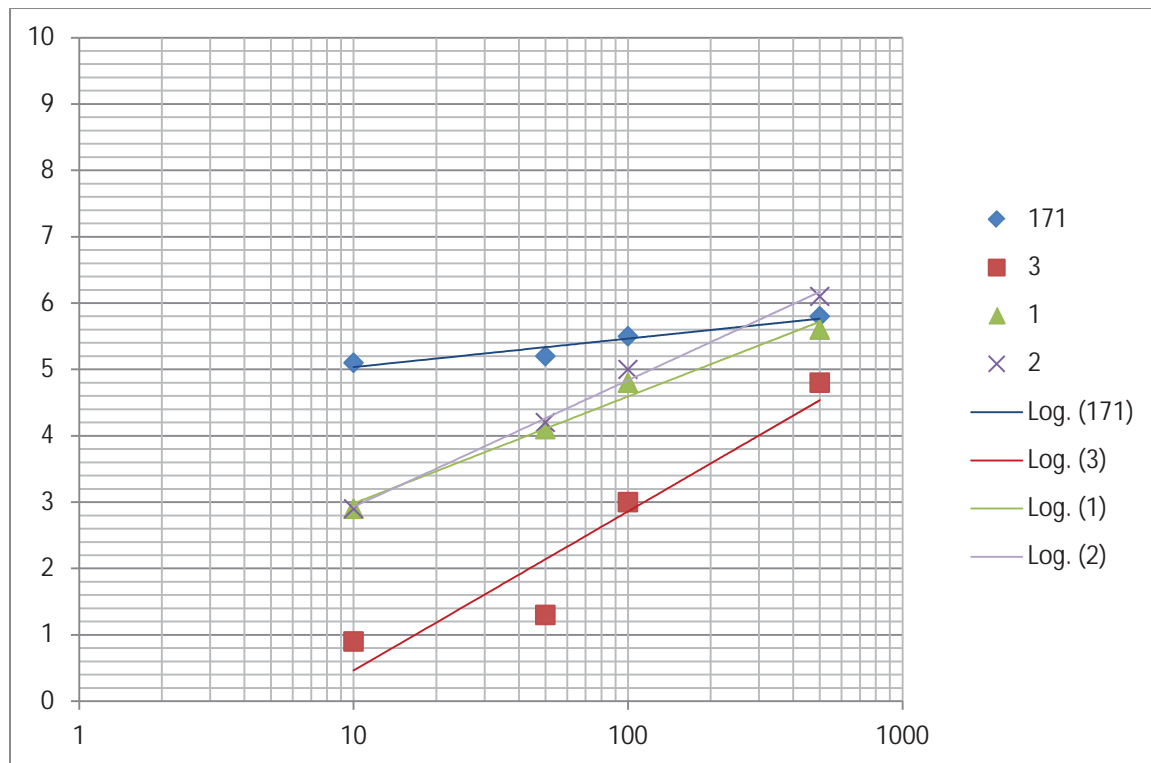
Figure 100 – Return Period (years) vs. Wave Period (s)

Table 93 – Computed ADCIRC Base 35 Year Hydraulic Boundary Conditions

2010 ADCIRC Base Model Results Computed From Graphs Hydraulic Boundary Conditions At 3% (35 Year) Return Period				
Levee Reach	2010 ADCIRC Point ID	Mean Still Water Level (ft)	Wave Height (ft)	Wave Period (s)
GIWW	3	6.4	1.1	1.8
C-North	171	9.6	1.6	5.3
Lockport to Larose (a)	1	6.2	3	3.9
Lockport to Larose (b)	2	5.4	1.9	4

Sea level rise was then applied and the hydraulic boundary conditions were converted to the appropriate years (2035 and 2085) as illustrated below.

Table 94 – ADCIRC 2035 Converted Still Water Levels (ft)

2035 ADCIRC Converted Model Results Mean Still Water Levels At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Still Water Level (ft)	1% (100 Year) Still Water Level (ft)
GIWW	3	7.2	9.2
C-North	171	10.4	14.0
Lockport to Larose (a)	1	7.0	8.7
Lockport to Larose (b)	2	6.2	7.5

Table 95 – ADCIRC 2035 Converted Wave Heights (ft)

2035 ADCIRC Converted Model Results Wave Heights At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Wave Height (ft)	1% (100 Year) Wave Height (ft)
GIWW	3	1.5	1.9
C-North	171	2.0	2.7
Lockport to Larose (a)	1	3.4	4.4
Lockport to Larose (b)	2	2.3	2.9

Table 96 – ADCIRC 2035 Converted Wave Periods (s)

2035 ADCIRC Converted Model Results Wave Periods At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Wave Periods (s)	1% (100 Year) Wave Periods (s)
GIWW	3	2.1	3.4
C-North	171	5.9	5.9
Lockport to Larose (a)	1	4.1	5.0
Lockport to Larose (b)	2	4.4	5.4

Table 97– ADCIRC 2085 Converted Still Water Levels (ft)

2085 ADCIRC Converted Model Results Mean Still Water Levels At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Still Water Level (ft)	1% (100 Year) Still Water Level (ft)
GIWW	3	8.7	10.7
C-North	171	11.9	15.5
Lockport to Larose (a)	1	8.5	10.2
Lockport to Larose (b)	2	7.7	9.0

Table 98 – ADCIRC 2085 Converted Wave Heights (ft)

2085 ADCIRC Converted Model Results Wave Heights At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Wave Height (ft)	1% (100 Year) Wave Height (ft)
GIWW	3	2.2	2.6
C-North	171	2.7	3.4
Lockport to Larose (a)	1	4.1	5.1
Lockport to Larose (b)	2	3.0	3.6

Table 99 – ADCIRC 2085 Converted Wave Periods (s)

2085 ADCIRC Converted Model Results Wave Periods At Varying Return Periods			
Levee Reach	2010 ADCIRC Point ID	3% (35 Year) Wave Periods (s)	1% (100 Year) Wave Periods (s)
GIWW	3	2.6	4.0
C-North	171	6.9	6.7
Lockport to Larose (a)	1	4.6	5.4
Lockport to Larose (b)	2	5.0	6.0

These hydraulic boundary conditions were then used to determine the levee and structure elevations. For the surge elevation, wave height, and wave period determined for the authorized level of risk reduction, the maximum allowable average wave overtopping of 0.1 cfs/ft at 90% level of assurance and 0.01 cfs/ft at 50% level of assurance for grass-covered levees. For the surge elevation, wave height, and wave period determined for the post authorization level of risk reduction, the maximum allowable average wave overtopping of 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at 50% level of assurance for wall type structures with appropriate protection on the back side. A Monte Carlo analysis was then applied. In the Monte Carlo analysis the overtopping algorithm is repeated to compute the overtopping rate many times (in this case 10,000 iterations).To determine the overtopping rate in the Monte Carlo analysis, the probabilistic overtopping formulations from Van der Meer are applied for levees (see text box below) and the Franco & Franco formulation for floodwalls. Besides the geometric parameters (levee height and slope), hydraulic input parameters for determination of the overtopping rate in Equations 1 and 2 are the water elevation (ζ), the significant wave height (H_s) and the peak wave period (T_p).

Van der Meer overtopping formulations

The overtopping formulation from Van der Meer reads (TAW, 2002)-

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_0\exp\left(-4.75\frac{R_c}{H_{m0}}\frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

$$with \quad \max imum: \quad \frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right) \tag{1}$$

With-

q - average overtopping rate [cfs/ft]

g - gravitational acceleration [ft/s^2]

H_{m0} - wave height at toe of the structure [ft]

ξ_0- surf similarity parameter [-]

α - slope [-]

R_c - freeboard [ft]

γ - coefficient for presence of berm (b), friction (f), wave incidence (β), vertical wall (v)

The surf similarity parameter ξ0 is defined herein as $\xi_0 = \tan\alpha / \sqrt{s_0}$ with α the angle of slope and s_0 the wave steepness. The wave steepness follows from $s0 = 2\pi H_{m0}/(g\, Tm_{-10}^2)$. The coefficients -4.75 and -2.6 in Equation 1 are the mean values. The standard deviations of these coefficients are equal to 0.5 and 0.35, respectively and these errors are normally distributed (TAW, 2002). The reader is referred to TAW (2002) for definitions of the various coefficients for presence of berm, friction, wave incidence, vertical wall.

Equation 1 is valid for $\xi_0 < 5$ and slopes steeper than 1-8. For values of $\xi_0 > 7$ the following equation is proposed for the overtopping rate-

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.92}\exp\left(-\frac{R_c}{\gamma_f\gamma_\beta H_{m0}(0.33+0.022\xi_0)}\right) \tag{2}$$

The overtopping rates for the range $5 < \xi_0 < 7$ are obtained by linear interpolation of Equation 1 and 2 using the logarithmic value of the overtopping rates. For slopes between 1-8 and 1-15, the solution should be found by iteration. If the slope is less than 1-15, it should be considered as a berm or a foreshore depending on the length of the section compared to the deep water wavelength. The coefficients -0.92 is the mean value. The standard deviation of this coefficient is equal to 0.24 and the error is normally distributed (TAW, 2002).

Figure 101 graphically shows the overtopping for a levee and floodwall situation including the most relevant parameters.

In the design process, we use the best estimate 1% values for these parameters from the JPM-OS method (Resio, 2007); uncertainty in these values exists. Resio (2007) has provided a method to derive the standard deviation in the 1% surge elevation. Standard deviation values of 10% of the average significant wave height and 20% of the peak period were used (Smith, 2006, pers. comm.). In absence of data, all uncertainties are assumed to be normally distributed. If additional data would show another distribution, that distribution has to be included in the methodology.
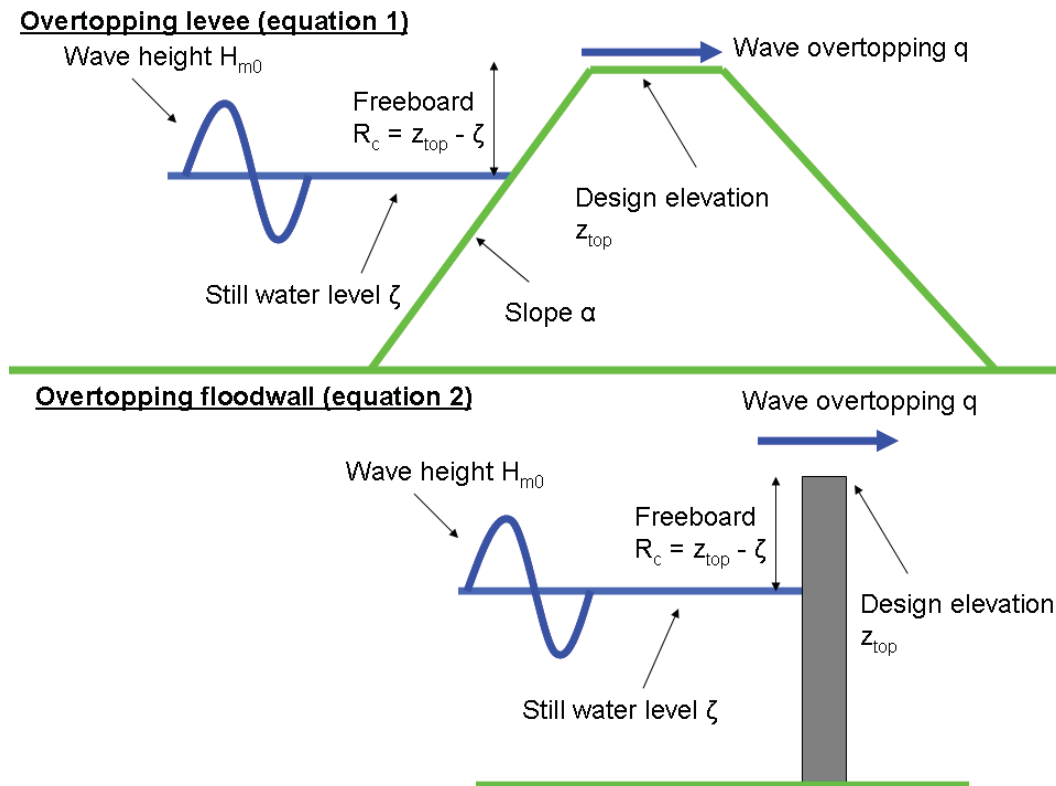
Figure 101 – Definitions for Overtopping for Levee and Floodwall

The Monte Carlo Analysis is executed as follows:

1. Draw a random number between 0 and 1 to set the exceedence probability (p).

2. Compute the water elevation from a normal distribution using the mean 1% surge elevation and standard deviation as parameters and with an exceedence probability (p).

3. Draw a random number between 0 and 1 to set the exceedence probability (p).

4. Compute the wave height and wave period from a normal distribution using the mean 1% wave height/wave period and the associated standard deviation and with an exceedence probability (p).

5. Repeat step 3 and 4 for the three overtopping coefficients independently.

6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in step 2, 4 and 5 using the Van der Meer overtopping formulations for levees or the Franco & Franco equation for floodwalls (see Equations 1 and 2 in the textbox).

7. Repeat the Step 1 through 5 a large number of times. (N)

8. Compute the 50% and 90% confidence limit of the overtopping rate. (i.e. q_{50} and q_{90})

The procedure is implemented in the numerical software package MATLAB because it is a computationally intensive procedure. MATLAB is a high-level technical computing language and interactive environment for algorithm development, data visualization, data analysis, and numeric computation.

The computation of the overtopping rate in the present MATLAB routine is limited in the sense that it can only take into account an average slope for the entire cross-section. If a wave berm exists, this effect is included in a berm factor. The following procedure was carried out to determine this berm factor. First, the overtopping rate is computed with PC-Overslag with the best estimates of surge level and waves. Next, the berm factor is calibrated with the Van der Meer overtopping formulations to get exactly same result from PC-Overslag. Then, the berm factor is checked to see if it is in between the recommended range of 0.6 – 1.0. Finally, the calibrated berm factor is applied in the uncertainty analysis (and keep this factor constant) throughout the Monte Carlo analysis in MATLAB.

The 1% and 3% levee and structure elevations for the years 2035 and 2085 were then computed for the "with" and "without" wave berm (GIWW and Larose C-North reaches only) scenarios.

2.8.3 Results

Levee and Wave Berm Design

Levee Elevation
In ft NAVD88 2004.65

Top of Berm Elevation
In ft NAVD88 2004.65

Upper
Slope 1:6

Toe of Berm Elevation
In ft NAVD88 2004.65

Berm slope 1:15

Lower Slope 1:6

Levee toe
In ft NAVD88 2004.65
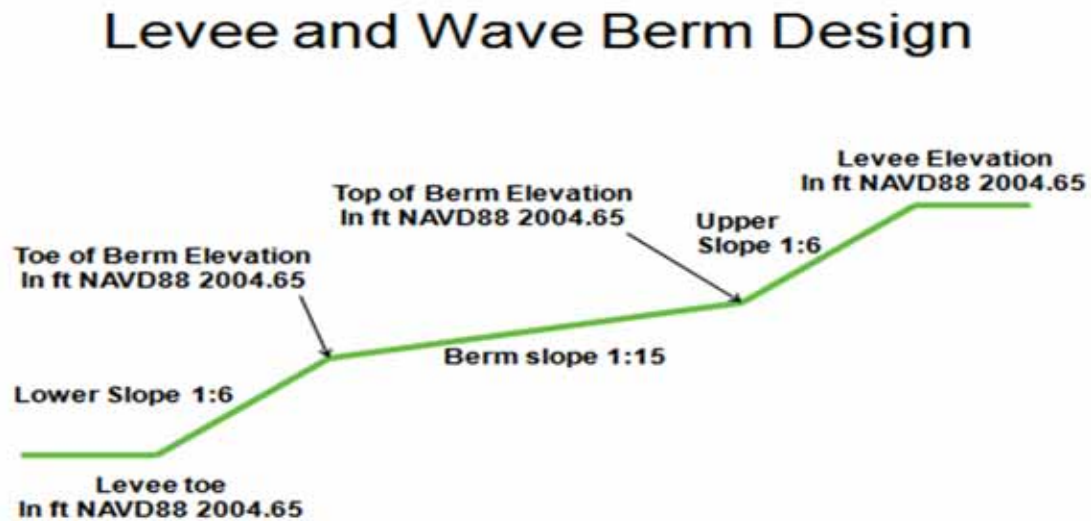
Figure 102 – Levee and Wave Berm Design

Table 100 – 2035 and 2085 3% Elevations with wave berms

2035 3% Elevations With Wave Berms									
Section	SWL	SWL	Hs	Tp	Levee Elev	Upper Slope	Top of Berm El	Berm Slope	Toe of Berm El	Lower Slope
	ft	Std. Dev.	ft	sec	ft		ft		ft	
C-North	10.4	1	2	5.9	13.5	1 on 6	10.4	1 on 15	8.4	1 on 6
GIWW	7.2	0.25	1.5	2.1	8.0	1 on 6	7.2	1 on 15	5.7	1 on 6
Lockport to Larose (a)	7	0.25	3.4	4.1	9.5	1 on 6	7.0	1 on 15	3.6	1 on 6
Lockport to Larose (b)	6.2	0.25	2.3	4.4	8.5	1 on 6	6.2	1 on 15	3.9	1 on 6

2085 3% Elevations With Wave Berms									
Section	SWL	SWL	Hs	Tp	Levee Elev	Upper Slope	Top of Berm El	Berm Slope	Toe of Berm El	Lower Slope
	ft	Std. Dev.	ft	sec	ft		ft		ft	
C-North	11.9	1	2.7	6.9	15.5	1 on 6	11.9	1 on 15	9.2	1 on 6
GIWW	8.7	0.25	2.2	2.6	9.5	1 on 6	8.7	1 on 15	6.5	1 on 6
Lockport to Larose (a)	8.5	0.25	4.1	4.6	12.0	1 on 6	8.5	1 on 15	4.4	1 on 6
Lockport to Larose (b)	7.7	0.25	3	5	11.0	1 on 6	7.7	1 on 15	4.7	1 on 6

Table 101 – 2035 and 2085 1% Elevations with wave berms

2035 1% Elevations With Wave Berms									
Section	SWL	SWL	Hs	Tp	Levee Elev	Upper Slope	Top of Berm El	Berm Slope	Toe of Berm El	Lower Slope
	ft	Std. Dev.	ft	sec	ft		ft		ft	
C-North	14	1.5	2.7	5.9	18.0	1 on 6	14.0	1 on 15	11.3	1 on 6
GIWW	9.2	0.5	1.9	3.4	11.0	1 on 6	9.2	1 on 15	7.3	1 on 6
Lockport to Larose (a)	8.7	0.5	4.4	5	12.5	1 on 6	8.7	1 on 15	4.3	1 on 6
Lockport to Larose (b)	7.5	0.5	2.9	5.4	10.5	1 on 6	7.5	1 on 15	4.6	1 on 6

2085 1% Elevations With Wave Berms									
Section	SWL	SWL	Hs	Tp	Levee Elev	Upper Slope	Top of Berm El	Berm Slope	Toe of Berm El	Lower Slope
	ft	Std. Dev.	ft	sec	ft		ft		ft	
C-North	15.5	1.5	3.4	6.7	20.5	1 on 6	15.5	1 on 15	12.1	1 on 6
GIWW	10.7	0.5	2.6	4	13.5	1 on 6	10.7	1 on 15	8.1	1 on 6
Lockport to Larose (a)	10.2	0.5	5.1	5.4	15.0	1 on 6	10.2	1 on 15	5.1	1 on 6
Lockport to Larose (b)	9	0.5	3.6	6	13.0	1 on 6	9.0	1 on 15	5.4	1 on 6

Table 102 – 2035 and 2085 3% Elevations without wave berms

2035 3% Elevations Without Wave Berms				
Section	SWL	SWL	Hs	Tp	Levee Elev
	ft	Std. Dev.	ft	sec	ft
C-North	10.4	1	2	5.9	15.0
GIWW	7.2	0.25	1.5	2.1	9.0

2085 3% Elevations Without Wave Berms				
Section	SWL	SWL	Hs	Tp	Levee Elev
	ft	Std. Dev.	ft	sec	ft
C-North	11.9	1	2.7	6.9	18.5
GIWW	8.7	0.25	2.2	2.6	11.0

1V to 4H levee slope

Table 103 – 2035 and 2085 1% Elevations without wave berms

2035 1% Elevations Without Wave Berms					
Section	SWL	SWL	Hs	Tp	Levee Elev
	ft	Std. Dev.	ft	sec	ft
C-North	14	1.5	2.7	5.9	20.5
GIWW	9.2	0.5	1.9	3.4	12.5
2085 1% Elevationsout Without Wave Berms					
Section	SWL	SWL	Hs	Tp	Levee Elev
	ft	Std. Dev.	ft	sec	ft
C-North	15.5	1.5	3.4	6.7	24.0
GIWW	10.7	0.5	2.6	4	15.0

1V to 4H levee slope

Table 104 – 2085 3% and 1% Structure Elevations

2085 3% Structure Elevations			
Section	90% SWL	Wave Height	Struct. Elev.
	ft	ft	ft
C-North	13.2	2.7	16.0/18.5*
GIWW	9	2.2	12.0
Lockport to Larose (a)	8.8	4.1	13.0
Lockport to Larose (b)	8	3	11.5

2085 1% Structure Elevations			
Section	90% SWL	Wave Height	Struct. Elev.
	ft	ft	ft
C-North	17.4	3.4	21.0/24.0*
GIWW	11.3	2.6	15.0
Lockport to Larose (a)	10.8	5.1	16.0
Lockport to Larose (b)	9.6	3.6	13.5

*Structure elevations should not be lower than the 2085 levee elevations for the same reach, per HSDRRS guidelines. If the without wave berm scenario is chosen, then the structure elevations should be raised to match the 2085 levee elevations for the corresponding without wave berm scenario.

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3   GEOTECHNICAL

3.1   Geology

3.1.1   Geology/Geomorphology

The project area is located within the south-central portion of the Mississippi River deltaic plain. Dominant physiographic features in the project area include abandoned courses of the Mississippi River (Bayous Black and Lafourche) and their associated natural levees, numerous abandoned distributary channels and associated natural levees, the GIWW, the HNC, swamp, marsh, lakes, ponds, and bays. Elevations are highest on the crests of the natural levees bordering Bayous Black and Lafourche and the numerous abandoned distributaries.

The project area has received sediments from two Mississippi River deltas within approximately the last 4,500 years. The Teche-Mississippi River Delta was actively building land in the western and central portions of the project area from approximately 4,500 to 3,500 years ago. Bayou Black, the trunk stream of the Teche-Mississippi, and major distributaries such as Bayous Penchant, Cocodrie, Piquant, Little Horn, and Carencro all trend southeast indicative of the direction of delta growth. The branching pattern of distributaries indicates that the Teche Delta prograded into shallow open water in this portion of the project area. Approximately 3,500 years ago, the Teche-Mississippi shifted eastward and started building the St. Bernard Delta. Subsidence and erosion of the Teche Delta became the dominant processes acting in the project area. Much of the original land area was submerged beneath Gulf waters or reworked by waves to become part of the transgressive shoreline which was located just north of Lake Penchant. The initial deposition from the Lafourche-Mississippi River Delta in the central and eastern portions of the project area began approximately 2000 years ago. Bayou Lafourche was the trunk stream of the Lafourche Delta. Major distributaries of the Lafourche Delta in the project area include Bayous Mauvais Bois, Dularge, Grand Caillou, Petit Caillou, Terrebonne, St. Jean Charles, and Pointe au Chenes. Like the Teche Delta, the Lafourche Delta prograded mainly into shallow open water in this portion of the project area. Approximately 1,000 years ago, the main flow of the Mississippi River shifted eastward, away from the project area. The Bayou Lafourche course was abandoned approximately 300 years ago.

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The entire project area is in the transgressive phase of the "delta cycle". During this phase, delta abandonment takes place and the processes of subsidence, erosion, and marine transgression dominate the landscape. The result is land loss, submergence, habitat change (driven mainly by elevation decrease and salinity increase), and an increase in water area.

The surface and shallow subsurface of the project area is generally composed of natural levee, swamp, and marsh deposits. Natural levee deposits are at the surface and underlie marsh and swamp deposits and occur adjacent to abandoned courses and distributaries. Natural levee deposits generally consist of soft to stiff, clays interbedded with layers and lenses of silt and silty sand. Natural levee deposits vary in thickness but generally range from 5 to 20 feet in thickness. Swamp and marsh deposits are located adjacent to natural levee deposits and comprise most of the land area in the project area. They consist mainly of very soft to medium, organic clays, with lenses of soft to medium lean clay, peat, silt, and silty sand. Swamp deposits contain wood. These deposits generally range from 5 to 20 feet thick. Interdistributary deposits underlie marsh, swamp, and natural levee deposits and consist of soft to medium clay interbedded with layers and lenses of very soft to medium lean clay, silt, and silty sand and occasional lenses of shell. Interdistributary deposits generally range from 80 to 120 feet thick. Swamp deposits are also frequently interbedded with interdistributary deposits. Intradelta deposits underlie marsh, swamp, and natural levee deposits and are interbedded with interdistributary deposits. Intradelta deposits are associated with delta progradation and are found adjacent to abandoned courses and major distributaries. Intradelta deposits consist of silt, silty sand and sand with occasional layers and lenses of soft to medium, fat and lean clays. Intradelta deposits vary in thickness but average 10 feet thick.

Cancienne-Gramercy and Commerce soils occur adjacent to the major distributaries and courses on the high to intermediate elevations of the natural levees. They are level, somewhat poorly drained soils that are loamy throughout. Schriever and Sharkey soils occur adjacent to the Cancienne-Gramercy and Commerce soils. These soils are on the low and intermediate elevations of the natural levees. They consist of level, poorly drained soils that have a loamy or clayey surface layer and a clayey subsoil. Allemands, Kenner, and Larose soils occur in the northern third of the project area. These soils are adjacent to the levees in freshwater marshes. They are level, very poorly drained soils that have a semifluid, mucky surface layer and semifluid, mucky and semifluid clayey underlying material. Lafitte and Clovelly soils occur in the middle

third of the project area. These soils are level, very poorly drained soils that have a semifluid, mucky surface layer and semifluid, clayey underlying material in brackish marshes. Timbalier and Scatlake soils occur in the southern third of the project area. These soils are level, very poorly drained soils in saline marshes.

3.1.2 Groundwater

Groundwater is at or near the surface throughout much of the project area except near the crests of natural levees and where artificially lowered within forced drainage areas. Intradelta deposits may be hydraulically connected to the adjacent water bodies.

3.2 General

In the study area beyond the narrow strips of land, natural ridges, that were formed by the bayous, generally flowing in a north to south direction, the area consists of marsh with a large percentage of open water. Sediments that comprise the natural ridges are stronger than the sediments in the lower surrounding areas, especially sediments that are in open water environments. Levee construction on top of the marsh sediments will be challenging from a stability and settlement viewpoint, especially for levee heights exceeding 15 feet. Smaller levees can initially be constructed and used to preload and strengthen the foundation and construction of subsequent lifts over time will help achieve the desired crown elevations.

Generally, the approach for levee construction consist of an initial preload lift to an elevation between +12 to +14 N.A.V.D.88 to provide a good base and working surface for construction of a geosynthetic fabric reinforced levee substantially higher than the initial preload lift. This study provides data and analyses to build a 3% AEP Level of Risk Reduction (LORR) base year (2035) and future year (2085) levee to the desired grade utilizing multiple lift construction as well as a 1% AEP LORR base and future year levee. Geotextile reinforcement, berms, and flat berm slopes were used to achieve stable levees. This report assumes a base year of 2035 when the 1% AEP project loop will provide protection to the design heights for the LORR and a base year of 2026 for the 3% AEP. Construction sequencing will be discussed later in the report. Much effort was spent on levee stability analyses and structure analysis. Settlement analyses are an important part of the study due to the added construction cost associated with settlement during and after construction.

3.3   Levee Alignments

Levees were placed as close as practical to the natural ridges along existing or former bayous to take advantage of the stronger foundation conditions and reduce the probability of failure during and after construction.

3.4   Levee Heights

The study presents two levee alternatives, namely the 3% AEP and 1% AEP LORR levees of various heights. Two time conditions were used for the investigation, the base year and future conditions. Under each condition, there are two alternative levels of protection, the 3% AEP protection levels and 1% AEP protection levels, as shown in Table 105.

Table 105 - Levee Alternatives

Base Year Condition (2035)	Future Condition (2085)
3% AEP level	3% AEP level
1% AEP level	1% AEP level

3.5   Geotechnical Exploration

Borings were obtained along the most likely alignments to provide design data for stability analyses, settlement calculations and time rate of settlement. Towards the end of this study, there were adjusted alignments for Reach G and the Barrier Alignment. Reach G has enough boring data scattered in the vicinity to provide a good basis for expected soil conditions along the newly selected alignment. The Barrier Alignment was bored for North Terrebonne Parish Drainage and Conservation District in year 2000.

Undisturbed borings were obtained for this study. Boring locations and detailed logs can be furnished upon request. Due to funding issues, borings were mostly obtained in two phases. A small portion of borings were obtained during a previous general investigation of the project area prior to year 2000. Boring investigation for Reaches F, G, H, and I were obtained in 2002 and 2003, including borings for levees and anticipated structure sites. In 2009 to 2010, Boring and CPT data was obtained for project Reaches A, B, E, K, and L. To analyze final designs if this project is authorized, additional deep undisturbed borings (approx. 400), shallow general type borrow borings

(approx. 400), and Cone Penetrometers (CPTs) (approx. 600) will be necessary. Details on this will be furnished upon request.

3.6   Laboratory Tests

For the undisturbed borings, visual classifications were made on all samples obtained from soil borings. Moisture content determinations were made on all cohesive soil samples. Shear strength tests included Unconfined Compression (UCT) tests and Unconsolidated Undrained (Q) tests. Liquid and plastic limits were determined on selected samples. Organic Matter Content tests were also performed on selected undisturbed samples for the 2009/10 boring samples. Consolidation testing was completed on selected undisturbed samples for all borings taken by the Corps or our A-E contractors.

3.7   Shear Strength

Design strengths for clay layers were selected based on the available testing from somewhat limited borings and CPTs. The soil borings were taken along the alignment corridor. Some structure borings near bayous were taken from the adjacent road shoulder, but where accessible by barge, many were obtained in the waterway. Generally, the project area contains large amounts of open water and many bayous, which results in heterogeneous soil foundations. Data from each boring has to be extrapolated over a large area, which can result in inexact assumptions. Efforts were made to provide a realistic design.

 Shear strength data from undisturbed borings, general type borings and CPTs were grouped together for soil reaches of similar deposits. For structures, shear strengths were based on borings in close vicinity of the structure sites. For structures with no site specific boring data, other borings in the vicinity were used to approximate soil properties. Presently, about one-third of the subsurface exploration required for construction of the project has been completed. It is anticipated that the selected shear strength lines for this Feasibility study may be conservative when future boring data and CPTs are used to supplement the shear information.

3.8 Survey Data

Some survey data was provided by the local sponsor's A-E firm. Collection and interpretation of survey data required much more effort than had been anticipated for the services. Ground surface elevations were approximated for reaches with limited survey data. Data provided prior to 2009 was presented in North American Vertical Datum (NGVD) and the 2009/2010 survey data and LIDAR are plotted in NAVD elevations.

3.9 Levee Stability Analyses

The study alignment was divided into stability reaches, based on the height of levee that is required to satisfy hydraulic conditions, types of soils in the foundation, strength of the foundation, and ground surface elevation. A graphical representation of the current proposed alignment will be furnished upon request. The project was divided into 10 stability reaches. Reaches G, H, I, and J were split into subreaches due to differences in levee heights, subsurface conditions, or surface conditions (natural ground elevations, water, miscellaneous). Reaches include the Barrier Alignment, Reaches A, B, E, F, G, H, I, J, K&L. Levee and berm slopes, on the flood (gulf) side, for the levees were controlled or mostly controlled by the hydraulic wave berm requirements. On the protected side, the slopes and berms were designed to meet stability requirements. Geotextile reinforcement was used to increase the stability factor of safety as the levees are enlarged to higher heights. Protection heights for levees analyzed for this study are substantially higher than typical hurricane protection levees in Southeast Louisiana. Generally, the levee footprint is designed to preload the footprint for the second lift to be buttressed by geosynthetic (geotextile fabric) reinforcement to decrease fill requirements as compared to an all earthen fill section. Many shallow bodies of water will be crossed to provide protection to the area. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is available. Such area that may require crossings or closures will be investigated with site specific data, or generalized data for typical crossings. For current designs, crossings and other anomalies were not specifically addressed. It was assumed that the cost to cross the bodies of water will not change the overall cost of each reach.

3.9.1 Stability Criteria

The preloading initial lifts were designed for a safety factor of 1.3 with flood water to the top of levee or Still Water Level (SWL), whichever is greater. Borrow pit locations were based on a levee failure into the pit for a minimum safety factor of 1.5 for the 3% AEP LORR protection levels for a base year (2035) section (with an estimated overbuild). It is anticipated that after years of settlement and subsurface strengthening, some shallow borings and some CPTs can be obtained to verify foundations shear strength assumptions for the next lift. The preload area are to be partially degraded to about elevation 4 (assumed), a reinforcement fabric laid down and compacted fill construction to provide for a substantially higher levee.

3.9.2 Initial Lift (Preload) Construction

When possible, the initial lift for levees will be constructed using adjacent cast soils to provide a base for future lifts. Due to the presence of near surface sands and silts in many of the soil foundations, dewatering or unwatering of borrow pits are not recommended due to uplift issues, stability issues, and potential seepage issues. It is anticipated that adjacent borrow will be excavated in wet conditions (no pump down of water) and will be side cast and stockpiled to dry the clay soils to a reasonable condition (water content) to be placed within section. In many areas of shallow water marsh, the first few feet of fill should be cast or bulldozed into place along the alignment, progressively pushing the borrow outwards away from the centerline of the levee, thus pushing a mud wave of softer, marsh surface soils. The mud wave should be periodically removed and discarded (possible use for marsh creation) so that the soft, weaker soils are not trapped into the levee section foundation. The initial lift will typically be constructed between elevation 12 to 14 with a berm footprint that will help consolidate and strengthen the foundation for future lifts. The Initial levee sections can be utilized for both the 3% AEP and 1% AEP LORR levees.

3.9.3 Stability Analyses for the 3% AEP LORR Base Condition

3.9.3.1 General Levee Conditions

Analyses were performed for the levee heights listed in Figures behind the "Hydraulics Requirements" tab. Levee heights vary from elev. 9 to elev. 18. Survey data ranges from adequate to limited for this scope of analysis. A typical ground surface elevation was estimated for use in the stability analyses. A minimum distance of 40 feet is

required between the levee toe and the intersection of the top of the borrow cut and the ground surface. A crown width of 10 feet was used and required wave berms were used on the floodside.

3.9.3.2   Stability Analyses

Results of the stability analyses will be furnished upon request.

For the 3% AEP LORR levees, Table 106 shows the stability criteria used for the study design sections.

Table 106 - Factor-of-Safety Criteria For 3% AEP Levees

Stability Method	Stability Case	Water Level	Req'd F.S.
SLOPE/W (Spencer)	Prot. Side Levee into Borrow Pit	SWL	1.5
SLOPE/W (Spencer)	Prot. Side Levee	SWL	1.5
SLOPE/W (Spencer)	Floodside Levee into Borrow Pit	LWL	1.5
SLOPE/W (Spencer)	Floodside Levee	LWL	1.4

where
- SWL = Still Water Level (Elev. 9 to 18)
- LWL = Low Water Level (Elev. -1)

The levee stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The levees were analyzed using Spencer Analysis for the SWL with Circular Failure options of the SLOPE/W program. Failure surfaces were optimized and tension cracks were added, both of which provide for a more conservative, lower factor-of-safety. For this study, a block analysis was not performed and could result in a lower possible safety factor. However, in most cases, the difference is minimal and in all cases will not result in a significant cost differential. If this study is approved and funded, future analysis will meet all current construction design criteria.

For analysis, the 3% AEP LORR Base Year (2035) levels and wave berm requirements were used with an estimated overbuild (OB). Conservative assumptions were made for subsurface strengthening due to the initial (preload) lift for the Base Year LORR analyses. In most cases, the overbuild is 1.5 to 2 feet for the crown with some overbuild

for the levee berms. As an added bonus, the overbuild section analyses show that predicted future levels are attainable with established safety factors.

3.9.4   Stability Analyses for the 1% AEP LORR Base Condition

3.9.4.1   General Levee Conditions

Analyses were performed for the levee heights determined through hydraulic analysis. Levee heights vary from elev. 14 to elev. 24. Survey data ranges from adequate to limited for this scope of analysis. A typical ground surface elevation was estimated for use in the stability analyses. A minimum distance of 40 feet is required between the levee toe and the intersection of the top of the borrow cut and the ground surface. A crown width of 10 feet was used and required wave berms were used on the floodside.

3.9.4.2   Stability Analyses

Results of the stability analyses will be furnished upon request.

For the 1% AEP LORR levees, Table 107 shows the stability criteria used for the study design sections.

Table 107- Factor-of-Safety Criteria for 1% AEP Levees

Stability Method	Stability Case	Water Level	Req'd F.S.
SLOPE/W (Spencer)	Prot. Side Levee into Borrow Pit	SWL	1.5
SLOPE/W (Spencer)	Prot. Side Levee	SWL	1.5
SLOPE/W (Spencer)	Floodside Levee into Borrow Pit	LWL	1.5
SLOPE/W (Spencer)	Floodside Levee	LWL	1.4

> where
> - SWL = Still Water Level (Elev. 13 to 23)
> - LWL = Low Water Level (Elev. -1)

The levee stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The levees were analyzed using Spencer Analysis for the SWL with Circular Failure options of the SLOPE/W program. Failure surfaces were optimized and tension cracks were added, both of

which provide for a more conservation, lower factor-of-safety. For this study, a block analysis was not performed and could result in a lower possible safety factor. However, in most cases, the difference is minimal and in all cases will not result in a significant cost differential. As a check, a few sections were evaluated with a block analysis. Generally, the results were similar for most runs, with the exception of a few shallow failure surfaces that ran along a soft clay layer. For these exceptions, the difference in levee fill volume was not significant (less than 2%).If this study is approved and funded, future analysis will meet all current construction design criteria.

For analysis, the 1% AEP LORR Base Year (2035) levels and wave berm requirements were used with an estimated overbuild (OB). Conservative assumptions were made for subsurface strengthening due to the initial (preload) lift for the Base Year LORR analyses. In most cases, the overbuild is 3 to 5 feet for the crown with some overbuild for the levee berms. As an added bonus, the overbuild section analyses show that predicted future levels are attainable within established safety factors.

3.10 Borrow Material

Borrow sources may include adjacent borrow pits and hauled in fill, whenever it is an acceptable source. Information used to determine the adequacy of borrow material along the various alignments included borrow borings where available, borings and CPTs along the baseline, and information supplied by the local sponsor. The undisturbed borings are generally in the foot print of the levee, but in many areas where those borings were obtained at natural ground elevations (below +3), assumptions can be made about the suitability of side cast borrow.

Some of the borings from the local sponsor are not along the alignment, resulting in more extrapolation. There are not enough borings, including vibracores, to accurately cover the area and determine the suitability of borrow with great certainty. A more extensive investigation would have been too time consuming and costly during this phase of the investigation. It is generally anticipated that the first section will primarily serve to preload the foundation of the levees. The fill generally will come from side cast borrow that will need some drying prior to construction of the levee section in horizontal layers not to exceed 3 feet in thickness for the first layer (or until 1 foot above the water) and not to exceed 2 feet in thickness for subsequent layers. To help reduce the water content, the clay material from adjacent borrow areas should be placed within or adjacent to the levee footprint and allowed to dry to a moisture content that shall not be

lower than 40 and no higher than 20 prior to incorporating the fill into the levee layers. The levee section layers will require a minimum effort of compaction (i.e., three passes of a dozer) to achieve a minimum cohesive strength of 300 psf.

Where adjacent material is used, the top five feet may be unsuitable due to the presence of highly organic soils. In some areas, material will be hauled from offsite borrow areas. A much more in-depth borrow investigation will have to be performed at the next level of study.

3.11 Foundation Settlement

Consolidation tests are expensive and time consuming. Some undisturbed boring samples were tested for this phase of the investigation. Settlement predictions are based on limited consolidation data and are not intended to be precise. Predicted settlement curves and associated analysis will be furnished upon request. Settlement predictions were analyzed for the 3% AEP and 1% AEP LORR levees. The area consists mainly of marshland.

Bayous, which pass through the area in a north to south direction and fan out into the gulf, deposit the coarsest soil along their banks. When the bayous meander and change coarse, areas of coarser and less prone to settlement materials are left in erratic configurations. The opposite occurs at areas away from the bayous where fine clay and organic materials accumulate. Settlement in the areas comprised of clay and organic matter is much greater than in the areas that contain the coarser soils. Another important feature of the area is the large number of canals and channels. Settlement calculations for the canal areas are beyond the scope of this study.

These areas will experience more settlement due to the greater heights of fill and the soft canal bottoms. The emphasis of the study was to arrive at settlement values that are in the average range, realizing that the actual value can be more than the calculated values.

3.12 Time Rate of Consolidation

The time required for the foundation to settle is difficult to determine because it depends on many variables, such as past overburden, the amount of organic soils, thickness of clay layers, size of clay platelets, amount of silt or sand in the foundation, the layering of

silts and sands, number of foundation drainage layers, and other factors. Time rate of consolidation was used to predict lift schedules for the economic analysis. The coefficient of consolidation C_v value was extrapolated from available testing data.

3.13 Levee Seepage

This project consists of multiple levee reaches due to foundation soil differences. Design reaches require design elevations that vary from 13.0 to 23.0 NAVD88 (for 1% AEP LORR, Base Yr). A seepage analysis was analyzed for the foundation of Reach F and Reach I. Reach F is believed to be the most vulnerable to seepage due to the presence of near surface sands and will represent a worst case seepage condition for the western portion of the project. Reach I is typical of many of the eastern reaches in regards to seepage. Details on the seepage analyses can be furnished upon request.

3.14 Recommendations

Based on the information that is available, some of the 3% AEP and most of the 1% AEP should not be erected by one lift construction. The amount of soil that is required to provide the required crown elevations for these reaches may exceed the resistance that is available in the foundation and increases the probability for stability failures to unacceptable levels. Construction sequencing is recommended to promote foundation soil strengthening to withstand the loading of future levee enlargements.

3.15 General Structural Design

3.15.1 Geotechnical Analysis of Structures

The project has numerous gated flood control structures across bayous, canals, and roads as well as many water control culverts to allow tidal flow that will be closed during hurricane conditions. Feasibility geotechnical design of sector gates, floodgates (roads), culvert structures, and T-Walls are presented below.

ALL FLOODWALLS: HSDRRS criteria for Floodwalls/T-Wall design as compared to the below referenced Ems, etc. consist of additional checks, list specific methods for analysis, and has a more stringent or equal criteria for various safety factors.

The HSDRRS criteria for T-Wall design list the following USACE Publications for General Design Guidance:

- EM 1110-2-2502, Retaining and Flood Walls, Sept. 89
- EM 1110-2-2906, Design of Pile Foundations, Jan. 91
- EM 1110-2-2504, Design of Sheet Pile Walls, Mar. 94
- EM 1110-2-1913, Design and Construction of Levees, Apr. 00
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, Dec 05
- DIVR 1110-1-400, Soil Mechanic Data, Dec. 98
- ETL 1110-2-569, Design Guidance for Levee Underseepage, May 05

3.15.2 Bayou Dularge Gate Structure Stability Analyses

The proposed sector gate structure was studied using previously obtained strength lines in this area. Both the sector gate and the T-wall were deemed geologically similar enough to use the same strength line for both. Elevations of the top of the structures were supplied by ED-T personnel. No detailed surveys were available at this stage, so ground surface elevations were assumed at EL -7 for the sector gate structure. Low ground water elevations of EL -1 were used. For the purposes of this study, the structures were analyzed with water to the top of the structures. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed.

Due to similarities in the soil parameters, loading, and dimensions, the analysis results for the Bayou Dularge sector gate structure are generally applicable to the Bayou Grand Caillou Sector Gate, Shell Canal E, Bayou Black, Shell Canal West, NAFTA Gates, Minors Canal Gate, and Falgout Canal Sector Gate.

3.15.2.1 Stability Criteria

The sector gates were analyzed for a safety factor of 1.4 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Spencer's stability method was used for analysis of global stability. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required.

3.15.2.2   Stability Analyses

The functionality of the sector gate precludes the use of stability berms. The structure stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The structures were analyzed using Spencer Analysis for the High Water Level (HWL) with Block Failure options of the SLOPE/W program. Failure surfaces were optimized to obtain the highest unbalanced load.

3.15.2.3   Stability Results

The sector gate structure was analyzed using a base slab elevation of -12.0, ground surface elevation of -7.0, flood side (FS) water to the top of the structure (elevation 25.5), and protected side (PS) water elevation of -1.0. Stability analysis determined there are no unbalanced loads for the sector gate. Results of the stability will be furnished upon request. A summary of these results are listed in Table 108 below.

Table 108   Stability Results for Bayou Dularge - Sector Gate

FS Water Elevation	Type of Search	Critical Failure Elevation	Required Factor of Safety	Factor of Safety Obtained
EL. 25.5	Block Search	EL. -51.0	1.40	1.40
EL 25.5	Fully Specified (Optimized)	EL -51.0	1.40	1.52

3.15.2.4   Temporary Retaining Structure

A Temporary Retaining Structure (TRS) was designed for the sector gate structure for cost estimating purposes. Design of the actual TRS is normally required of the contractor. Results of the TRS design will be furnished upon request. A summary of these results are listed in Table 109. The TRS was designed using a combination of CWALSHT and VWALSHT. CWALSHT is a Case program developed by the USACE for the use in designing and analyzing sheetpile structures. VWALSHT is an excel spreadsheet developed by Richard Varuso, PH.D, P.E. for the use in designing a TRS. Due to the similarities in soil parameters, loading, and dimensions, The Bayou Dularge TRS design calculations are applicable for Bayou Grand Caillou Sector Gate, Shell Canal E, Bayou Black, Minors Canal Gate, Falgout Canal Sector Gate, Bayou

Terrebonne, Grand Bayou Sector Gate, Humble Canal Sector Gate, Placid Canal Sector Gate, and Bayou Petite Caillou Sector Gate.

Table 109 Results of Bayou Dularge-Sector Gate TRS Design

Tip Elevation	Bending Moment (Ft-Kips)	Anchor Force 1 (Kips)	Anchor Force 2 (Kips)	Anchor Force 3 (Kips)
EL. -66.3	96.9	7.7 @ EL. -2.0	8.1 @ EL. -7.0	24.8 @ EL. -13.0

3.15.3 Bayou Dularge T-wall Stability Analyses

The Bayou Dularge T-wall was studied using previously obtained strength lines in this area. Both the sector gate and the T-wall were deemed geologically similar enough to apply the same strength line to both. Elevations of the top of the structures were supplied by ED-T personnel. No detailed surveys were available at this design stage, so realistic ground surface elevations were assumed at -1 for the T-wall. Low ground water elevations were also assumed at -1. For the purposes of this study, the structures were analyzed with water to the top of the structures. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed. Due to similarities in the soil parameters, loading, and dimensions, the T-wall analysis results for the Bayou Dularge structure are applicable to the Bayou Grand Caillou Sector Gate, Shell Canal West, NAFTA Gates, Minors Canal Gate, and Falgout Canal Sector Gate.

3.15.3.1 Stability Criteria

The T-wall was designed for a safety factor of 1.4 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Only Spencer's stability method was required for this study. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method will be required.

3.15.3.2 Stability Analyses

No stability berms were designed for the T-wall. The structure stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-

Studio 2007 Suite. The structures were analyzed using Spencer Analysis for the HWL with Block Failure options of the SLOPE/W program. Failure surfaces were optimized to obtain the highest unbalanced load.

3.15.3.3 Stability Results

The T-wall was analyzed using a base elevation of -7.0, ground surface elevation of -1.0, flood side (FS) water to the top of the structure (elev. 25.5), and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was required for the T-wall. Results of the stability analyses will be furnished upon request. These results are summarized in Table 110 below.

Table 110   Stability Results for Bayou Dularge T-Wall

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 25.5	Fully Specified (Optimized)	EL. -51.0	N/A	N/A	1.40	1.11
EL. 25.5	Fully Specified (Optimized)	EL. -51.0	16,500 lbs	EL. -29.0	1.40	1.40

3.15.4 Bayou Terrebonne Sector Gate

The proposed sector gate structure was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were assumed. Only water to the top of the structure was analyzed to provide a reasonable cost estimate. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed. Due to similarities in the soil parameters, loading, and dimensions, the analysis results for the Bayou Terrebonne structure are applicable to the Grand Bayou Sector Gate, Humble Canal Sector Gate, Placid Canal Sector Gate, and Bayou Petite Caillou Sector Gate.

3.15.4.1   Stability Criteria

The sector gate was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Only Spencer's method was required for this study. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required.

3.15.4.2   Stability Analyses

The sector gate was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The structure was analyzed using Spencer Analysis for the HWL with Block Failure and Fully Specified options of the SLOPE/W program. No stability berms were designed since functionality of the sector gate precludes their use. Failure surfaces were optimized to obtain the most critical factor of safety.

3.15.4.3   Stability Results

The sector gate structure was analyzed using a base elevation of -15.0, ground surface elevation of -9.0, flood side (FS) water to the top of the structure (elev. 33.0), and protected side (PS) water elevation of -1.0. Stability analysis (see Table 111 below) determined that an unbalanced load was required for the sector gate.

Table 111   Stability Results for Bayou Terrebonne Sector Gate

Structure Sector Gate Analysis					
Structure	**Sill EL.**	**Base EL.**	**Structural Analysis Factor of Safety**	**Unbalanced Load Needed**	**Factor of Safety without Unbalanced Load**
Bayou Terrebonne SG	-9	-15	1.4	21000 lbs @ EL. -25	0.87

3.15.4.4   Temporary Retaining Structure

A Temporary Retaining Structure (TRS) was not designed for the Bayou Terrebonne sector gate structure. Instead, due to similarities in the soil parameters, loading, and dimensions, the analysis results for the Bayou Dularge Sector Gate (TRS) are applicable to the Bayou Terrebonne Sector Gate. Design of the actual TRS is normally done by the contractor. Results of the TRS design (for cost estimating) are included in Table 112. The TRS was designed using a combination of CWALSHT and VWALSHT. CWALSHT is a Case program developed by the USACE for the use in designing and analyzing sheetpile structures. VWALSHT is an excel spreadsheet developed by Richard Varuso, PH.D, P.E. for the use in designing a TRS.

Table 112   Results of Bayou Terrebonne TRS Design (Via Bayou Dularge)

TRS Analysis				
Tip Elevation	Bending Moment (Ft-Kips)	Anchor Force 1 (Kips)	Anchor Force 2 (Kips)	Anchor Force 3 (Kips)
EL. -66.3	96.9	7.7 @ EL. -2.0	8.1 @ EL. -7.0	24.8 @ EL. -13.0

3.15.5 Bayou Terrebonne T-Wall

The proposed T-Wall was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied by ED-T personnel. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were applied. For the purposes of this study, the structures were analyzed with water to the top of the structures. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed. Due to similarities in the soil parameters, loading, and dimensions, the analysis results for the Bayou Terrebonne structure are applicable to the Grand Bayou Sector Gate, Humble Canal Sector Gate, Placid Canal Sector Gate, and Bayou Petite Caillou Sector Gate.

3.15.5.1   Stability Criteria

The T-Wall was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Spencer's stability method was used for analysis of global stability. For further detailed design and

development for plans, both Spencer's stability and verification by use of another stability method is recommended and required.

3.15.5.2   Stability Analyses

The T-Wall was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The T-Wall was analyzed using Spencer Analysis for the HWL with Block Failure and Fully Specified options of the SLOPE/W program. No stability berms were designed at this time. Failure surfaces were optimized to obtain the most critical factor of safety and the highest unbalanced load (UBL).

3.15.5.3   Stability Results

The T-Wall was analyzed using a base elevation of -9.0, ground surface elevation of -3.0, flood side (FS) water to the top of the structure (elev 33.0), and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was required for the T-Wall. Results of this analysis are summarized in Table 113.

Table 113   Stability Results for Bayou Terrebonne T-Wall

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 33.0	Fully Specified (Optimized)	EL. -35.0	N/A	N/A	1.40	0.69
EL. 33.0	Fully Specified (Optimized)	EL. -35.0	33000 lbs	EL. -22.0	1.40	1.41

3.15.6 Bush Canal Sector Gate

The proposed sector gate structure was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were assumed. For the purposes of this study, the structures were analyzed with water

to the top of the structures. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed.

3.15.6.1 Stability Criteria

The sector gate was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Spencer's stability method was required for this study. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required.

3.15.6.2 Stability Analyses

The sector gate was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. No stability berms were designed since functionality of the sector gate precludes their use. The structures were analyzed using Spencer Analysis for the High Water Level (HWL) with Block Failure options of the SLOPE/W program. Failure surfaces were optimized to obtain the highest unbalanced load.

3.15.6.3 Stability Results

The sector gate structure was analyzed using a base elevation of -18.0, ground surface elevation of -12.0, flood side (FS) water to the top of the structure (elev. 33.0), and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was not required for the sector gate. The results of the analysis on the GIWW-West sector gate are listed in Table 114 .

Table 114   Stability Results for GIWW-West Sector Gate

FS Water Elevation	Type of Search	Critical Failure Elevation	Required Factor of Safety	Factor of Safety Obtained
EL. 33.0	Fully Specified (Optimized)	EL. -50.0	1.40	2.00

3.15.6.4 Temporary Retaining Structure

A Temporary Retaining Structure (TRS) was designed for the sector gate structure to provide a high level cost estimate. Design of the actual TRS is normally done by the contractor. Results of the TRS design are listed in Table 115. The TRS was designed using a combination of CWALSHT and VWALSHT. CWALSHT is a Case program developed by the USACE for the use in designing and analyzing sheetpile structures. VWALSHT is an excel spreadsheet developed by Richard Varuso, PH.D, P.E. for the use in designing a TRS.

Table 115 Results of GIWW-West TRS Design

Tip Elevation	Bending Moment (Ft-Kips)	Anchor Force 1 (Kips)	Anchor Force 2 (Kips)	Anchor Force 3 (Kips)
EL. -51.4	62.4	15.5 @ EL. -5.0	12.5 @ EL. -12.0	17.2 @ EL. -19.0

3.15.7 Bush Canal T-Wall

The proposed T-Wall was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied by ED-T personnel. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were assumed. For the purposes of this study, the T-Walls were analyzed with water to the top of the walls. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed.

3.15.7.1 Stability Criteria

The T-Wall was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required.

3.15.7.2 Stability Analyses

The T-Wall was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The T-Wall was analyzed using Spencer

Analysis for the HWL with Block Failure options of the SLOPE/W program. No stability berms were designed at this time. Failure surfaces were optimized to obtain the most critical factor of safety and the highest unbalanced load (UBL).

3.15.7.3   Stability Results

The T-Wall was analyzed using a base elevation of -12.0, ground surface elevation of -6.0, flood side (FS) water to the top of the structure (elev. 33.0), and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was required for the T-Wall. The results of the analysis on the GIWW-West T-Wall are presented in Table 116.

Table 116   Stability Results for GIWW-West T-Wall

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 33.0	Fully Specified (Optimized)	EL. -30.0	N/A	N/A	1.40	1.39
EL. 33.0	Fully Specified (Optimized)	EL. -30.0	800 lbs	EL. -18.0	1.40	1.40

3.16 Larose (GIWW) Floodwall (T-Wall)

The proposed T-Wall was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied by ED-T personnel. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were applied. For the purposes of this study, the structures were analyzed with water to the top of the structures.

The proposed Larose Floodwall will serve a dual purpose. It will serve to protect the Larose area from high waters in the Gulf Intracoastal Waterway (GIWW). For the Larose to Golden Meadow loop protection, the GIWW is the "floodside" of the protection. If the Morganza to the Gulf protection is authorized, funded and constructed, the GIWW side of the new Larose floodwall will be inside the GIWW East Floodgate and thus will be part of the "protected" side of the MTG loop protection.

3.16.1.1 Stability Criteria

The T-Wall was designed for a safety factor of 1.40 with flood water to the top of wall (TOW). The HSDRRS guidelines were utilized on this study as applicable. Spencer's stability method was used for analysis of global stability. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method is recommended and required.

3.16.1.2 Stability Analyses

The T-Wall was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.19), part of the Geo-Studio 2007 Suite. The T-Wall was analyzed using Spencer Analysis for the HWL (or TOW) with "NonCircular Failure" analysis and "Fully Specified" failure analysis options of the SLOPE/W program. No stability berms were designed at this time. Failure surfaces were optimized to obtain the most critical factor of safety and the highest unbalanced load (UBL).

3.16.1.3 Stability Results

The T-Wall was analyzed using a base elevation of -0.5, ground surface elevation of +2.5, flood side (FS) water to the top of the wall (elev 15.0), and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was required for the T-Wall. The results of the analysis on the Larose Floodwall (T-Wall) are shown in Table 117 .

Table 117 - Stability Results for Larose Floodwall (T-Wall)

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 15.0	Fully Specified (Optimized)	EL. -40.0	N/A	N/A	1.40	1.28
EL. 15.0	Fully Specified (Optimized)	EL. -40.0	55000 lbs	EL. -18.7	1.40	1.40

3.16.2 Pointe Aux Chenes Stability Analyses

The proposed sector gate structure was studied using previously obtained strength lines in this area. The elevation of the top of the structure was supplied. No detailed surveys were available at this stage so realistic ground surface elevations (el. -6.0) were assumed at the sector gate structure. Normal ground water elevation (el. -1.0) was also assumed. Only water to the top of the structure was analyzed to provide a reasonable cost estimate. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed.

3.16.2.1 Stability Criteria

The sector gates were designed for a safety factor of 1.4 with flood water to the top of structure as shown in Table 118. The HSDRRS guidelines were utilized on this study as applicable. Spencer's method was required for this study. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required.

Table 118 - Pointe Aux Chenes Stability Criteria

Stability Method	Stability Case	Water Level	Req'd F.S.
SLOPE/W (Spencer)	TOS FAILING TO PS	HWL	1.4

where
- HWL = High Water Level (Elev. 33.0)
- PS = Protected Side
- TOS = Top of Structure

3.16.2.2 Stability Analyses

The sector gate was analyzed and had unbalanced loads (see Table 119 below). Results of the stability analyses will be furnished upon request. The functionality of the sector gate precludes the use of stability berms. The structure stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The structure was analyzed using Spencer Analysis for the HWL

with Block Failure options of the SLOPE/W program. Failure surfaces were optimized to obtain the highest unbalanced load.

Table 119 - Pointe Aux Chenes Analysis

Structure	Sill EL.	Base EL.	TRS Dredge Elevation	Structural Analysis Factor of Safety	Unbalanced Load Needed	FOS w/o Unbalanced Load
Pointe Au Chenes	-6	-12	-14	1.4	7440 lbs @ EL. -12.5	0.87

3.16.2.3　　　**Temporary Retaining Structure**

A Temporary Retaining Structure (TRS) was designed for the sector gate structure to provide a high level cost estimate (see Table 120 below). Design of the actual TRS is normally done by the contractor. Results of the TRS design will be furnished upon request. The TRS was designed using a combination of CWALSHT and VWALSHT. CWALSHT is a Case program developed by the USACE for the use in designing and analyzing sheetpile structures. VWALSHT is an excel spreadsheet developed by Richard Varuso, Ph.D., P.E. for the use in designing a TRS.

Table 120 - Pointe Aux Chenes TRS Analysis

TRS Analysis	Tip EL. *	Bending Moment (Ft-Kips)	Anchor Force 1 (Kips)	Anchor Force 2 (Kips)	
Pointe Au Chenes	-45	56.2	12.8 @ EL. -5	6.6 @ EL. -13	
All values are to be used for design (bending moments and Tip FS=1.3 and Anchor Force FS=1.0)					
* Sheets for Pointe Au Chenes should be tipped at Elevation -45 to cut off the sand layers above that elevation.					
** Lowest Anchor Force will be Tremie Slab					

3.16.3 Pointe Aux Chenes Stability Analyses

The proposed T-wall was studied using previously obtained strength lines in this area. The elevation of the top of the T-wall was supplied by ED-T personnel. No detailed surveys were available at this stage so realistic ground surface elevations (el. 0.0) were assumed at the T-wall. Normal ground water elevation (el. -1.0) was also assumed.

Only water to the top of the T-wall was analyzed to provide a reasonable cost estimate. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed.

3.16.3.1 Stability Criteria

The T-wall was designed for a safety factor of 1.4 with flood water to the top of wall as shown in Table 121. The HSDRRS guidelines were utilized on this study as applicable. Spencer analysis method was required for this study. For further detailed design and development for plans, both Spencer's stability and verification by use of another stability method (program) is recommended and required during PED.

Table 121 - Pointe aux Chenes T-Wall Stability Criteria

Stability Method	Stability Case	Water Level	Req'd F.S.
SLOPE/W (Spencer)	TOW FAILING TO PS	HWL	1.4

where
- HWL = High Water Level (Elev. 33.0)
- PS = Protected Side
- TOW = Top of Wall

3.16.3.2 Stability Analyses

The T-wall was analyzed and had unbalanced loads (see Table 122). Results of the stability analyses will be furnished upon request. No stability berms were designed for the T-wall. The T-wall stability was analyzed with an industry accepted computer program SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The T-wall was analyzed using Spencer Analysis for the HWL with Block Failure options of the SLOPE/W program. Failure surfaces were optimized to obtain the highest unbalanced load.

Table 122 - Pointe Aux Chenes T-Wall Analysis

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 33.0	Fully Specified (Optimized)	EL. -11.0	N/A	N/A	1.40	0.99
EL. 33.0	Fully Specified (Optimized)	EL. -11.0	6600 lbs	EL. -8.5	1.40	1.40

3.16.4 GIWW-West Sector Gate

The proposed sector gate structure was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied. No detailed surveys were available at this stage so realistic ground surface and ground water elevations were assumed. Only water to the top of the structure was analyzed to provide a reasonable cost estimate. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed. Due to similarities in the soil parameters, loading, and dimensions, the analysis results for the GIWW-West structure are applicable to the GIWW-East structure.

3.16.4.1   Stability Criteria

The sector gate was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Only Spencer's method was required for this study. Both Spencer's and another method will be required during PED.

3.16.4.2   Stability Analyses

The sector gate was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The structure was analyzed using Spencer Analysis for the HWL with Block Failure options of the SLOPE/W program. No stability berms were designed since functionality of the sector gate precludes their use. Failure surfaces were optimized to obtain the most critical factor of safety.

3.16.4.3   Stability Results

The sector gate structure was analyzed using a base elevation of -26.0, ground surface elevation of -16.0, flood side (FS) water to the top of the structure, or elevation 23.0, and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was not required for the sector gate. The results of the analysis on the GIWW-West sector gate are presented in Table 123.

Table 123 - Stability Results for GIWW-West Sector Gate

FS Water Elevation	Type of Search	Critical Failure Elevation	Required Factor of Safety	Factor of Safety Obtained
EL. 23.0	Fully Specified (Optimized)	EL. -49.0	1.40	2.40

3.16.4.4   Temporary Retaining Structure

A Temporary Retaining Structure (TRS) was designed for the sector gate structure to provide a high level cost estimate. Design of the actual TRS is normally done by the contractor. Results of the TRS design are included in Table 124 and Table 125 below. The braced excavation TRS was designed using a combination of CWALSHT and VWALSHT. CWALSHT is a Case program developed by the USACE for the use in designing and analyzing sheetpile structures. VWALSHT is an excel spreadsheet developed by Richard Varuso, PH.D, P.E. for the use in designing a TRS.

Table 124 - Results of GIWW-West TRS Design (Braced Excavation)

Tip Elevation	Bending Moment (Ft-Kips)	Anchor Force 1 (Kips)	Anchor Force 2 (Kips)	Anchor Force 3 (Kips)	Anchor Force 4 (Kips)	Anchor Force 5 (Kips)
EL. -116.6	631.5	10.4 @ EL. -3.0	13.6 @ EL. -9.0	23.0 @ EL. -14.0	31.1 @ EL. -19.0	40.2 @ EL. -27.0
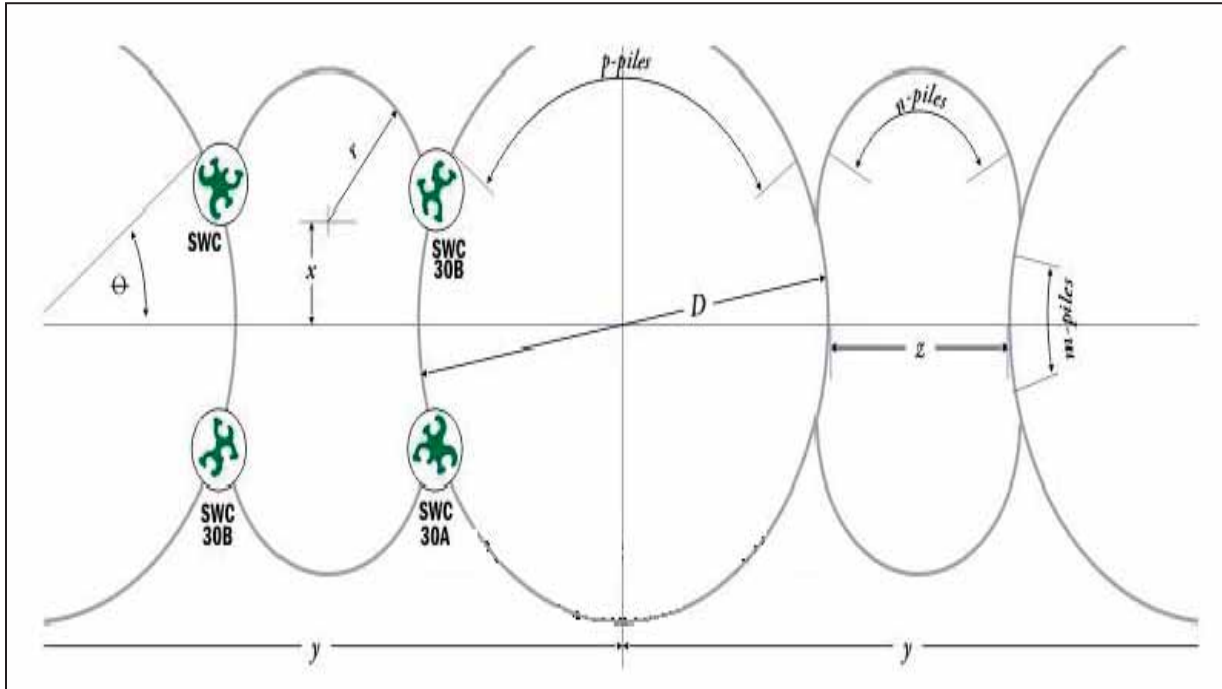
Table 125 - Results of GIWW-West TRS Design (Cellular Cofferdam)

Tip Elevation	# piles	D	z	y	r	X		Number of Piles			Area		Average Width
											within circle sq ft	between circles sq ft	
ft	/ Cell	ft	ft	ft	ft	ft	deg	m	n	p	sq ft	sq ft	ft
-76	132	67.28	19.56	86.83	14.33	16.93	31	22	27	42	3555	1406	57.1

Notes-

1. See Figure below for explanation of terms.
2. Deep Soil Mixing in center of cell to provide an increase in shear strength to El -82 (5 feet below the tip elevation.

3.16.5 GIWW-West T-Wall

The proposed T-Wall was studied using previously obtained strength lines in this area. Elevations of the top of the structures were supplied. No detailed surveys were

available at this stage so realistic ground surface and ground water elevations were assumed. Only water to the top of the T-Wall was analyzed to provide a reasonable cost estimate. Analyses of sporadic low areas (intermittent) are beyond the scope of this study, especially with the data that is currently available. For current designs, crossings and other anomalies were not specifically addressed. Due to similarities in the soil parameters, loading, and dimensions, the analysis results for the GIWW-West T-Wall are applicable to the GIWW-East T-Wall.

3.16.5.1 Stability Criteria

The T-Wall was designed for a safety factor of 1.40 with flood water to the top of structure. The HSDRRS guidelines were utilized on this study as applicable. Only Spencer's method was required for this study. Both Spencer's and another method will be required during PED.

3.16.5.2 Stability Analyses

The T-Wall was analyzed with an industry accepted computer program, SLOPE/W (ver. 7.15), part of the Geo-Studio 2007 Suite. The T-Wall was analyzed using Spencer Analysis for the HWL with Block Failure options of the SLOPE/W program. No stability berms were designed at this time. Failure surfaces were optimized to obtain the most critical factor of safety and the highest unbalanced load (UBL).

3.16.5.3 Stability Results

The T-Wall was analyzed using a base elevation of -16.0, ground surface elevation of -10.0, flood side (FS) water to the top of the structure, or elevation 23.0, and protected side (PS) water elevation of -1.0. Stability analysis determined that an unbalanced load was required for the T-Wall. The results of the analysis on the GIWW-West T-Wall are show in Table 126.

Table 126 - Stability Results for GIWW-West T-Wall

FS Water Elevation	Type of Search	Critical Failure Elevation	Max. UBL (lbs)	Elevation of UBL	Required Factor of Safety	Factor of Safety Obtained
EL. 23.0	Fully Specified (Optimized)	EL. -30.0	N/A	N/A	1.40	0.96
EL. 23.0	Fully Specified (Optimized)	EL. -30.0	9600 lbs	EL. -20.0	1.40	1.40

3.16.6 Dewatering and Sheet Pile Cutoff Design

It was determined, that a geotechnical analysis of the uplift pressures during dewatering needed to be performed for several structures in the Morganza to the Gulf Feasibility Study. Additionally, it was determined that design needed to be performed for the sheet pile seepage cutoff beneath the T-wall and structures. Included herein are the results of those analyses and the assumptions that were made.

3.16.6.1   Uplift Design

Uplift pressures were assumed to develop in the underlying sand strata beneath the 17 structures that were analyzed. Top elevations for temporary retaining structures (TRS), and dredge line elevations where provided, for each of the excavations analyzed. 1% AEP still water elevations (SWE) were provided. Water elevations were assumed to be at the top of the TRS at elevation +6 feet. The total uplift driving head assumed was the difference in elevation from the top of the TRS to the dredge elevation of the excavation.

The USACE standardized method of calculating the FOS for uplift pressures was performed by dividing the resisting force (the weight of the overlying soil strata) by the driving force (pore water pressure in the critical sand strata). More details on this design will be furnished upon request.

3.16.6.2   Seepage Cutoff Design

The 21 structures were designed by Lanes Creep Method for sheet pile seepage cutoff for the levee to structure transitions beneath the T-wall. Top of wall (TOW) structure elevations and sill elevations were provided for each of the structures analyzed. 1%

AEP still water elevations (SWE) were provided. The water head assumed was the difference in elevation from the TOW to the protected side water elevation of -1 feet.

Seepage cutoff design was performed for each of the provided reaches. Each of the provided reaches had a varying number of benches and elevations in the transition cross sections. The varying number of benches and elevations in the different transition cross sections affected the sheet pile design by reach, because of the soil stratification that the sheet piling was tipped. A numerical system of labeling the benches from below the structure elevation to the next highest bench towards the levee and so forth was used for sheet pile tip elevation design.

For sheet piling design it was assumed that the soil beneath the pile founded T-wall will settle and leave a gap between the base and the soil for a flow path. Thus the only seepage cutoff path taken into consideration in the Lanes Weighted Creep Ratio analysis was that of the sheet piling beneath the T-wall foundation. Sand and silt layers that the sheet piling penetrated were transformed by their corresponding creep ratios to that of a CH layer thickness with an equivalent creep ratio. More details of this analysis will be furnished upon request.

3.16.7 Pile Capacity For Structures

3.16.7.1   Design Methods and Assumptions

Computations were made to determine the estimated allowable single pile load capacities for various sizes of precast concrete piles and steel H-Piles. Capacities were computed for piles driven from ground surfaces or from the excavated base elevations for various structures. The pile capacities were computed for Q-case and S-case soil parameters in accordance with EM 1110-2-2906 and the HSDRRS design guidelines (Jun 08 revised version).

The allowable load capacities for piles will be furnished upon request. Some provided curves are used for the multiple structures in the area with similar foundation conditions. The S-case analyses do not govern design. The pile capacities provided should be reduced by 33 percent (67 percent of indicated capacities) if a pile load test is not performed.

Q and S-case computations are plotted as ultimate capacity. Without a site specific pile

test, a Factor-of-Safety of 3.0 and 1.5 is required for the Q-Case and S-Case, respectively, against failure of a single pile.

Where unbalanced loads exist at the T-wall structures, the axial capacity of the supporting piles above the identified critical depth should be ignored for support of the structure.

3.16.7.2   Pile Group Capacity and Spacing

Piles will derive a majority of their supporting capacity from skin friction. Therefore, it will be necessary to consider the effect of group action.

3.16.7.3   Estimated Settlement

Long-term settlement of individual pile foundations are typically not significant and usually in the range of ½ to ¾ inch. This estimate assumes piles will be driven in rows and does not include the elastic deformation of the piles. Elastic deformation can better be defined during the pile load test.

3.16.7.4   Pile Driving

Close field supervision should be maintained by experienced personnel to ensure proper procedures are followed and accurate records are kept during pile driving operations. The driving record should include the pile type, overall length, tip and butt diameters, embedment below finished grade, and number of blows per foot of penetration. An accurate driving record is especially important to verify piles are installed to the required tip embedment and to give an indication of any unusual driving characteristics that may indicate pile breakage. Square precast concrete piles and steel H-Piles should be driven with a single acting air hammer with the hammer manufacturer's recommended rated energy (ft-lbs) per blow for each type (and length) of pile.

3.16.7.5   Dynamic Pile Tests

If needed, a precast concrete test piles can be evaluated by a Dynamic Pile Test (DPT) using a Pile Driving Analyzer® (PDA) during the pile's installation. The PDA will monitor driving stresses during installation and evaluate pile integrity during installation. The PDA will also evaluate installation efficiency by monitoring the energy transferred to the

pile by the hammer.

3.16.7.6   Test Piles

Test piles should be installed along the project site. The number and location of the test piles will depend on the type and location of the project features. The test pile program will be developed once the project features are finalized.

3.16.7.7   Static Load Tests

A series of load tests will be performed on piles considered for the project. The number of load tests will depend on the project features and will be provided during preparation of the plans and specifications. In general, load tests should be performed in accordance with ASTM D 1143. Project specifications will require load tests to failure or 300 percent of design load, whichever is achieved first. Static load tests will be performed no earlier than 21 days after initial pile installation.

3.16.7.8   Monitoring Considerations

Installation of piles may affect nearby structures. When structures are nearby, vibrations should be monitored during the test pile program, installation of job piles, installation and removal of sheetpiles, and any demolition or other construction activities. The monitoring should be performed with a seismograph to evaluate peak particle velocities and frequency at critical structures during pile driving. The record of peak particle velocities should provide information in assessing potential damage and the need for changes in driving operations.

4   CIVIL DESIGN

4.1   Design Hurricane

The design hurricane for feasibility purposes are the 1% AEP hurricane and the 3% AEP hurricane. These storms represent a 1% AEP chance of occurring and 3% AEP chance of occurring in any given year respectively. See Section 2 of this report for specifics on storm modeling.

4.2   Levee Elevations

Levee elevations were determined by ensuring that the levee elevation equaled or

exceeded the required hydraulic design elevation at any given point in time. To achieve this, each reach requires 2 or 3 lifts between 2019 and 2070 due to subsidence, consolidation, and other factors detailed in Section 2 and Section 3 of this report. Settlement curves and lift schedules with specific settlement and elevation data will be furnished upon request. Each reach was analyzed independently.

The initial construction resulting from this authorization will be considered and termed the Pre-Load (Initial Levee). With the exception of reach LGM (overlap) all reaches will require Pre-Load (Initial Levee) construction. Reach LGM (overlap) existing levee is within the proposed levee envelope and substantial enough to warrant accounting. All other reaches will ignore existing levees due to mis-alignment of centerline and insubstantial quantities. Enlargements to the Pre-Load (Initial Levee) will be termed lifts.

The Pre-Load (Initial Levee) design elevation will vary between elevation 10.0' and 14.0' NAVD88 depending on the reach and serves primarily as preload for future lifts. The Pre-Load (Initial Levee) design sections for all reaches, except for reach L2L B, will be the same for 1% AEP and 3% AEP level of risk reduction (LORR). Final lift elevations for the 3% AEP LORR will vary between elevations 13.0' and 20.0' NAVD88 depending on reach. Final lift elevations for the 1% AEP LORR will vary between elevations 22.0' and 28.0' NAVD88 depending on reach.

4.3 Levee Alignment

See Section 1.1 for location and description of project area. The Levee alignment includes 14 levee reaches including reaches BA, A, B, E, F, G, H, I, J, K, L, LGM (Overlap), L2L A, and L2L B and is broken down further into sub reaches as shown in Table 127.

Table 127 – Levee Sub Reaches

Reach Data				
Reach	Start Station	End Station	Length (ft.)	Lenth (mi.)
BA	1000+00	1828+22	82,822	15.7
A	1828+22	2259+26	43,104	8.2
B	2259+26	2526+34	26,708	5.1
E	2526+34	2758+84	23,250	4.4
F	2758+84	2987+35	22,851	4.3
G	2987+35	3224+12	23,677	4.5
H	3224+12	3640+67	41,655	7.9
I	3640+67	3941+76	30,109	5.7
J	3941+76	4438+85	49,709	9.4

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Reach Data				
Reach	Start Station	End Station	Length (ft.)	Lenth (mi.)
K	4438+85	4706+99	26,814	5.1
L	4706+99	5021+78	31,479	6.0
		Sub Total	402,178	76.2
LGM (Overlap)	2953+00	3193+50	24,050	4.6
L2L A	2481+08	2810+00	32,892	6.2
L2L B	2160+69	2481+08	32,039	6.1
		Sub Total	88,981	16.9
		Total	491,159	93
Sub Reach Data				
Sub-Reach	Start Station	End Station	Length (ft.)	Lenth (mi.)
BA	1000+00	1828+22	82,822	15.7
A	1828+22	2259+26	43,104	8.2
B	2259+26	2526+34	26,708	5.1
E2	2526+34	2646+34	12,000	2.3
E1	2646+34	2758+84	11,250	2.1
F2	2758+84	2860+35	10,152	1.9
F1	2860+35	2987+35	12,700	2.4
G1	2987+35	3098+96	11,161	2.1
G2	3098+96	3191+53	9,257	1.8
G3	3191+53	3224+12	3,259	0.6
H1	3224+12	3319+12	9,500	1.8
H2	3319+12	3460+81	14,168	2.7
H3	3460+81	3640+67	17,986	3.4
I1	3640+67	3733+30	9,263	1.8
I2	3733+30	3832+26	9,896	1.9
I3	3832+26	3941+76	10,950	2.1
J2	3941+76	4202+33	26,058	4.9
J1	4202+33	4367+74	16,541	3.1
J3	4367+74	4438+85	7,111	1.3
K	4438+85	4706+99	26,814	5.1
L	4706+99	5021+78	31,479	6.0
		Total	402,178	76.2

Reach Data				
Reach	Start Station	End Station	Length (ft.)	Lenth (mi.)
LGM (Overlap)	2953+00	3193+50	24,050	4.6
L2L A	2481+08	2810+00	32,892	6.2
L2L B	2160+69	2481+08	32,039	6.1
		Sub Total	88,981	16.9
		Total	491,159	93

4.4   Construction

For each reach (11-total), the Pre-Load (Initial Levee), will be constructed to the design dimensions and elevations. All design information and data is available upon request. All reaches except AB (3% only) and A (3% only) are designed with reinforcement geotextile. During first lift construction, the pre-load (initial levee) will be degraded for installation of reinforcing geotextile to approximate elevation +4.0' NAVD88 except reach L2L A and L2L B where degrading existing levee and placement of geotextile will placed during pre-load (initial levee) . Prior to the embankment construction, clearing and grubbing of the levee footprint will be required. For reach AB to L, the Pre-Load (Initial Levee) material will be excavated from the adjacent government furnished borrow area(s) land and/or flood side of levee within new rights of way. For reach LGM (overlap) to L2L B material will be hauled in. In adjacent borrow cases, the distance between the levee toe and top of borrow pit will vary from 50 feet to 125 feet. Borrow pits adjacent to and flood side of levee will require plugs or natural crossings every 500-feet to prevent erosive channeling of the pits. Borrow pits were sized assuming in place borrow to in place levee embankment ratio of 2-1 applied after stripping the top 3'-5' of unsuitable material for levee construction. The unsuitable levee material will be used for wetland mitigation. Borrow pit geometry will be 1V to 4H side and end slopes excavated to bottom elevation -20.0' NAVD, varying bottom widths, and discrete varying lengths adjacent and parallel to the centerline alignment on either the flood side or protected side. Levee grade material will be temporarily stockpiled then processed and placed within the authorized rights-of-way. Levee grade material will be placed and compacted in lifts and strictly tested in accordance with the QA/QC guidelines described in Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRS).

4.5 Embankment Material (Levee Grade Material)

Material used for embankment will be levee grade material meeting the HSDRRS guidelines. Levee grade material is currently defined and specified as follows-Earth materials naturally occurring or Contractor blended materials that are classified in accordance with ASTM D2487 as CL or CH with less than 35% sand content are suitable for use as embankment fill (Materials classified as ML are suitable if blended to produce a material that classifies as CH or CL according to ASTM D 2487). Materials shall be free from masses of organic matter, sticks, branches, roots, and other debris including hazardous and regulated solid wastes. Isolated pieces of wood will not be considered objectionable in the embankment provided their length does not exceed 1 foot, their cross-sectional area is less than 4 square inches, and they are distributed throughout the fill. Not more than 1 percent (by volume) of objectionable material shall be contained in the earth material placed in each cubic yard of the levee section. Pockets and/or zones of wood shall not be placed in the embankment. Materials placed in the section must be at or above the Plasticity Index of 10. Materials placed in the section must be at or below organic content of 9 percent by weight, as determined by ASTM D 2974, Method C.

All levee grade material will be moisture controlled and compacted. Levee grade material will be compacted to at least 90% maximum dry density as determined by ASTM D 698 (Standard Proctor Compaction Test) at a moisture content within the limits of plus 5 to minimum 3 percentage points of optimum moisture content determined from ASTM D 698. Compactive techniques and effort vary but are typically some combination of mechanical rollers, scrapers, dozers and dump trucks to achieve the required compaction.

4.5.1 Future Levee Enlargements

The levee will be constructed in lifts as described in section 4.2 Levee Elevations. Stability/wave berms are a key component of future levee design, providing additional stability and wave protection. Prior to first lift construction, the existing levee is projected to settle to approximate elevation+10.0' NAVD88. Settlement curves and lift schedules for reach specific settlement and elevation data will be furnished upon request. The first levee enlargement (lift one) will begin in years 2019 thru 2045. Reinforecment geotextile will be placed as described above. The material excavated as a result of the degrade operation will be temporarily stockpiled then re-used as compacted fill within the new levee section. Final lifts will occur in Years 2045 to 2070 completing the authorized levee. Borrow for construction of future lifts embankment will be obtained offsite at Government furnished borrow sources and truck hauled to the levee site. The average haul distance between the borrow source and construction site was assumed to be 25 miles for the 1% AEP alternative and 20 miles for the 3% AEPalternative. Borrow pit locations are not identified at this time. The material

excavated for construction of embankment will be temporarily stockpiled and processed at the borrow pit site within Government rights-of-way prior to vehicular transport to the levee construction site. The embankment operation will also include borrow pit development, clearing and grubbing of the levee footprint, placing and compacting levee grade material, and fertilizing and seeding all disturbed areas.

4.6 Quantity

Pre-Load (Initial Levee) volume quantities were calculated by average end area method between the proposed Pre-Load (Initial Levee) section and existing grades. Existing grades were roughly estimated from low resolution Contractor furnished surveys and typically simplified as a plane at elevations varying between -2.0' and 0.0' NAVD88 depending on reach. See typical sections for reach specific existing grades. Lift volume quantities were calculated by average end area method between the lift and the Pre-Load (Initial Levee) and/or the existing grade where the lift extends beyond the Pre-Load (Initial Levee). The Pre-Load (Initial Levee)section was vertically adjusted to account for settlement shown in the settlement curves prior to area calculations for future lifts. Levee crowns were adjusted by the full amount shown on the settlement curves and levee berms were adjusted by an average of 1.0'. Finally a shrinkage factor of 12% was applied to all volume quantities. The volume of the existing levee is not included in volume calculations.

5   STRUCTURE DESIGN

5.1   Structural Project Features

This section summarizes the feasibility design work that was performed to develop sufficient quantities for the structural features that are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. A limited design approach was followed due to the volume of structures within the alignment as well as the limited geotechnical and civil site data at some locations. For most structural features, designs were prepared for a limited number of structures and the remainder of the structures were pro-rated. Some structural features were not designed at all, rather quantified based on similar features from historical structures constructed with MVN, such as timber guidewalls and pile clusters. More critical, costly components such as the pile foundations for the structures were designed and quantified for each structure where adequate geotechnical site data was available.

5.2 General Structural Design Criteria

5.2.1 References

All design is in accordance with the Hurricane and Storm Damage Risk Reduction System Design Guidelines, New Orleans District Engineering Division, dated 23 October 2007 with Addenda dated 12 June 2008 along with other applicable Corps engineering guidance and applicable industry standards. Where there are discrepancies between the Hurricane and Storm Damage Risk Reduction System Design Guidelines and other references, the requirements of the Hurricane and Storm Damage Risk Reduction System Design Guidelines superseded the other references.

5.2.1.1   Technical Publications

- American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary (ACI 318-08).
- American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, 9th Edition.

5.2.1.2   Corps of Engineers Publications

- Hurricane and Storm Damage Reduction System Design Guidelines, New Orleans District, 12 June 2008.
- EM 1110-2-2000   Standard Practice for Concrete for Civil Works Structures Change 2 (Mar 01).
- EM 1110-2-2104   Strength Design Criteria for Reinforced Concrete Hydraulic Structures (Jun 92, Aug 03).
- EM 1110-2-2105 Design of Hydraulic Steel Structures Change 1 (May 94).
- EM 1110-2-2502 Retaining and Floodwalls (Sep 89).
- EM 1110-2-2503   Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures (Sep 89).
- EM 1110-2-2703 Lock gates and Operating Equipment (Jun 94).
- EM 1110-2-2906 Design of Pile Foundations (Jan 91).
- ER 1110-2-8152 Planning and Design of Temporary Cofferdams and Braced Excavation (Aug 94).

5.2.1.3   Computer Programs

- Structural Analysis and Design Software, "STAAD.Pro 2006", release 23W, Research Engineers
- CE Pile Group Analysis Program, "CPGA", CASE Program No. X0080
- "Mathcad", Version 14.0.2.5, Parametric Technology Corporation

- "Microsoft Excel", 2007, Microsoft Corporation

5.2.2 General Design Criteria

Table 128 below provides the general load cases that were examined for all flood protection elements. The allowable overstress values taken are based on the 12 June 2008 revision of the HSDRRS criteria. For hydraulic steel structure (HSS) design, the allowable stress was multiplied by 0.83 times that allowed by AISC in accordance with EM 1110-2-2105.

Table 128 - General Load Cases

LOAD CASE	% ALLOWABLE OVERSTRESS	
	WALL	FOUNDATION
CONSTRUCTION	16 2/3	16 2/3
CONSTRUCTION + WIND	33 1/3	33 1/3
SWL	0	0
SWL + WIND	33 1/3	33 1/3
SWL + WAVE	33 1/3	33 1/3
SWL + WIND + BOAT IMPACT (BI)	50	33 1/3
REVERSE HEAD	0	0
REVERSE HEAD + WIND	33 1/3	33 1/3
REVERSE HEAD + WIND + BI	33 1/3	33 1/3
TOW	33 1/3	33 1/3
TOW + UNBALANCED LOAD	50	50
MAINTENANCE DEWATERING	16 2/3	16 2/3
MAINTENANCE DEWATERING + WIND	33 1/3	33 1/3
MAINTENANCE DEWATERING + BI	50	50

5.2.2.1 Unit Weights

Unit weights utilized for structural design are summarized in Table 129.

Table 129 - Unit Weights

Item	LBS/CF*
Water	62.4
Steel	490
Granular Fill(saturated)	120
Stone	132
Stabilization Slab Concrete	135
Normal Weight Concrete	150

*Unit weights taken from HSDRRS guidelines.

5.2.2.2 Loadings

5.2.2.2.1 Water Elevations

The water elevations used for design are shown in Table 130.

Table 130 - Water Elevations

Structure	SWL F/S 3% AEP	SWL F/S 1% AEP	SWL P/S	TOW F/S 3% AEP	TOW F/S 1% AEP	TOW P/S	Reverse F/S	Reverse P/S	Maint. Dewat.
Bayou Black 56' SG	10.6	14.71	-1	15	22	-1	-0.87	4.58	5
Shell Canal East 56' SG	10.93	15.24	-1	16	23.5	-1	-1.35	2.87	5
Minors Canal 56'SG	11.32	15.6	-1	16	23	-1	-1.25	2.54	5
Falgout Canal 56' SG	11.85	15.61	-1	16.5	23	-1	-0.74	3.06	5
Bayou DuLarge 56' SG	13.77	18.21	-1	18	25.5	-1	-0.74	3.06	5
Bayou Grand Caillou 56' SG	13.77	18.21	-1	18	25.5	-1	-0.74	3.11	5
Bayou Petite Caillou 56' SG	13.71	17.7	-1	22.5	30.5	-1	-2.76	3.49	5
Placid Canal 56' SG	14.74	18.64	-1	24	31.5	-1	-2.76	3.49	5
Bush Canal 56' SG	15.48	19.63	-1	25	33	-1	-1.14	2.99	5
Bayou Terrebonne 56' SG	15.48	19.63	-1	25	33	-1	-1.64	3.10	5
Humble Canal 56' SG	15.48	19.63	-1	25	33	-1	-1.64	1.19	5
Pointe Aux Chenes 56' SG	15.48	19.63	-1	25	33	-1	-1.64	3.00	5
Grand Bayou 56' SG	14.69	19.75	-1	21	29.5	-1	-2.76	3.49	5
GIWW West 125' SG	11.32	15.6	-1	16	23	-1	-1.25	3.04	5
GIWW East 125'SG	12.06	18.2	-1	17	25	-1	-1.64	3.96	5
Elliot Jones 20' Stoplog Gate	10.93	15.24	-1	16	23.5	-1	-1.35	3.28	5
Humphreys Canal 20' Stoplog Gate	11.31	15.75	-1	16	23.5	-1	-1.35	3.28	5
Shell Canal West 30' Stoplog	10.93	15.24	-1	16	23.5	-1	-1.35	2.87	5

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Structure	SWL F/S 3% AEP	SWL F/S 1% AEP	SWL P/S	TOW F/S 3% AEP	TOW F/S 1% AEP	TOW P/S	Reverse F/S	Reverse P/S	Maint. Dewat.
Gate									
Marmande Canal 30' Stoplog Gate	11.85	15.61	-1	16.5	23.0	-1	-2.76	3.30	5
Four Point Bayou 30' Stoplog Gate	13.71	17.7	-1	22.5	30.0	-1	-2.76	3.30	5
Barrier 1 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 2 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 3 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 4 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 5 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 6 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Barrier 7 ECS	11.31	15.75	-1	16.0	23.5	-1	N/A	N/A	5.0
Reach A ECS	11.32	13.71	-1	16.0	22.5	-1	N/A	N/A	5.0
Reach E-1 ECS	13.77	18.21	-1	18.0	25.5	-1	N/A	N/A	5.0
Reach E-2 ECS	13.77	18.21	-1	18.0	25.5	-1	N/A	N/A	5.0
Reach G-2 – 1 ECS	13.71	17.70	-1	22.5	30.5	-1	N/A	N/A	5.0
Reach G-2 – 2 ECS	13.71	17.70	-1	22.5	30.5	-1	N/A	N/A	5.0
Reach G-2 – 3 ECS	13.71	17.70	-1	22.5	30.5	-1	N/A	N/A	5.0
Reach H-1 – 1 ECS	13.71	17.70	-1	22.5	30.5	-1	N/A	N/A	5.0
Reach H-1 – 2 ECS	13.71	17.70	-1	22.5	30.5	-1	N/A	N/A	5.0
Reach J2 – 1 ECS	15.48	19.63	-1	25.0	33.0	-1	N/A	N/A	5.0
Reach J2 – 2 ECS	15.48	19.63	-1	25.0	33.0	-1	N/A	N/A	5.0
Reach J2 – 3 ECS	15.48	19.63	-1	25.0	33.0	-1	N/A	N/A	5.0
Reach K – 1 ECS	14.69	19.75	-1	21.0	29.5	-1	N/A	N/A	5.0
Reach K – 2 ECS	14.69	19.75	-1	21.0	29.5	-1	N/A	N/A	5.0
Reach L ECS	14.69	19.75	-1	21.0	29.5	-1	N/A	N/A	5.0
Madison PS Fronting Protection	15.48	19.63	N/A	25.0	33.0	N/A	N/A	N/A	N/A
Pointe Aux Chenes PS Fronting Protection	15.48	19.63	N/A	25.0	33.0	N/A	N/A	N/A	N/A
Bayou Black PS Fronting Protection	11.31	15.75	N/A	16.0	23.5	N/A	N/A	N/A	N/A
Hanson Canal PS Fronting Protection	11.31	15.75	N/A	16.0	23.5	N/A	N/A	N/A	N/A
Hwy 315 Swing Gate	13.77	18.21	N/A	18.0	25.5	N/A	N/A	N/A	N/A
Hwy 55 Swing Gate	13.71	17.7	N/A	25.0	33.0	N/A	N/A	N/A	N/A
Hwy 56 Swing Gate	13.71	17.7	N/A	22.5	30.0	N/A	N/A	N/A	N/A

Structure	SWL F/S 3% AEP	SWL F/S 1% AEP	SWL P/S	TOW F/S 3% AEP	TOW F/S 1% AEP	TOW P/S	Reverse F/S	Reverse P/S	Maint. Dewat.
Hwy 665 Swing Gate	15.48	19.63	N/A	25.0	33.0	N/A	N/A	N/A	N/A
Four Point Road Swing Gate	15.48	19.63	N/A	22.5	30.0	N/A	N/A	N/A	N/A
NAFTA Swing Gate	11.31	15.75	-1	16	23.5	-1	-1.35	3.28	5
C North Gulf South Pipeline	13.2	17.4	-1	18	23	-1	N/A	N/A	N/A
C North American Midstream Pipeline	13.2	17.4	-1	18	23	-1	N/A	N/A	N/A
C North Williams Discovery Pipeline	13.2	17.4	-1	18	23	-1	N/A	N/A	N/A
ECS Lockport to Larose 1	8.8	10.8	-1	15	18	-1	-0.74	3.11	5
ECS Lockport to Larose 2	8.8	10.8	-1	15	18	-1	-0.74	3.11	5
Union Pacific Railroad 36' Swing Gate	8.8	10.8	-1	15	18	-1	N/A	N/A	N/A
Larose FG 56' SG	9	11.3	-1	14	17	-1	-0.74	3.11	5
GIWW Floodwall and Hwy 24 and Hwy 3235 36' Swing Gates	9	11.3	-1	14	17	-1	N/A	N/A	N/A

5.2.2.2.2 Lateral Pressure

Use Unit Weight and K at rest values
Ko = 0.8 for clay
Ko = 0.5 for granular materials
Ko = 0.5 for rip rap

The above at rest pressure coefficients are per the HSDRRS guidelines.

5.2.2.2.3 Wind Pressures

The wind force utilized for design was 50 psf for hurricane conditions and 20 psf for maintenance conditions.

5.2.2.2.4 Wave Loadings

Wave loadings utilized for design were provided by ED-H. The ED-H analysis will be furnished upon request. Wave loadings were not investigated for the Larose to Lockport Reach due to the short duration of the feasibility design effort for that reach. Wave load conditions typically did not govern for the remainder of the Morganza to the

Gulf alignment structures.

5.2.2.2.5 Boat Impact

5.2.2.2.5.1   Concrete Structures

Although the HSDRRS design guidelines do not specifically provide impact maps for Morganza to the Gulf, judgment was used to apply a 50 kip pleasure craft load to all 56' sector gate crossings and stop log gate crossings based on the fishing vessels and pleasure craft that will use these structures. The 125' sector gate crossings were designed for the unusual impact force of 200 kips in accordance with the HSDRRS design guidelines. A .5 kip/ft debris impact force was applied to all environmental control structures and fronting protection T-Walls based on the structures not being located on navigable waterways. The T-Walls along the GIWW and the tie-in walls for the Larose FG were designed for the unusual impact force of 200 kips in accordance with the HSDRRS design guidelines for a protected side impact from the GIWW and designed for the 50 kip pleasure craft load on the flood side.

5.2.2.2.5.2   Sector Gate Channel Truss in Open Position and Sector Gate in Closed Position

The sector gate leaves were designed for a 125 kip impact force applied at each joint along the channel truss/skin plate in accordance with the requirements of EM 1110-2-2703, "Lock gates and Operating Equipment".

5.2.2.2.5.3   Steel Roadway Swing Gates

Although the HSDRRS design guidelines do not specifically provide impact maps for Morganza to the Gulf, judgment was used to apply a 50 kip pleasure craft load to the roadway swing gates based on the fact that they are adjacent to navigation structures that fishing vessels and pleasure craft use.

5.2.2.2.6 Uplift Conditions

Uplift conditions utilized for design were in accordance with Chapter 5 of the HSDRRS design guidelines.

Impervious - Sheet pile cutoff is assumed 100% effective
Pervious - Linearly varying between the F/S and P/S elevations

5.2.3 Concrete Design General Requirements

5.2.3.1   Reinforced Concrete Strength

All reinforced concrete will have a design compressive strength of 4000 psi.

5.2.3.2   Load Factors

Reinforced concrete hydraulic structures were designed in accordance with EM 1110-2-2104. EM 1110-2-2104 procedures are referenced to the load factors and strength reduction factors found in ACI 318-08, Appendix C.

A single load factor of 1.7 was used for dead and live loads in addition to a hydraulic factor of 1.3.

Strength reduction factor for bending = 0.9

Strength reduction factor for shear = 0.85

5.2.4 HNC Lock

Houma Navigation Lock was not designed as part of this study. The 50% Design Documentation Report (DDR) dated July 2008 by URS Group, Inc. was used to develop costs for this particular feature. Because the 50% DDR was based off a different level of protection (El 24.5) and sill elevation (El -23.0); the quantities were pro-rated based on the elevation/ hydrostatic pressure differences. The quantities contained within this study are based on a sill elevation of El -18.0 in conjunction with a 3% AEP level of protection of El 22.5 and a 1% AEP level of protection of El 30.5.

5.2.5 56' Sector Gates

This section contains a summary of work for the 13, 56' sector gate structures, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 131 lists the structures examined.

Table 131 - Structures

Structure	Sill Elevation	Top Elevation (3% AEP)	Top Elevation (1% AEP)	Top of Guidewalls
Bayou Black	-12	15	22	10
Shell Canal East	-12	16	23.5	10
Minors Canal	-9	16	23	10
Falgout Canal	-9	16.5	23	10
Bayou DuLarge	-7	18	25.5	10
Bayou Grand Caillou	-12	18	25.5	10
Bayou Petite Caillou	-8	22.5	30.5	10
Placid Canal	-8	24	31.5	10
Bush Canal	-12	25	33	10
Bayou Terrebonne	-9	25	33	10
Humble Canal	-9	25	33	10
Pointe Aux Chenes	-6	25	33	10
Grand Bayou	-9	21	29.5	10
Larose	-12.3	14	17	10

5.2.5.1   Physical Features

The physical features associated with the construction of the 56 ft sector gate structures are as follows:

- Temporary Bypass Channels
- Phase 1 and 2 Interior Braced Cofferdams
- Sector Gate Concrete Monolith
- Sector Gate Pile Foundation
- Steel Sector Gate
- Needle Girder, Needles and Supports
- Needle Girder Storage Platform
- Guidewalls and Pile Clusters
- Sluice Gate Concrete Monolith*
- Sluice Gate Pile Foundation*
- Sluice Gates*
- Sluice Gate Bulkheads*
- Tie-in T-Walls
- Electrical Controls and Circuitry
- Mechanical Equipment

*(Bayou Grand Caillou, Bush Canal, Falgout Canal, Grand Bayou, Placid Canal and Bayou Petite Caillou only)

5.2.5.2   Construction Sequencing

All sector gates will be constructed approximately in the center of the existing channels. A minimum 60' temporary bypass channel will be constructed as the first order of construction, allowing navigation passage during construction. Once navigation is routed through the temporary bypass channel, a cofferdam will be constructed, permitting the construction of the 56' sector gate monolith and the sluice gate monoliths, if applicable. Reduced power will be required for vessels passing through the construction area to reduce the risk of impact to the cofferdam. A timber guidewall and pile clusters will be provided along the bypass channel to prevent vessel impact on the cofferdam. Once construction of the 56' sector gate monolith and sluice gate monoliths is completed, navigation will be re-routed through the permanent sector gate structure. A phase 2 cofferdam will be required for the T-Walls adjacent to the sector gate/sluice gate structures. Once navigation is re-routed, the phase 2 cofferdam, needle girder storage platform, permanent guidewalls and pile clusters, tie-in t-walls and final civil site work can be completed.

5.2.5.3   Structural Design

5.2.5.3.1 Cofferdams

A Phase 1 cofferdam will be constructed to permit the in the dry construction of the sector gate concrete monolith and the sluice gate concrete monolith (if applicable). The cofferdam is an internally braced cofferdam with wide-flange walers and pipe braces supporting PZ sheet piling. Anchor forces, bending moment in the sheet piling, and required sheet piling tip elevation were computed for for Bush Canal, Bayou Dularge and Point Aux Chenes. Bayou Dularge cofferdam design was conservatively used for all remaining structures where no design was performed. Details of the Phase 1 cofferdam can be found on Plate S-024.

A phase 2 cofferdam will be constructed to permit the construction of the adjacent T-Walls to the sector gate/sluice gate structures that will be in the water. The same anchor forces, moments, and tips used for the Phase 1 cofferdams will be conservatively used for the Phase 2 cofferdams. Details of the Phase 2 cofferdam can be found on Plate S-100.

ER 1110-2-8152 will be followed throughout the project design process, requiring that all cofferdams will be designed by the Government.

5.2.5.3.2 Sector Gate Monolith Concrete

Details of the concrete monolith are shown on Plates S-002 to S-004 (3% AEP LORR) and S-002A to S-004A (1% AEP LORR).

5.2.5.3.2.1 **Sector Gate Wall**

Sector gate walls were designed as a cantilever beam extending from the base slab. A constant wall thickness was assumed the full height of the wall. A typical wall was designed for each sector gate. No pro-rating of wall thickness was performed. The resulting calculated wall thicknesses are summarized in Table 132.

Table 132 - Wall Thickness

Structure	Wall Thickness (ft) (3% AEP)	Wall Thickness (ft) (1% AEP)
Bayou Black	3.25	4.50
Shell Canal East	3.25	4.50
Minors Canal	3.00	4.50
Falgout Canal	3.00	4.50
Bayou DuLarge	3.00	4.50
Bayou Grand Caillou	3.50	5.00
Bayou Petite Caillou	3.50	6.00
Placid Canal	4.00	6.00
Bush Canal	4.75	7.00
Bayou Terrebonne	4.25	6.50
Humble Canal	4.25	6.50
Pointe Aux Chenes	4.00	6.00
Grand Bayou	3.50	6.00
Larose	3.00	3.50

5.2.5.3.2.2 **Sector Gate Thrust Block and Machinery Block**

Sector gate thrust and machinery blocks were not designed because of their relatively small quantity compared to that of the remainder of the walls. Historical data from previously constructed sector gates was utilized to size the thrust and machinery blocks.

5.2.5.3.2.3 **Sector Gate Base Slab**

The 56' sector gate base slab will be 134 ft long by 78 ft wide. The sector gate base slab thickness was determined utilizing 2D transverse and longitudinal strips. The transverse strip was taken beneath the thrust block while the longitudinal strip was taken beneath the machine and thrust blocks. The strips were designed as solid beams, given the property of the width of the slab that was examined. All loads acting along the width of the beams were input into STAAD (Structural Analysis and Design) and resolved along the centroid of the beam. Piles were modeled as pinned supports. The 2D strips were analyzed for the Falgout Canal and Bush Canal sector gates only. The remaining structures' base slab thickness was pro-rated based on the analysis performed for Falgout Canal and Bush Canal. The base slab thicknesses are summarized in Table 133.

Table 133 - Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
Bayou Black	6.50	8.50
Shell Canal East	6.50	9.00
Minors Canal	6.00	8.00
Falgout Canal	6.00	8.00
Bayou DuLarge	6.00	8.00
Bayou Grand Caillou	7.00	9.50
Bayou Petite Caillou	7.00	10.00
Placid Canal	7.50	10.50
Bush Canal	8.50	12.00
Bayou Terrebonne	8.00	11.00
Humble Canal	8.00	12.00
Pointe Aux Chenes	7.50	10.00
Grand Bayou	7.00	10.00
Larose	6.50	7.50

5.2.5.3.3 Sector Gate Pile Foundation

5.2.5.3.3.1 **General**

The pile foundation for the 3% AEP LORR sector gates will include 138 HP 14X73 piles battered on 3 vertical to 1 horizontal slope while the pile foundation for the 1% AEP LORR sector gates will include 193 HP 14X73 piles on a similar batter. The design Factors of Safety utilized for the design comply with EM 1110-2-2906 and the latest

requirements in the HSDRRS design guidelines. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks are provided on all piles on the flood side of the sheet pile cutoff- wall to handle the maximum tensile load. CPGA analysis was performed for each sector gate. No pro-rating was performed. Details for the pile foundation are shown on Plate S-005 (3% AEP LORR) and S-005A (1% AEP LORR). Alternative pile types and arrangements will be investigated during detailed design for each structure to optimize the pile foundation.

5.2.5.3.3.2 **CPGA Analysis**

CPGA was utilized to develop the pile layouts for the gate structures and determine the required tip elevation. The piles were modeled as pinned connections with the piles providing all of the lateral resistance. The horizontal subgrade modulus was based on the soil in the top ten pile diameters. The horizontal subgrade modulus was reduced for group effects in accordance with EM 1110-2-2906.

5.2.5.3.3.3 **Pile Curves and Horizontal Subgrade Modulus**

Pile curves and horizontal subgrade modulus were calculated for a limited number of structures. Existing pile curves were utilized for those structures where no pile curve was calculated. A summary of the pile curve used for each structure is summarized below-

56' Gate Structure		Pile Curve Utilized
Bayou Grand Caillou	→	*Bayou Grand Caillou*
Bush Canal	→	*Bush Canal*
Bayou Terrebonne	→	*Bayou Terrebonne*
Humble Canal	→	*Humble Canal*
Bayou Pointe Aux Chenes	→	*Bayou Pointe Aux Chenes*
Grand Bayou	→	*Grand Bayou*
Bayou Black	→	*Box Culverts for Barrier Alignment*
Shell Canal East	→	*Box Culverts for Barrier Alignment*
Minors Canal	→	*GIWW 175' Sector Gate West*
Falgout Canal	→	*Bayou Grand Caillou*
Bayou Dularge	→	*Bayou Grand Caillou*
Bayou Petite Caillou	→	*Lapeyrouse Canal*
Placid Canal	→	*Bayou Terrebonne*
Larose FG	→	*Larose to Golden Meadow Feasibility Study*

The resulting pile tips are summarized in Table 134.

Table 134 - Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
Bayou Black	-145.5	-111.0
Shell Canal East	-145.5	-111.0
Minors Canal	-141.0	-124.0
Falgout Canal	-126.5	-116.0
Bayou DuLarge	-124.5	-116.0
Bayou Grand Caillou	-135.5	-128.5
Bayou Petite Caillou	-157.0	-157.0
Placid Canal	-160.0	-160.0
Bush Canal	-158.0	-124.0
Bayou Terrebonne	-167.0	-167.0
Humble Canal	-125.5	-129.5
Pointe Aux Chenes	-137.0	-137.0
Grand Bayou	-151.0	-151.0
Larose	-145.5	-126.0

5.2.5.3.3.4 **Cut-off Wall**

A cut-off sheetpile wall will be provided to reduce possible seepage, scouring and uplift. A PZC-13 sheetpile meeting the requirements of ASTM A572, Grade 50 was assumed for the cutoff wall. Tip elevations were provided by New Orleans District Engineering

Division Geotechnical Branch utilizing Lane's Weighted Creep Ratio for each structure. Details and tips for the cut-off wall are shown on Plate S-005 (3% AEP LORR) and S-005A (1% AEP LORR).

5.2.5.3.4 Steel Sector Gate

5.2.5.3.4.1 General

The structural design of the sector gates was performed in accordance with Corps engineering guidance and applicable industry standards. The Corps criteria are specified in EM1110-2-2105 and EM 1110-2-2703. The sector gates will consist of structural wide-flange sections supporting the vertical ribs and skin plate with a central angle of 60 degrees. All connections will be welded connections. Gates were designed for Falgout Canal and Bush Canal sector gates for the 3% AEP LORR and for Bayou Grand Caillou and Bush Canal for the 1% AEP LORR. The remaining gates were pro-rated based on the gates designed. A rack and pinion gear system will operate the gate. All steel members on the gate will be painted with a coal tar epoxy paint system. Details of the steel sector gate are shown on Plates S-006 to S-016 (3% AEP LORR) and S-006A to S-016A (1% AEP LORR).

5.2.5.3.4.2 Skin Plate

The skin plate was designed conservatively as a simply supported member by vertical angles, spaced 2' on center. An allowable stress of 0.50 times the yield stress was permitted for basic loading conditions with a permissible increase of one-third for abnormal loading conditions. EM 1110-2-2703 requires that the skin plate be designed with a 1/16" reduction in thickness.

5.2.5.3.4.3 Vertical Ribs

The skin plate will be attached to the vertical ribs by continuous welds. The ribs were designed as simply supported members between the horizontal wide flanges. The skin plate was considered as an effective part of the vertical ribs, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. A minimum depth of ribs is required to be 8 in. to facilitate painting and maintenance. The ribs will be constructed from material conforming to ASTM A-588 Grade 50 steel.

5.2.5.3.4.4 Horizontal Beams

The gate leaf consists of horizontal beams supporting the vertical ribs and skin plate. The beam was designed as a continuous member supported by the horizontal struts and braces at midpoint between the struts. The curve of the beam was neglected, with

the length used for design equal to the arc length along the center line of the beam. The beams will be constructed from material conforming to ASTM A572 Grade 50 steel. The dead weight applied to the girders included, where applicable, the walkway weight, the weight of the intercostals and ribs, and the self-weight of the girder.

5.2.5.3.4.5 3D Modeling of Gate

The trusses and frames were analyzed as a three-dimensional space frame in STAAD Pro 2006. The chords of the trusses were analyzed as fixed members while the minor members of the trusses along with the members of the frames were analyzed as pinned connections.

The hinge was modeled to resist forces in the horizontal plane (F_x, F_y) while the pintle was modeled as a pinned connection to resist forces in both the horizontal and vertical planes (F_x, F_y, F_z). For gate open cases with boat impact, a roller support was added at the location of the gate stop to stabilize the structure during boat impacts while in the gate closed cases with boat impact, a roller support was added to the machinery to resist boat impacts and stabilize the structure. The vertical dead load was carried only by the pintle.

5.2.5.3.4.6 Hinge and Pintle

The gate frames will be supported at the top by a hinge and at the bottom by a pintle. In order to assure good pintle and hinge alignment, a spherical pin will be provided in the hinge to compliment the spherical pintle. All vertical loads will be transferred to the concrete base through the pintle. Horizontal reactions will be transferred to the thrust block through bronze bushings. Bearing pressures on the bushings were limited to 2500 psi for operating conditions and 5000psi for maintenance conditions. The hinge and pintle were designed for the Bush Canal sector gate for the 3% AEP LORR and 1% AEP LORR. The hinge and pintle for the remaining gates were pro-rated based on the Bush Canal sector gate design.

5.2.5.3.4.7 Fender

A fendering system was provided on the channel side truss of the sector gate to protect the truss from a barge impact of 125 kips. This load corresponds to the load recommended in EM 1110-2-2703 "Lock Gates and Operating Equipment" for sector gates. The entire fendering system will be removable to permit maintenance and painting of the gate as needed.

The impact load was assumed to be distributed evenly between two 8 in x 12 in

composite marine timbers, supported on 5 ft centers by vertical W 10 x 77 sections. The composite marine timbers were designed as continuous members supported by the vertical members. The vertical members were designed as simply supported members between the horizontal members. Two large horizontal W24X104 sections, which are bolted on at panel points on the channel side truss of the gate, transfer the impact load back to the channel side truss and were designed as continuous members.

5.2.5.3.4.8 Walkway

The walkway will extend around the trusses of the gate leaf as well as along the skin plate. A 4 ft walkway width will be provided, designed for an imposed live load of 200 psf. Aluminum grating with 1-1/4 in by 3/16 in bearing bars was selected to span the 4 ft required width of walkway. Aluminum handrails will be provided along the entire walkway to resist a force of 200 lb applied at the top rail in accordance with EM 385-1-1.

5.2.5.3.5 Needle Girders, Needles and Supports

The needle girder system arrangement was designed to dewater the entire gatebay to permit maintenance of the sector gates. The needle girder system was designed for a sill elevation of -12.0 with a water elevation of +5.0. Twelve steel needles will be provided (6 on each side of the structure), measuring 9'-3" in width, used to dewater the concrete gatebay monoliths. The steel needles consist of vertical WT 8X33.5 members with a 3/8" skin plate. The needles are supported by the sill of the concrete gatebay and the needle girder at El 5.0. The needle girder was designed as a simply supported, built-up girder, spanning across the 56' gate opening. The girder will be supported along its weak axis by 2 support towers. The girder at mid-span will have a depth of 4'-6" with 5/8" web and 1"x15" flanges. The girder will taper down to a depth of 3'-2" at the ends. The support towers will consist of welded HSS connections, supporting the dead and vertical live loads of the needle girder. Details of the needle girder, needles and support are shown on Plates S-018 to S-019. Only Grand Bayou, Bush and Bayou Grand Caillou will have a needle girder system built as part of those gates' construction. The remainder of the gates will utilize those needle girder systems for their dewaterings.

5.2.5.3.6 Needle Girder Storage Platform

The needle girder storage platform will be a reinforced concrete structure measuring 20 ft wide by 171 ft long. It is assumed that each 56' gate structure will have a needle girder storage platform. The 120' long longitudinal beams, measuring 2 ft wide by 1'-9" deep, were designed as continuous beams with the dead load of the needle girder system, a 100psf live load and the self-weight of the beam. The 9 transverse beams, measuring 1'-4" wide by 1'-9" deep, were analyzed as compression members bracing

the longitudinal beams. 10% of the vertical load applied to the longitudinal beams was applied axially to the transverse beams. The storage platform will be supported by 18, 14" square precast pre-stressed concrete (PPC) piles, 80' long. Details of the needle girder storage platform are shown on Plate S-020.

5.2.5.3.7 Guidewalls and Pile Clusters

Guidewalls and pile clusters will be provided as aids to navigation and to protect the main flood gate structure from impact. Details were taken from historical 56' sector gate structures constructed in New Orleans District rather than performing actual design on this component. The wall lengths and details on the walls and pile clusters are shown on Plates S-021 to S-023.

5.2.5.3.8 Control Houses

A precast 14'x14' concrete control house will be provided for each gate leaf to shelter the gate control systems and machinery and provide space for a gate operator as required. The buildings are considered small and were not designed, so historical dimensions were used for cost estimation purposes. It is assumed that these buildings will be pre-fabricated during construction.

5.2.5.3.9 Sluice Gate Monolith Concrete

Details of the concrete monolith are shown on Plates S-054 to S-057 (3% AEP) and S-054A to S-057A (1% AEP).

5.2.5.3.9.1   Sluice Gate Walls

The breast walls were designed for the hydrostatic pressure differential above the sluice gates, fixed between the pier walls. A portion of the load from the hydraulic cylinder and walkway were also placed on the breast wall, but were not examined because the breast wall functions as a very deep beam with a large capacity in the plane that the hydraulic cylinder load is applied. A 3' thick breast wall was used for all sluice gate structures.

The operating platform beams were designed for the dead and live loads imparted by the sluice gate and its machinery as well the dead and live loads from the operating platform. The beam was designed as fixed between the pier wall supports. A 2' wide by 2'-6" deep operating platform beam was used for all sluice gate structures.
The pier walls were designed as a wall with the combined axial load and moment imparted by the breast walls, operating platform beam, and lateral load from a dewatered condition. The pier wall design section was set equal to the thickness of the

pier wall with loads extended through the depth of the wall at a 45 degree angle. An interaction diagram was setup to verify that the ultimate moment and compression were within the allowable limits. The pier wall was assumed as a cantilever to determine the moments associated with the dewatering loads. The live and dead load moments from the breast walls and operating platform beam were taken assuming those beams are fixed at the pier walls. A 3' thick pier wall was used for all sluice gate structures

5.2.5.3.9.2   Sluice Gate Base Slab

The sluice gate base slab thickness was determined utilizing a 2D strip with a width equal to the width of one sluice gate bay. The strips were designed as solid beams, given the property of the width of the slab that was examined. All loads acting along the width of the beams were input into STAAD (Structural Analysis and Design) and resolved along the centroid of the beam. Piles were modeled as pinned supports. The 2D strips were analyzed for the Bayou Grand Caillou and Bush Canal sluice gates only. The remaining structures' base slab thicknesses were selected based on similar head differentials as the sluice gate structures that were analyzed. The base slab thicknesses are summarized in Table 135.

Table 135 - Sluice Gate Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
Bayou Grand Caillou	5	8
Bayou Petite Caillou	7	10
Placid Canal	7	10
Bush Canal	7	10
Falgout Canal	5	8
Grand Bayou	7	10

5.2.5.3.10   Sluice Gate Pile Foundation

5.2.5.3.10.1   General

The pile foundation for the sluice gates will include HP 14X73 piles battered on 3 vertical to 1 horizontal. The design Factors of Safety utilized for the design comply with EM 1110-2-2906 and the latest requirements Hurricane and Storm Damage Risk Reduction System Design Guidelines. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks will be provided on all piles on the flood side of the sheet pile cutoff- wall to handle the maximum tensile load. CPGA

analysis was performed for each sluice gate. No pro-rating was performed. Details for the pile foundation are shown on Plate S-058 (3% AEP LORR) and S-058A (1% AEP LORR). Alternative pile types and arrangements will be investigated during detailed design for each structure to optimize the pile foundation.

5.2.5.3.10.2 **CPGA Analysis**

CPGA was utilized to develop the pile layouts for the sluice gate structures and determine the required tip elevation. The piles were modeled as pinned connections with the piles providing all of the lateral resistance. The horizontal subgrade modulus was based on the soil in the top ten pile diameters. The horizontal subgrade modulus was reduced for group effects in accordance with EM 1110-2-2906.

5.2.5.3.10.3 **Pile Curves and Horizontal Subgrade Modulus**

Pile curves and horizontal subgrade modulus were calculated for a limited number of structures. Existing pile curves were utilized for those structures where no pile curve was calculated. A summary of the pile curve used for each structure is summarized below:

Sluice Gate Structure   Pile Curve Utilized
Bayou Grand Caillou   →   *Bayou Grand Caillou*
Bush Canal   →   *Bush Canal*
Bayou Terrebonne   →   *Bayou Terrebonne*
Grand Bayou   →   *Grand Bayou*
Bayou Petite Caillou   →   *Lapeyrouse Canal*
Placid Canal   →   *Bayou Terrebonne*

The resulting pile tips are summarized in Table 136.

Table 136 - Sluice Gate Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
Bayou Grand Caillou	-124.5	-124.5
Bayou Petite Caillou	-157.0	-157.0
Placid Canal	-155.0	-155.0
Bush Canal	-130.0	-130.0
Falgout Canal	-113.0	-113.0
Grand Bayou	-140.0	-140.0

5.2.5.3.10.4   **Cut-off Wall**

A cut-off sheetpile wall will be provided to reduce possible seepage, scouring and uplift. A PZC-13 sheetpile meeting the requirements of ASTM A572, Grade 50 was assumed for the cutoff wall. Tip elevations were calculated utilizing Lane's Weighted Creep Ratio for each structure. Details and tips for the cut-off wall are shown on Plate S-058 (3% AEP LORR) and S-058A (1% AEP LORR).

5.2.5.3.11   Sluice Gates

The sluice gates will be manufactured 16'x16' or 16'x12' cast iron gates.

5.2.5.3.12   Sluice Gate Bulkheads

The sluice gate bulkheads are designed to dewater the entire gatebay to permit maintenance of the sluice gates and concrete gatebay. The bulkheads were designed for a sill elevation of -12.0 with a water elevation of +5.0. Each sluice gate structure will be provided with 4 bulkheads, permitting the dewatering of 2 sluice gate bays at a time.

The steel bulkheads consist of horizontal L8X4X1/2   members with a 3/8" skin plate. The skin plate was designed conservatively as a simply supported member between the horizontal angles. The horizontal angles were designed as simply supported members between the sluice gates walls. The skin plate was considered as an effective part of the horizontal angles, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. All steel will be constructed from material conforming to ASTM A-572 Grade 50. Details of the sluice gate bulkheads are shown on Plate S-059.

5.2.5.3.13   Tie-in T-Walls

Tie-in T-Walls extend from the sector gate/sluice gate structures to the full levee section. The distance from the gate structure to the full levee section was calculated. T-Walls were designed in accordance with the latest requirements of the HSDRRS design guidelines. A 30' sheetpile cutoff will be embedded into the levee at the transition between the tie-in T-Walls and the levee section. Nine inches of reinforced concrete scour protection will be provided at the transition area. A pre-load 2'-0" above the final grade along the T-Walls will be provided to eliminate settlement induced bending effects.

T-Walls were broken into different types according to hydraulic reach and base elevation and typical designs were performed. The required pile tip was determined individually for each structure based on the pile capacity demand from the typical

designs. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks are provided on all piles on the flood side of the sheet pile cutoff-wall to handle the maximum tensile load. Details of the tie-in T-Walls can be found on Plates S-092 to S-099 (3% AEP LORR) and S-092A to S-099A (1% AEP LORR).

5.2.5.4 Electrical Design

5.2.5.4.1 Electrical Service

The Electrical service to the structure will be 480/277 volt, 3 phase, 4 wire grounded secondary service from the local utility company. The service will be sized to support the structure loads including power for Gate machinery, lighting, controls, and any other miscellaneous loads.

5.2.5.4.2 Emergency Generator

A diesel generator will be provided for back-up power in the case of loss of utility power. The fuel supply for the generator will be provided from a fuel tank to a skid mounted UL-Listed double-walled day tank. Alarms will be locally annunciated on the generator.

5.2.5.4.3 Grounding System

The structure grounding system will be in accordance with the NFPA 70 - National Electrical Code. The grounding system will consist of copper ground rods interconnected with copper conductors. All jumpers and grounding electrode conductor connections will be done by exothermic weld. All electrical equipment, machinery, and exposed metal will be bonded to the grounding electrode system.

5.2.5.4.4 Lighting System

All exterior lighting fixtures will be provided with vandal-proof shields. The fixtures will be HPS and shall be controlled by photocells. Fluorescent light fixtures will be provided in the control houses.

5.2.5.4.5 Conduit and Boxes

All wiring will be installed in rigid metal conduit except that motors and other electrical equipment subject to vibration will be connected with liquid-tight flexible metal conduit. All pull boxes and junction boxes will be of cast metal of sufficient thickness or provided with bosses to accommodate the required threads for the conduit connections of size specified. All outlet boxes for receptacles, switches, and lighting fixtures will be of cast metal with bosses drilled and tapped or with threaded hubs of sizes specified. The

edges will be designed to take a heavy cover gasket with four or more screws for attaching covers or fixtures.

5.2.5.4.6 Controls

A hard wired control system will be installed to operate the Gates. The control consoles will be installed in the control houses. Additionally, local controls will be provided at each sluice gate.

5.2.5.4.7 Lightning Protection System

A lightning protection system will be designed to protect the structure from lightning strikes. The system will be designed in accordance with NFPA 780-Installation of Lightning Protection Systems. Surge suppression devices on all incoming power and communication lines will be provided.

5.2.5.5   Mechanical Design

5.2.5.5.1 Gate Operation

Gate operation will be two speeds with a time dependent 1 to 4 second speed ramp at start, stop and speed changes. The dual speed and speed ramp will be accomplished electronically by way of a hydraulic proportional valve. A slow gate speed of 3.5 degrees per minute will be used near the end of gate travel, (1 to 3 feet from fully close or fully open, measured at the skin plate). A higher speed of 30 degrees per minute will be used in between the ends of travel.

5.2.5.5.2 Gate Operating Loads

The gate operating loads consist of friction from hinge, pintle and seal and hydrodynamic loads. The hydrodynamic loads were based on differential hydrostatic head applied over the gate end beams. Three load cases were considered; a balanced head case, a direct head case and a reverse head case.

5.2.5.5.3 Gate Operating Machinery

The gate operating machinery will be a rack and pinion gear drive. The rack will be attached to the gate along the outside radius of the gate's skin plate. A pinion drive gear will be attached to a low speed high torque hydraulic (LSHT) motor mounted on the wall. A Hagglunds Viking Series 64 LSHT hydraulic motor operating at 2000 psi was used for design purposes. Each gate will be equipped with its own hydraulic power unit (HPU). The HPU will include a variable delivery pressure compensated pump

driven by an electric motor. The electric motor will be 10 horsepower. Additional HPU items will include valves, manifold, gauges, filters, clean vent, and storage tank. The gate operating machinery is shown on Plate M-001.

5.2.5.5.4 Hinge and Pintle Bushing Material

Hinge and pintle bushings will be split in the vertical plane. Hinge and pintle bushings will be a greaseless/self-lubricating system with an approved composite. The material will have a dynamic coefficient of friction that is less than or equal to 0.08 dry and 0.10 water-lubricated for a bearing pressure load of 2 ksi at surface speed 90 fpm. The ultimate compressive strength of the material will be a minimum of 50 ksi and its water absorption will be less than 0.10 percent by weight. Bushing material and dowels will meet the requirements of ASTM B 148, Alloy C95500, ASTM B 271, Alloy C95500, or ASTM A705, Type 630, minimum hardness 40 Rc.

5.2.5.5.5 Sluice Gate Operation

Sluice gate operation will be to raise, lower, or hold in intermediate positions to allow, prevent, or meter drainage water flow and to prevent backflow during storm conditions. Operating time was designed to operate in less than fifteen minutes to totally raise or lower the sluice gate.

5.2.5.5.6 Sluice Gate Operating Loads

The sluice gate operating loads consist of friction from the stem, sluice gate weight, stem weight, hydrodynamic loads. The hydrodynamic loads were developed from differential hydrostatic head applied over the sluice gate.

5.2.5.5.7 Sluice Gate Operating Machinery

The sluice gate operating machinery will be an operating stem, bevel gearbox, and electric actuator. The operating stem will be attached to the top of the sluice gate within the stem pocket. The operating stem will be machine threaded AISI 316 stainless steel. The aluminum bronze ASTM B505 lift nut within the bevel gearbox will be machine threaded to mate with the AISI 316 stainless steel operating stem. The electric actuator will mount onto the bevel gearbox used to rotate the operating stem to raise or lower the sluice gate. The bevel gearbox used for design purposes for the 16'x16' sluice gates was a Diamond Gear 105BG7. The bevel gearbox used for design purposes for the 16'x12' sluice gates was a Diamond Gear 57BG6. The electric actuator considered for the 16'x16' sluice gates was a 30 horsepower Biffi Icon 2000 050/1440-173. The electric actuator considered for the 16'x12' sluice gates was a 10 horsepower Biffi Icon 2000 040/720-173. The sluice gate operating machinery is shown on Plate S-055.

5.2.6 125' Sector Gate

This section contains a summary of work for the two, 125' sector gate structures, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 137 lists the structures examined-

Table 137 - 125' Sector Gate Structures

Structure	Sill Elevation	Top Elevation (3% AEP)	Top Elevation (1% AEP)	Top of Guidewalls
GIWW West	-16	16	23	10
GIWW East	-16	17	25	10

5.2.6.1 Physical Features

The physical features associated with the construction of the 125 ft sector gate structures are-

- Temporary Bypass Channels
- Phase 1 Cellular Cofferdam
- Phase 2 Interior Braced Cofferdams
- Sector Gate Concrete Monolith
- Sector Gate Pile Foundation
- Steel Sector Gate
- Needle Girder, Needles and Supports
- Needle Girder Storage Platform
- Guidewalls
- End Cell Dolphins
- Sluice Gate Concrete Monolith
- Sluice Gate Pile Foundation
- Sluice Gates
- Sluice Gate Bulkheads
- Tie-in T-Walls
- Electrical Controls and Circuitry
- Mechanical Equipment

5.2.6.2 Construction Sequencing

Both 125' sector gates will be constructed in the approximate center of the existing channels. A minimum 125' temporary bypass channel will be constructed as the first

order of construction, to allow navigation passage during construction. Once navigation is routed through the temporary bypass channel, a cellular cofferdam will be constructed, permitting the construction of the 125' sector gate monolith and the sluice gate monoliths. Reduced power will be required for vessels passing through the construction area to reduce the risk of impact to the cofferdam. A timber guidewall and pile clusters will be provided along the bypass channel to minimize potential vessel impact on the cofferdam. Once construction of the 125' sector gate monolith and sluice gate monoliths is completed, navigation will be re-routed through the permanent sector gate structure. A phase 2 cofferdam will be required for the T-Walls adjacent to the sector gate/sluice gate structures. Once navigation is re-routed, the phase 2 cofferdam, needle girder storage platform, permanent guidewalls, end cell dolphins, tie-in T-Walls and final civil site work can be completed.

5.2.6.3 Structural Design

5.2.6.3.1 Phase 1 Cellular Cofferdam

A Phase 1 cellular cofferdam will be constructed to permit the in the dry construction of the sector gate concrete monolith and the sluice gate concrete monolith. The cofferdam will be a sheet pile cellular cofferdam in-filled with sand. Deep soil mixing will be necessary in the interior of the cellular structure to provide adequate geotechnical safety factors. Details of the Phase 1 cellular cofferdam can be found on Plate S-050.

ER 1110-2-8152 will be followed throughout the project design process, requiring that all cofferdams will be designed by the Government.

5.2.6.3.2 Phase 2 Interior Braced Cofferdams

A phase 2 cofferdam will be constructed to permit the construction of the adjacent T-Walls to the sector gate/sluice gate structures that will be in the water. The anchor forces, moments, and tips used for the Phase 1 Bayou Dularge sector gate phase 1 cofferdams will be conservatively used for the Phase 2 cofferdams. Details of the Phase 2 cofferdam can be found on Plate S-100.

ER 1110-2-8152 will be followed throughout the project design process, requiring that all cofferdams will be designed by the Government.

5.2.6.3.3 Sector Gate Monolith Concrete

Details of the concrete monolith are shown on Plates S-026 to S-027 (3% AEP LORR) and S-026A to S-027A (1% AEP LORR).

5.2.6.3.3.1   **Sector Gate Wall**

Sector gate walls were designed as a cantilever beam extending from the base slab. A constant wall thickness was assumed the full height of the wall. A typical wall was designed for each sector gate. No pro-rating of wall thickness was performed. The resulting calculated wall thicknesses are shown in Table 138.

Table 138 - 125' Sector Gate Wall Thickness

Structure	Wall Thickness (ft) (3% AEP)	Wall Thickness (ft) (1% AEP)
GIWW West	4	4.75
GIWW East	4	4.75

5.2.6.3.3.2   **Sector Gate Thrust Block and Machinery Block**

Sector gate thrust and machinery blocks were not designed because of their relatively small quantity compared to that of the remainder of the walls. Historical data from previously constructed sector gates was utilized to size the thrust and machinery blocks.

5.2.6.3.3.3   **Sector Gate Base Slab**

The 125' sector gate base slab will measure 310'-6" long by 117'-8" wide. The sector gate base slab thickness was determined utilizing 2D transverse and longitudinal strips. The transverse strip was taken beneath the thrust block while the longitudinal strip was taken beneath the machine and thrust blocks. The strips were designed as solid beams, given the property of the width of the slab that was examined. All loads acting along the width of the beams were input into STAAD (Structural Analysis and Design) and resolved along the centroid of the beam. Piles were modeled as pinned supports. The 2D strips were analyzed for the GIWW West sector gate only. It was assumed that GIWW East would have the same slab thickness. The base slab thicknesses are summarized in Table 139.

Table 139 - 125' Sector Gate Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
GIWW West	10	12
GIWW East	10	12

5.2.6.3.4 Sector Gate Pile Foundation

5.2.6.3.4.1   General

The pile foundation for the 3% AEP LORR sector gates will include 246 twenty-four inch pipe piles with ½" thick wall thickness battered on 4 vertical to 1 horizontal slope while the pile foundation for the 1% AEP LORR sector gates will include 300 twenty-four inch pipe piles with ½" thick wall thickness on a similar batter. The design Factors of Safety utilized for the design comply with EM 1110-2-2906 and the latest requirements Hurricane and Storm Damage Risk Reduction System Design Guidelines. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks are provided on all piles. CPGA analysis was performed for each sector gate. No pro-rating was performed. Details for the pile foundation are shown on Plate S-029 (3% AEP LORR) and S-029A (1% AEP LORR). Alternative pile types and arrangements will be investigated during detailed design for each structure to optimize the pile foundation.

5.2.6.3.4.2   CPGA Analysis

CPGA was utilized to develop the pile layouts for the gate structures and determine the required tip elevation. The piles were modeled as pinned connections with the piles providing all of the lateral resistance. The horizontal subgrade modulus was based on the soil in the top ten pile diameters. The horizontal subgrade modulus was reduced for group effects in accordance with EM 1110-2-2906.

5.2.6.3.4.3   Pile Curves and Horizontal Subgrade Modulus

Pile curves and horizontal subgrade modulus were calculated for the two sector gates. The resulting pile tips are summarized in Table 140.

Table 140 - 125' Sector Gate Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
GIWW West	-160.0	-160.0
GIWW East	-161.50	-161.50

5.2.6.3.4.4   Cut-off Wall

A cut-off sheetpile wall will be provided to reduce possible seepage, scouring and uplift. A PZC-13 sheetpile meeting the requirements of ASTM A572, Grade 50 was assumed for the cutoff wall. Tip elevations were calculated utilizing Lane's Weighted Creep Ratio for each structure. Details and tips for the cut-off wall are shown on Plate S-029 (3% AEP LORR) and S-029A (1% AEP LORR).

5.2.6.3.5 Steel Sector Gate

5.2.6.3.5.1 **General**

The structural design of the sector gates was performed in accordance with Corps engineering guidance and applicable industry standards. The Corps criterion is specified in EM1110-2-2105 and EM 1110-2-2703. The sector gates will consist of structural pipe sections supporting the vertical ribs and skin plate with a central angle of 70. All connections will be welded connections. The GIWW West gate was designed and the GIWW East gate quantities were pro-rated based on that design. A rack and pinion gear system will operate the gate. All steel members on the gate will be painted with a coal tar epoxy paint system. Details of the steel sector gate are shown on Plates S-030 to S-041 (3% AEP) and S-030A to S-041A (1% AEP).

5.2.6.3.5.2 **Skin Plate**

The skin plate was designed conservatively as a simply supported member by vertical angles, spaced 2' on center. An allowable stress of 0.50 times the yield stress was permitted for basic loading conditions with a permissible increase of one-third for abnormal loading conditions. EM 1110-2-2703 requires that the skin plate be designed with a 1/16" reduction in thickness.

5.2.6.3.5.3 **Vertical Ribs**

The skin plate will be attached to the vertical ribs by continuous welds. The ribs were designed as simply supported members between the horizontal built-up plate girders. The skin plate was considered as an effective part of the vertical ribs, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. A minimum depth of ribs will be required to be 8 in. to facilitate painting and maintenance. The ribs are also constructed from material conforming to ASTM A-588 Grade 50 steel.

5.2.6.3.5.4 **Horizontal Beams**

The gate leaf consists of horizontal beams supporting the vertical ribs and skin plate. The beam will be designed as a continuous member supported by the horizontal struts and braces at midpoint between the struts. The curve of the beam will be neglected, with the length used for design equal to the arc length along the center line of the beam. The beams are constructed from material conforming to ASTM A-572 Grade 50 steel. The dead weight applied to the girders included, where applicable, the walkway weight, the weight of the intercostals and ribs, and the self-weight of the girder.

5.2.6.3.5.5   **3D Modeling of Gate**

The trusses and frames were analyzed as a three-dimensional space frame in STAAD Pro 2006. The chords of the trusses were analyzed as fixed members while the minor members of the trusses along with the members of the frames were analyzed as pinned connections.

The hinge was modeled to resist forces in the horizontal plane (F_x, F_y) while the pintle was modeled as a pinned connection to resist forces in both the horizontal and vertical planes (F_x, F_y, F_z). For gate open cases with boat impact, a roller support was added at the location of the gate stop to stabilize the structure during boat impacts while in the gate closed cases with boat impact, a roller support was added to the machinery to resist boat impacts and stabilize the structure.   The vertical dead load was carried only by the pintle.

5.2.6.3.5.6   **Hinge and Pintle**

The gate frames will be supported at the top by a hinge and at the bottom by a pintle. In order to assure good pintle and hinge alignment, a spherical pin will be provided in the hinge to compliment the spherical pintle. All vertical loads will be transferred to the concrete base through the pintle. Horizontal reactions will be transferred to the thrust block through bronze bushings. Bearing pressures on the bushings were limited to 2500 psi for operating conditions and 5000psi for maintenance conditions.   The hinge and pintle were designed for the GIWW East sector gate for the 3% AEP LORR and 1% AEP LORR at a 175' opening. The hinge and pintle for the GIWW East and West gates at a 125' opening were pro-rated based on the GIWW East sector gate design at a 175' opening.

5.2.6.3.5.7   **Fender**

A fendering system was provided on the channel side truss of the sector gate to protect the truss from a barge impact of 125 kips. This load corresponds to the load recommended in EM 1110-2-2703 "Lock Gates and Operating Equipment" for sector gates. The entire fendering system will be removable to permit maintenance and painting of the gate as needed.

The impact load was assumed to be distributed evenly between two 8 in x 12 in composite marine timbers, supported on 5 ft centers by vertical W 10 x 77 sections. The composite marine timbers were designed as continuous members supported by the vertical members. The vertical members were designed as simply supported members between the horizontal members. Two large horizontal W27X146 sections, which are

bolted on at panel points on the channel side truss of the gate, transfer the impact load back to the channel side truss and were designed as continuous members.

5.2.6.3.5.8 Walkway

The walkway extends around the trusses of the gate leaf as well as along the skin plate. A 4'-6" walkway width was provided, designed for an imposed live load of 200 psf. Aluminum grating with 1 1/4 in by 3/16 in bearing bars was selected to span the 4 ft required width of walkway. Aluminum handrails are provided along the entire walkway to resist a force of 200 lb applied at the top rail in accordance with EM 385-1-1.

5.2.6.3.6 Needle Girders, Needles and Supports

The needle girder system arrangement was designed to dewater the entire gatebay to permit maintenance of the sector gates. The needle girder system was designed for a sill elevation of -16.0 with a water elevation of +5.0. Each gate structure will be provided with 24 steel needles (12 on each side of the structure), measuring 14'-6" in width, used to dewater the concrete gatebay monoliths. The steel needles will consist of vertical WT 8X38.5 members with a 7/16" skin plate. The needles will be supported by the sill of the concrete gatebay and the needle girder at El 5.0. The needle girder was designed as a simply supported, built-up girder, spanning across the 125' gate opening. The girder will be supported along its weak axis by 3 support towers. The girder at mid-span has a depth of 8'-4" with 3/4" web and 2"x20" flanges. The girder will taper down to a depth of 5'-4" at the ends. The support towers will consist of welded HSS connections, supporting the dead and vertical live loads of the needle girder. Details of the needle girder, needles and support are shown on Plates S-043 to S-044.

5.2.6.3.7 Needle Girder Storage Platform

The needle girder storage platform will be a reinforced concrete structure measuring 71 ft wide by 135 ft long. The structure will consist of an 8" cast in-place slab supported by 40" wide by 30" deep cast in-place beams, spaced 9' O.C. The storage platform will be supported by 60 twenty-four inch square precast pre-stressed concrete (PPC) piles, 80' long. Details of the needle girder storage platform are shown on Plates S-045 to S-046.

5.2.6.3.8 Guidewalls

Guidewalls will be provided as aids to navigation and to protect the main flood gate structure from impact. Details were taken from the HNC Lock structure as both structures will see similar vessel traffic. The wall lengths and details on the walls are shown on Plates S-047 to S-048.

5.2.6.3.9 End Cell Dolphins

End Cell Dolphins will protect the main flood gate structure and guidewalls from head-on impact from errant vessels. The end cell design was taken from the Western Closure Complex 225' Sector Gate, where similar vessel traffic is seen along the GIWW. The end cell will consist of a 60' sheet pile cellular structure with a concrete ring in the interior or the cell. The inside of the concrete ring will be in-filled with lightweight fill material. The concrete structure will be supported by 18" diameter pipe piles. Details are shown on Plate S-049.

5.2.6.3.10   Control Houses

A precast 14'x14' concrete control house will be provided for each gate leaf to shelter the gate control systems and machinery and provide space for a gate operator as required. The buildings are considered small and were not designed, so historical dimensions were used for cost estimation purposes. It is assumed that these buildings will be pre-fabricated during construction.

5.2.6.3.11   Sluice Gate Monolith Concrete

See Section 5.2.5.3.9

5.2.6.3.11.1 Sluice Gate Walls

See Section 5.2.5.3.9.1

5.2.6.3.11.2 Sluice Gate Base Slab

The sluice gate base slab thickness was determined utilizing a 2D strip with a width equal to the width of one sluice gate bay. The strips were designed as solid beams, given the property of the width of the slab that was examined. All loads acting along the width of the beams were input into STAAD (Structural Analysis and Design) and resolved along the centroid of the beam. Piles were modeled as pinned supports. The 2D strips were analyzed for the Bayou Grand Caillou sluice gates, which had similar hydrostatic loadings as the GIWW West and East sluice gates. The base slab thicknesses are summarized in Table 141.

Table 141 - Sluice Gate Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
GIWW West	5	8
GIWW East	5	8

5.2.6.3.12 Sluice Gate Pile Foundation

5.2.6.3.12.1 General

See Section 5.2.5.3.10.1

5.2.6.3.12.2 CPGA Analysis

See Section 5.2.5.3.10.1

5.2.6.3.12.3 Pile Curves and Horizontal Subgrade Modulus

Pile curves and horizontal subgrade modulus were calculated. The resulting pile tips are summarized in Table 142.

Table 142 - Sluice Gate Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
GIWW West	-130.0	-130.0
GIWW East	-137.0	-131.5

5.2.6.3.12.4 Cut-off Wall

See Section 5.2.5.3.10.4

5.2.6.3.13 Sluice Gates

The sluice gates will be manufactured 16'x16' or 16'x12' cast iron gates.

5.2.6.3.14 Sluice Gate Bulkheads

See Section 5.2.5.3.12.

5.2.6.3.15   Tie-in T-Walls

See Section 5.2.5.3.13.

5.2.6.4   Electrical Design

5.2.6.4.1 Electrical Service

The Electrical service to the structure will be 480/277 volt, 3 phase, 4 wire grounded secondary service from the local utility company. The service will be sized to support the structure loads including power for Gate machinery, lighting, controls, and any other miscellaneous loads.

5.2.6.4.2 Emergency Generator

A diesel generator will be provided for back-up power in the case of loss of utility power. The fuel supply for the generator will be provided from a fuel tank to a skid mounted UL-Listed double-walled day tank. Alarms will be locally annunciated on the generator.

5.2.6.4.3 Grounding System

The structure grounding system will be in accordance with the NFPA 70 - National Electrical Code. The grounding system will consist of copper ground rods interconnected with copper conductors. All jumpers and grounding electrode conductor connections will be done by exothermic weld. All electrical equipment, machinery, and exposed metal will be bonded to the grounding electrode system.

5.2.6.4.4 Lighting System

All exterior lighting fixtures will be provided with vandal-proof shields. The fixtures will be HPS and shall be controlled by photocells. Fluorescent light fixtures will be provided in the control houses.

5.2.6.4.5 Conduit and Boxes

All wiring will be installed in rigid metal conduit except that motors and other electrical equipment subject to vibration will be connected with liquid-tight flexible metal conduit. All pull boxes and junction boxes will be of cast metal of sufficient thickness or provided with bosses to accommodate the required threads for the conduit connections of size specified. All outlet boxes for receptacles, switches, and lighting fixtures will be of cast metal with bosses drilled and tapped or with threaded hubs of sizes specified. The

edges will be designed to take a heavy cover gasket with four or more screws for attaching covers or fixtures.

5.2.6.4.6 Controls

A hard wired control system will be installed to operate the Gates. The control consoles will be installed in the control houses. Additionally, local controls will be provided at each sluice gate.

5.2.6.4.7 Lightning Protection System

A lightning protection system will be designed to protect the structure from lightning strikes. The system will be designed in accordance with NFPA 780-Installation of Lightning Protection Systems. Surge suppression devices on all incoming power and communication lines will be provided.

5.2.6.5   Mechanical Design

5.2.6.5.1 Gate Operation

Gate operation will be two speeds with a time dependent 1 to 4 second speed ramp at start, stop and speed changes. The dual speed and speed ramp will be accomplished electronically by way of a hydraulic proportional valve. A slow gate speed of 3.5 degrees per minute will be used near the end of gate travel, (1 to 3 feet from fully close or fully open, measured at the skin plate). A higher speed of 30 degrees per minute will be used in between the ends of travel.

5.2.6.5.2 Gate Operating Loads

The gate operating loads consist of friction from hinge, pintle and seal and hydrodynamic loads. The hydrodynamic loads were based on differential hydrostatic head applied over the gate end beams. Four load cases were considered.

5.2.6.5.3 Gate Operating Machinery

The gate operating machinery will be a rack and pinion gear drive. The rack will be attached to the gate along the outside radius of the gate's skin plate. A pinion drive gear will be attached to a low speed high torque hydraulic (LSHT) motor mounted on the wall. A Hagglunds Viking Series 84 LSHT hydraulic motor operating at 3600 psi was used for design purposes. Each gate will be equipped with its own hydraulic power unit (HPU). The HPU will include a variable delivery pressure compensated pump driven by an electric motor. The electric motor will be 30 horsepower. Additional HPU

items will include valves, manifold, gauges, filters, clean vent, and storage tank. The gate operating machinery is shown on Plate M-002.

5.2.6.5.4 Hinge and Pintle Bushing Material

Hinge and pintle bushings will be split in the vertical plane. Hinge and pintle bushings will be a greaseless/self-lubricating system with an approved composite. The material will have a dynamic coefficient of friction that is less than or equal to 0.08 dry and 0.10 water-lubricated for a bearing pressure load of 2 ksi at surface speed 90 fpm. The ultimate compressive strength of the material will be a minimum of 50 ksi and its water absorption will be less than 0.10 percent by weight. Bushing material and dowels will meet the requirements of ASTM B 148, Alloy C95500, ASTM B 271, Alloy C95500, or ASTM A705, Type 630, minimum hardness 40 Rc. The hinge bushing shall normally be dry but may be exposed to rain water and a marine environment.

5.2.6.5.5 Sluice Gate Operation

Sluice gate operation will be to raise, lower, or hold in intermediate positions to allow, prevent, or meter drainage water flow and to prevent backflow during storm conditions. Operating time was designed to operate in less than fifteen minutes to totally raise or lower the sluice gate.

5.2.6.5.6 Sluice Gate Operating Loads

The sluice gate operating loads consist of friction from the stem, sluice gate weight, stem weight, hydrodynamic loads. The hydrodynamic loads were developed from differential hydrostatic head applied over the sluice gate.

5.2.6.5.7 Sluice Gate Operating Machinery

The sluice gate operating machinery will be an operating stem, bevel gearbox, and electric actuator. The operating stem will be attached to the top of the sluice gate within the stem pocket. The operating stem will be machine threaded AISI 316 stainless steel. The aluminum bronze ASTM B505 lift nut within the bevel gearbox will be machine threaded to mate with the AISI 316 stainless steel operating stem. The electric actuator will mount onto the bevel gearbox used to rotate the operating stem to raise or lower the sluice gate. The bevel gearbox used for design purposes for the 16'x16' sluice gates was a Diamond Gear 105BG7. The bevel gearbox used for design purposes for the 16'x12' sluice gates was a Diamond Gear 57BG6. The electric actuator considered for the 16'x16' sluice gates was a 30 horsepower Biffi Icon 2000 050/1440-173. The electric actuator considered for the 16'x12' sluice gates was a 10 horsepower Biffi Icon 2000 040/720-173. The sluice gate operating machinery is shown on Plate S-055.

5.2.7 Stop-Log Gates

This section contains a summary of work for the 3, 30' stop log gate structures and the 2, 20' stop log gate structures, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP alternatives. Table 143 lists the structures examined-

Table 143 - Stop-Log Gates

Structure	Sill Elevation	Top Elevation (3% AEP)	Top Elevation (1% AEP)	Top of Guidewalls
Elliot Jones	-8.0	16.0	23.5	10.0
Humphreys Canal	-8.0	16.0	23.5	10.0
Shell Canal West	-8.0	16.0	23.5	10.0
Marmande Canal	-8.0	16.5	23.0	10.0
Four Point Bayou	-8.0	22.5	30.0	10.0

5.2.7.1 Physical Features

The physical features associated with the construction of the stop log gate structures are-

- Interior Braced Cofferdams
- Stop Log Gate Concrete Monolith
- Stop Log Gate Pile Foundation
- Stop Log Gate
- Crane Platform T-Wall
- Needle Girder and Needles
- Bulkhead Storage Platform
- Guidewalls & Pile Clusters
- Tie-in T-Walls
- Mechanical Equipment

5.2.7.2 Construction Sequencing

All stop log gates will be constructed approximately in the center of the existing channels. A minimum 20' or 30' (depending on gate opening size) temporary bypass channel will be constructed as the first order of construction, allowing navigation passage during construction. Once navigation is routed through the temporary bypass channel, a cofferdam will be constructed, permitting the construction of the stop log gate monolith and the crane platform T-Wall monolith. Reduced power will be required for vessels passing through the construction area to reduce the risk of impact to the

cofferdam. A timber guidewall and pile clusters will be provided along the bypass channel to prevent vessel impact on the cofferdam. Once construction of the stop log gate monolith and the crane platform TWall monolith is completed, navigation will be re-routed through the permanent stop log gate structure. A phase 2 cofferdam will be required for the T-Wall adjacent to the stop log gate structures. Once navigation is re-routed, the phase 2 cofferdam, bulkhead storage platform, permanent guidewalls and pile clusters, tie-in T-Walls and final civil site work can be completed.

5.2.7.3 Structural Design

5.2.7.3.1 Cofferdams

A Phase 1 cofferdam will be constructed to permit the in the dry construction of the stop log concrete monolith and the crane platform T-Wall monolith. The cofferdam is an internally braced cofferdam with wide flange walers and pipe braces supporting PZ sheet piling. Anchor forces, bending moment in the sheet piling, and required sheet piling tip elevation were calculated for Bayou Dularge sector gate and conservatively used for the stop log gate structures. Details of the Phase 1 cofferdam for the sluice gate can be found on Plate S-101.

A phase 2 cofferdam will be constructed to permit the construction of the adjacent T-Walls to the stop log gate that will be in the water. The same anchor forces, moments, and tips used for the Phase 1 cofferdams will be conservatively used for the Phase 2 cofferdams. Details of the Phase 2 cofferdam can be found on Plate S-100.

ER 1110-2-8152 will be followed throughout the project design process, requiring that all cofferdams will be designed by the Government.

5.2.7.3.2 Stop Log Gate Monolith Concrete

Details of the concrete monolith are shown on Plates S-061 to S-062 (3% AEP) and S-061A to S-062A (1% AEP).

5.2.7.3.2.1 Stop Log Gate Walls

Stop log gate walls were designed as a cantilever beam extending from the base slab. A constant wall thickness was assumed the full height of the wall. A typical wall was designed for each sluice gate. Because the walls act as very deep beams when receiving the load from the stoplogs, the dewatering load case governed the design of the walls, resulting in a required wall thickness of 4' for all stop log gates.

5.2.7.3.2.2 **Stop Log Gate Base Slab**

Stop log gate base slabs were not designed rather historical data from previously constructed sluice gates was utilized to size the base slab. The selected base slab thicknesses are summarized in Table 144.

Table 144 - Stop Log Gate Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
Elliot Jones	6	6
Humphreys Canal	6	6
Shell Canal West	6	6
Marmande Canal	6	8
Four Point Bayou	6	8

5.2.7.3.3 Stop Log Gate Pile Foundation

5.2.7.3.3.1 **General**

The pile foundation for the 3% AEP LORR 20'stop log gates will include 30 HP 14X73 piles battered on 3 vertical to 1 horizontal slope while the pile foundation for the 1% AEP LORR 20' stop log gates will include 30 HP 14X73 piles on a similar batter. The pile foundation for the 3% AEP LORR 30 'stop log gates will include 49 HP 14X73 piles battered on 3 vertical to 1 horizontal slope while the pile foundation for the 1% AEP LORR 30' stop log gates will include 56 (49 for Shell Canal West) HP 14X73 piles on a similar batter. The design Factors of Safety utilized for the design comply with EM 1110-2-2906 and the latest requirements Hurricane and Storm Damage Risk Reduction System Design Guidelines. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks will be provided on all piles. CPGA analysis was performed for each stop log gate with the exception of Marmande Canal, where the Four Point Bayou was used. Details for the pile foundation are shown on Plates S-063 and S-070 (3% AEP LORR) and S-066A and S-070A (1% AEP LORR).   Alternative pile types and arrangements will be investigated during detailed design for each structure to optimize the pile foundation.

5.2.7.3.3.2 **CPGA Analysis**

CPGA was utilized to develop the pile layouts for the gate structures and determine the required tip elevation. The piles were modeled as pinned connections with the piles providing all of the lateral resistance. The horizontal subgrade modulus was based on the soil in the top ten pile diameters. The horizontal subgrade modulus was reduced for

group effects in accordance with EM 1110-2-2906.

5.2.7.3.3.3 Pile Curves and Horizontal Subgrade Modulus

Pile curves and horizontal subgrade modulus were calculated for a limited number of structures. Existing pile curve were utilized for those structures where no pile curve was calculated. A summary of the pile curve used for each structure is summarized below-

Stop Log Gate Structure		Pile Curve Utilized
Ellio Jones	→	*Box Culverts for Barrier Alignment*
Humphreys Canal	→	*Box Culverts for Barrier Alignment*
Shell Canal West	→	*Box Culverts for Barrier Alignment*
Four Point Bayou	→	*Lapeyrouse Canal*
Marmande Canal	→	*Lapeyrouse Canal*

The resulting pile tips are summarized in Table 145.

Table 145 - Stop Log Gate Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
Elliot Jones	-122.1	-131.6
Humphreys Canal	-122.1	-131.6
Shell Canal West	-108.1	-122.4
Marmande Canal	-117.4	-129.1
Four Point Bayou	-117.4	-129.1

5.2.7.3.3.4 Cut-off Wall

A cut-off sheetpile wall will be provided to reduce possible seepage, scouring and uplift. A PZC-13 sheetpile meeting the requirements of ASTM A572, Grade 50 was assumed for the cutoff wall. Tip elevations were calculated utilizing Lane's Weighted Creep Ratio for each structure. Details and tips for the cut-off wall are shown on Plates S-066 and S-070 (3% AEP LORR) and S-066A and S-070A (1% AEP LORR).

5.2.7.3.4 Stop Log Gate

5.2.7.3.4.1 General

The structural design of the stop log gates was performed in accordance with Corps engineering guidance and applicable industry standards. The Corps criterion is specified in EM1110-2-2105. The stop log gates will consist of horizontal wide-flanges supporting the vertical ribs and skin plate. All connections will be welded connections. A

crane mounted on an adjacent T-Wall will be used to lower the gate in place. All steel members on the gate will be painted with a coal tar epoxy paint system. Details of the steel stop log gates are shown on Plates S-064 and S-071 (3% AEP LORR) and S-064A and S-071A (1% AEP LORR).

5.2.7.3.4.2 **Skin Plate**

The skin plate was designed conservatively as a simply supported member by vertical angles, spaced 2' on center. An allowable stress of 0.50 times the yield stress was permitted for basic loading conditions with a permissible increase of one-third for abnormal loading conditions. EM 1110-2-2703 requires that the skin plate be designed with a 1/16" reduction in thickness.

5.2.7.3.4.3 **Vertical Ribs**

The skin plate will be attached to the vertical ribs by continuous welds. The ribs were designed as simply supported members between the horizontal wide-flanges. The skin plate was considered as an effective part of the vertical ribs, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. A minimum depth of ribs will be required to be 8 in. to facilitate painting and maintenance. The ribs are also constructed from material conforming to ASTM A-588 Grade 50 steel.

5.2.7.3.4.4 **Horizontal Beams**

The gate consists of horizontal beams supporting the vertical ribs and skin plate, spanning between the walls of the stop log gate concrete monolith. The beams are constructed from material conforming to ASTM A-572 Grade 50 steel.

5.2.7.3.5 Crane Platform T-Wall

The crane platform T-Wall will be located adjacent to the stop log gate monolith and functions as a T-Wall, as detailed in Section 5.2.5.3.13, with the addition of a crane load imposed on the monolith.

5.2.7.3.6 Needle Girders and Needles

The needle girder system arrangement was designed to dewater the entire gatebay to permit maintenance of the sluice gate concrete gatebay if necessary. The needle girder system was designed for a sill elevation of -8.0 with a water elevation of +5.0. Each stop log gate structure will utilize existing steel needles from other structures in the Morganza to the Gulf alignment as it is not anticipated that maintenance dewatering will be necessary during the design life of the structure. The needles are supported by the

sill of the concrete gatebay and the needle girder at El 5.0. The needle girder was designed as a simply supported, built-up girder, spanning across the 20' or 30' gate opening. The girder will be a plate girder with a depth of 2'–1½" with 5/8" web and ¾"x12" flanges. Details of the needle girder are shown on Plates S-064 and S-071.

5.2.7.3.7 Bulkhead Storage Platform

The bulkhead storage platform for the 20' stop log gate structures will be a reinforced concrete structure measuring 22'-6" wide by 30' long. The structure consists of a 12" cast in-place slab supported by 22" wide by 16" deep cast in-place beams, spaced 14'-1" O.C. The storage platform will be supported by 15, 14" square precast pre-stressed concrete (PPC) piles. Details of the bulkhead storage platform for the 20' stop log gate structures are shown on Plates S-065.

The bulkhead storage platform for the 30' stop log gate structures will be a reinforced concrete structure measuring 22'-6" wide by 30' long. The structure consists of a 15" cast in-place slab supported by 22" wide by 24" deep cast in-place beams, spaced 19'-1" O.C. The storage platform will be supported by fifteen 14" square precast pre-stressed concrete (PPC) piles. Details of the bulkhead storage platform for the 30' stop log gate structures are shown on Plates S-072.

5.2.7.3.8 Guidewalls and Pile Clusters

Guidewalls and pile clusters will be provided as aids to navigation and to protect the main flood gate structure from impact. Details were taken from historical 56' sector gate structures constructed in New Orleans District rather than performing actual design on this component. The wall lengths and details on the walls and pile clusters are shown on Plates S-021 to S-023.

5.2.7.3.9 Tie-in T-Walls

See Section 5.2.5.3.13.

5.2.7.4   Electrical Design

There is no electrical design associated with the stop log gate structures.

5.2.7.5   Mechanical Design

5.2.7.5.1 Crane Operation
The crane will be provided with a turret mounted operator cabin. The control station will provide two-lever joystick control of the hoisting, slewing, extend/retract, and luffing

functions. All motions will have step-less speed control from zero to maximum. Two motions may be operated at the same time at full capacity, but with reduced speed. The control station will include a panel mounted display screen, power on-off switch, local-remote selector switch, emergency pump unit stop switch, audible warning horn, start and stop switches for the pump unit motor.

5.2.7.5.2 Crane Operating Machinery

Crane operation will be accomplished by a pedestal mounted, double tapered, sealed box section boom crane incorporating a hydraulically driven winch. The hydraulically driven winches with planetary reduction gearing will be designed for operating in a marine environment. Winches will be provided with both a multiple disc wet friction brake which is spring set and hydraulically released and a counterbalance brake valve attached directly to a hydraulic piston motor for controlled load movement. Sufficient rope for the maximum hook drop will be specified. Load blocks will include roller bearings, center pin lubricated sheaves, dual action forged quenched and tempered alloy steel hooks and automatic safety latches. The pedestal will be manufactured from high strength low alloy structural steel in accordance with ASTM 514 Grade B. The upper flange of the pedestal will be machined to receive a slew bearing with an inspection access opening. A hydraulic connection plate, electrical and control junction box and hydraulic pressure filter will be mounted to the body of the pedestal. 360 degree continuous rotation will be provided via a planetary slew drive with fail-safe brake and dynamic motion control valve. The slew bearing will have a three-point contact ball bearing designed for heavy duty applications. Fasteners for the slew bearing will be high-grade tension bolts ensuring the rigidity of the bearing during operation. The gear and pinion will be located internally to protect the gearing from the dusty environment and will be equipped with suitable grease nipples. Hydraulic tube assemblies will be of AISI 316 stainless steel. Hydraulic supply hose will be neoprene or synthetic rubber tube with stainless steel braid reinforcement. Suction and return hose will be synthetic rubber tube with two or four layers of spiral textile reinforcement.

5.2.7.5.3 Crane Operating Loads

The crane operating loads were calculated based on the dead weight of the individual stop logs to be placed within the stop log gate. The lifting capacity of the crane was calculated using a safety factor of 1.35.

5.2.8 Environmental Control Structures

This section contains a summary of work for the 23 environmental control structures, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 146 lists the structures examined.

Table 146 - Environmental Control Structures

Structure	Culvert Type	Sill Elevation	Top Elevation (3% AEP)	Top Elevation (1% AEP)
Barrier 1	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 2	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 3	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 4	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 5	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 6	6 – 6' X 6'	-4.5	16.0	23.5
Barrier 7	6 – 6' X 6'	-4.5	16.0	23.5
Reach A	6 – 6' X 6'	-4.5	16.0	22.5
Reach E-1	9 – 6' X 6'	-4.5	18.0	25.5
Reach E-2	9 – 6' X 6'	-4.5	18.0	25.5
Reach G-2 - 1	6 – 6' X 6'	-4.5	22.5	30.5
Reach G-2 - 2	4 – 6' X 6'	-4.5	22.5	30.5
Reach G-3 - 1	4 – 6' X 6'	-4.5	22.5	30.5
Reach H-1 – 1	1 – 6' X 6'	-4.5	22.5	30.5
Reach H-1 – 2	6 – 6' X 6'	-4.5	22.5	30.5
Reach J2 – 1	4 – 5' X 10'	-3.5	25.0	33.0
Reach J2 – 2	4 – 5' X 10'	-3.5	25.0	33.0
Reach J2 – 3	5 – 5' X 10'	-3.5	25.0	33.0
Reach K – 1	2 – 6' X 6'	-4.5	21.0	29.5
Reach K – 2	2 – 6' X 6'	-4.5	21.0	29.5
Reach L	6 – 6' X 6'	-4.5	21.0	29.5
Larose to Lockport 1	3 – 5' X 10'	-3.5	15.0	18.0
Larose to Lockport 2	2 – 6.5' X 7.5'	-4.5	15.0	18.0

All elevations listed in this text and shown on the Tables and Plates, unless otherwise noted, are in feet, NAVD88.

5.2.8.1   Physical Features

The physical features associated with the construction of the environmental control structures are-

- Interior Braced Cofferdam
- Concrete Monolith
- Pile Foundation
- Sluice Gate
- Bulkheads
- Trash Racks
- Wingwalls
- Tie-in T-Walls
- Mechanical Equipment

5.2.8.2 Construction Sequencing

All environmental control structures will be constructed approximately in the center of the existing channels. A cofferdam will be constructed, permitting the construction of the environmental control structure concrete monolith and the wingwalls.

5.2.8.3 Structural Design

5.2.8.3.1 Cofferdams

A cofferdam will be constructed to permit the in the dry construction of the environmental control structure. The cofferdam is an internally braced cofferdam with wide flange walers and pipe braces supporting PZ sheet piling. Anchor forces, bending moment in the sheet piling, and required sheet piling tip elevation were calculated for Bayou Dularge sector gate and conservatively used for the environmental control structures.

ER 1110-2-8152 will be followed throughout the project design process, requiring that all cofferdams will be designed by the Government.

5.2.8.3.2 Environmental Control Structure Concrete Monolith

Details of the concrete monolith are shown on Plates S-081 to S-084 (3% AEP LORR) and S-081A to S-084A (1% AEP LORR).

5.2.8.3.2.1 Walls

The dimensions for the interior and exterior walls for the environmental control structures were not designed. Historical data from previously constructed environmental control structures was utilized to size these exterior and interior walls. The stem wall (main wall resisting hydrostatic forces) was designed as a cantilever from the driveway slab.

5.2.8.3.2.2 Base Slab

The sluice gate base slab thickness was determined utilizing a 2D strip with a width equal to the width of one pile spacing. The strip was designed as a solid beam, given the property of the width of the slab that was examined. All loads acting along the width of the beam were input into STAAD (Structural Analysis and Design) and resolved along the centroid of the beam. Piles reactions from CPGA were applied to the beam, which was assumed to be cantilevered off of the stem. The 2D strips were analyzed for all of the environmental control structures. The selected base slab thicknesses are

summarized in Table 147.

Table 147 - Environmental Control Structure Base Slab Thickness

Structure	Slab Thickness (ft) (3% AEP)	Slab Thickness (ft) (1% AEP)
Barrier 1	4.5	5.0
Barrier 2	4.5	5.0
Barrier 3	4.5	5.0
Barrier 4	4.5	5.0
Barrier 5	4.5	5.0
Barrier 6	4.5	5.0
Barrier 7	4.5	5.0
Reach A	4.5	5.0
Reach E-1	4.5	5.0
Reach E-2	4.5	5.0
Reach G-2 - 1	4.5	5.0
Reach G-2 - 2	4.5	5.0
Reach G-2 - 3	4.5	5.0
Reach H-1 – 1	4.5	5.0
Reach H-1 – 2	4.5	5.0
Reach J2 – 1	4.5	5.0
Reach J2 – 2	4.5	5.0
Reach J2 – 3	4.5	5.0
Reach K – 1	4.5	5.0
Reach K – 2	4.5	5.0
Reach L	4.5	5.0
Larose to Lockport 1	4.5	4.5
Larose to Lockport 2	4.5	4.5

5.2.8.3.3 Environmental Control Structure Pile Foundation

5.2.8.3.3.1 General

The pile foundation for the environmental control structures will include HP 14X73 piles battered on 3 vertical to 1 horizontal slope. The design Factors of Safety utilized for the design comply with EM 1110-2-2906 and the latest requirements Hurricane and Storm Damage Risk Reduction System Design Guidelines. All pile capacities used assumed compression pile testing, but no tension pile testing. Tension hooks are provided on all piles on the flood side of the sheet pile cutoff. CPGA analysis was performed for each environmental control structure. Details for the pile foundation are shown on Plates S-085 (3% AEP LORR) and S-085A (1% AEP LORR). Alternative pile types and

arrangements will be investigated during detailed design for each structure to optimize the pile foundation.

5.2.8.3.3.2   CPGA Analysis

CPGA was utilized to develop the pile layouts for the gate structures and determine the required tip elevation. The piles were modeled as pinned connections with the piles providing all of the lateral resistance. The horizontal subgrade modulus was based on the soil in the top ten pile diameters. The horizontal subgrade modulus was reduced for group effects in accordance with EM 1110-2-2906.

5.2.8.3.3.3   Pile Curves and Horizontal Subgrade Modulus

Pile curves and horizontal subgrade modulus were calculated for a limited number of structures. Existing pile curves were utilized for those structures where no pile curve was calculated. A summary of the pile curve used for each structure is summarized below:

Stop Log Gate Structure		Pile Curve Utilized
Barrier Alignment	→	*Box Culverts for Barrier Alignment*
Reach A	→	*Box Culverts for Reach B*
Reach E	→	*Box Culverts for Reach E & G*
Reach G	→	*Box Culverts for Reach E & G*
Reach H	→	*Box Culverts for Reach H*
Reach J	→	*Box Culverts for Reach H*
Reach K	→	*Box Culverts for Reach H*
Reach L	→	*Box Culverts for Reach H*
Lockport to Larose	→	*Box Culverts for Barrier Alignment*

The resulting pile tips are summarized in Table 148.

Table 148 - Environmental Control Structures Pile Tips

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
Barrier 1	-75	-85
Barrier 2	-75	-85
Barrier 3	-75	-85
Barrier 4	-75	-85
Barrier 5	-75	-85
Barrier 6	-75	-85

Structure	Pile Tip (ft) (3% AEP)	Pile Tip (ft) (1% AEP)
Barrier 7	-75	-85
Reach A	-75	-90
Reach E-1	-70	-90
Reach E-2	-70	-90
Reach G-2 - 1	-85	-105
Reach G-2 - 2	-85	-105
Reach G-3 - 1	-85	-105
Reach H-1 – 1	-65	-80
Reach H-1 – 2	-65	-80
Reach J2 – 1	-75	-90
Reach J2 – 2	-75	-90
Reach J2 – 3	-75	-90
Reach K – 1	-70	-85
Reach K – 2	-70	-85
Reach L	-65	-80
Larose to Lockport 1	-80	-80
Larose to Lockport 2	-80	-80

5.2.8.3.3.4 **Cut-off Wall**

A cut-off sheetpile wall will be provided to reduce possible seepage, scouring and uplift. A PZC-13 sheetpile meeting the requirements of ASTM A572, Grade 50 was assumed for the cutoff wall. Tip elevations were calculated utilizing Lane's Weighted Creep Ratio for each structure. Details and tips for the cut-off wall are shown on Plates S-085 (3% AEP LORR) and S-085A (1% AEP LORR).

5.2.8.3.4 Sluice Gates

The sluice gates will be manufactured 6'x6', 5'x10', 6.5'x7.5' or 5'x10' cast iron gates.

5.2.8.3.5 Bulkheads

The bulkheads are designed to dewater the sluice gate bays to permit maintenance of the sluice gates and concrete gatebay. The bulkheads were designed for a sill elevation of -4.5 with a water elevation of +5.0. Each sluice gate structure will be provided with 2 bulkheads, permitting the dewatering of 1 sluice gate bay at a time.

The steel bulkheads consist of horizontal angle shapes with a 3/8" skin plate. The skin plate was designed conservatively as a simply supported member between the

horizontal angles. The horizontal angles were designed as simply supported members between the sluice gates walls. The skin plate was considered as an effective part of the horizontal angles, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. All steel will be constructed from material conforming to ASTM A-572 Grade 50. Details of the sluice gate bulkheads are shown on Plate S-086.

5.2.8.3.6 Trash Racks

Trash racks will be provided on both the flood and protected sides of the sluice gates to prevent large debris from blocking the closure of the sluice gates. Historical data from previously constructed environmental control structures was utilized to size the trash racks. The tracks will be constructed of galvanized steel plate conforming to the requirements of ASTM A-572 Grade 50. Details of the trash racks are shown on Plate S-088.

5.2.8.3.7 Wingwalls

Wingwalls will be provided on all 4 corners of the environmental control structure to retain fill and to provide a smooth flow transition into the environmental control structures. The wingwalls are pile founded TWall type concrete monoliths. Historical data from previously constructed environmental control structures was utilized to size the wing walls. The wing walls will be supported on HP 14X73 steel piling, whose tips will be extended to the same tip elevation of the environmental control structure pile tips to prevent differential settlement. Details of the wingwalls are shown on Plate S-087.

5.2.8.3.8 Tie-in T-Walls

See Section 5.2.5.3.13.

5.2.8.4 Electrical Design

There is no electrical design associated with the environmental control structures.

5.2.8.5 Mechanical Design

5.2.8.5.1 Sluice Gate Operation

Sluice gate operation will be raised, lowered, or held in intermediate positions to allow, prevent, or meter water flow respectively. Operating time is designed to operate in less than fifteen minutes to totally raise or lower the sluice gate.

5.2.8.5.2 Sluice Gate Operating Loads

The sluice gate operating loads consist of friction from the stem, sluice gate weight, stem weight, hydrodynamic loads. The hydrodynamic loads were developed from differential hydrostatic head applied over the sluice gate.

5.2.8.5.3 Sluice Gate Operating Machinery

The sluice gate operating machinery will be an operating stem, bevel gearbox, and portable actuator. The operating stem will be attached to the top of the sluice gate within the stem pocket. The operating stem will be machine threaded AISI 316 stainless steel. The aluminum bronze ASTM B505 lift nut within the bevel gearbox shall be machine threaded to mate with the AISI 316 stainless steel operating stem. The portable actuator will be able to exercise the sluice gate via the 2" AWWA nut installed within the bevel gearbox. The bevel gearbox used for design purposes was a Diamond Gear 7.2BG3. The portable actuator considered was a Waterman GP-6 gasoline powered portable actuator. The sluice gate operating machinery is shown on Plate S-082.

5.2.9 Pump Station Fronting Protection

This section contains a summary of work for the 4 pump station fronting protections, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 149 lists the structures examined.

Table 149 - Pump Stations

Pump Station	Pump Sizes	Top Elevation (3% AEP)	Top Elevation (1% AEP)
Madison	2 – 48"	25.0	33.0
Pointe Aux Chenes	2 – 20"	25.0	33.0
Bayou Black	2 – 42"	16.0	23.5
Hanson Canal	2 – 42"	16.0	23.5

5.2.9.1   Physical Features

The physical features associated with the construction of the pump station fronting protection are as follows:

- Fronting Protection T-Walls
- Mechanical Equipment – Butterfly Valves

5.2.9.2 Construction Sequencing

All fronting protections will be constructed on the flood side of the existing protection. Based on site visits conducted for this report, the discharge pipes extend far enough such that additional pipe length will not be needed.

5.2.9.3 Structural Design

5.2.9.3.1 Fronting Protection Walls

All fronting protection walls were designed as T-Walls as described in Section 5.2.5.3.13.

5.2.9.4 Electrical Design

There is no electrical design associated with the butterfly valves.

5.2.9.5 Mechanical Design

No calculations were performed on the existing pumps and engines to determine effects (if any) from head losses associated with butterfly valves and discharge piping modifications. It was assumed that the existing pumps and engines will be adequate.

5.2.9.5.1 Butterfly Valve Operation

Butterfly valve operation will be opened to allow pumping discharge for interior drainage or closed to prevent backflow during storm conditions. Operating time was designed to operate in less than fifteen minutes to totally open or close the butterfly valve.

5.2.9.5.2 Butterfly Valve Operating Loads

The butterfly valve operating loads consist of friction from the disc, disc weight, and hydrodynamic loads. The hydrodynamic loads considered closing against a full discharge pipe in a backflow condition.

5.2.9.5.3 Butterfly Valve Operating Machinery

The butterfly valve operating machinery will be the butterfly valve, gearbox, and portable actuator. Butterfly valves on the discharge piping lines will conform to AWWA C504. The valve will be suitable for air tight seating in either direction. The body will be ASTM A 126 Class B heavy cast iron. All valves will have flanged by flanged ends. 75# ANSI B16.1 or AWWA C207 flanged ends, with shaft of ASTM A 276 Type 304 Stainless

Steel. Packing will be self adjusting "V" type made of PTFE. Shaft bearings will be corrosion resistant, self-lubricating type, Teflon lined, sleeve-type bearings. The valve disc will be constructed of ASTM A 536 ductile iron with a stainless steel seating edge. The disc will not have any hollow chambers that can entrap water. Disc and shaft connection will be made with stainless steel pins. Valves will be painted on its interior and exterior with the same coatings used for the discharge piping. The portable actuator will be able to exercise the sluice gate via the 2" AWWA nut installed within the gearbox. The portable actuator considered was a Waterman GP-6 gasoline powered portable actuator. The butterfly valve and operating machinery is shown on Plate M-003.

5.2.10 Roadway/Railroad Gates

This section contains a summary of work for the 10 roadway/railroad swing gates, which are part of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 150 lists the structures examined.

Table 150 - Roadway Gates

Roadway	Gate Opening (ft)	Top Elevation (3% AEP)	Top Elevation (1% AEP)
Hwy 315	36	18.0	25.5
Hwy 55	36	25.0	33.0
Hwy 56	36	22.5	30.0
Hwy 665	36	25.0	33.0
NAFTA	36	16.0	23.5
Four Point Road	36	22.5	30.0
Hwy 24	36	14.0	17.0
Hwy 3235 - 1	36	14.0	17.0
Hwy 3235 - 2	36	14.0	17.0
Union Pacific RR	36	15.0	18.0

5.2.10.1 Physical Features

The physical features associated with the construction of the roadway gates structures are as follows:

- Steel Swing Gate
- Traffic Control Devices
- Falsework (Railroad Gates)
- Concrete Monolith
- Tie-in T-Walls

5.2.10.2   Construction Sequencing

All roadway gates except for the NAFTA gate are directly adjacent to navigation gates; therefore they will be constructed concurrent with those structures. The roadway gate concrete monoliths will be constructed in two- halves to permit traffic flow during construction of the concrete monoliths. Detours and traffic control will conform to LADOTD Standards. Railroad gates will be constructed with temporary falsework to minimize disruptions to the railroad during construction

5.2.10.3   Structural Design

5.2.10.3.1   Steel Swing Gates

5.2.10.3.1.1 General

The structural design of the steel swing gates was performed in accordance with Corps engineering guidance and applicable industry standards. The Corps criterion is specified in EM1110-2-2105 and EM 1110-2-2705. The swing gates will consist of structural wide flange sections supporting the vertical ribs and skin plate. All connections will be welded connections. All steel members on the gate will be painted with a vinyl paint system. Details of the steel swing gate are shown on Plate S-076 (3% AEP) and S-076A (1% AEP).

5.2.10.3.1.2 Skin Plate

The skin plate was designed conservatively as a continuously supported member by vertical intercostals. An allowable stress of 0.50 times the yield stress was permitted for basic loading conditions with a permissible increase of one-third for abnormal loading conditions.

5.2.10.3.1.3 Vertical Intercostals

The skin plate will be attached to the vertical intercostals by continuous welds. The intercostals were designed as simply supported members between the horizontal girders. The skin plate was considered as an effective part of the vertical intercostals, with the effective width of skin plate determined according to the AISC specifications for a non-compact flange. A minimum depth of intercostals will be required to be 8 in. to facilitate painting and maintenance. The intercostals will be constructed from material conforming to ASTM A-572 Grade 50 steel.

5.2.10.3.1.4   **Horizontal Beams**

The gate will consist of horizontal wide flange sections supporting the vertical intercostals and skin plate. The beam was designed as simply supported between the concrete pilasters of the swing gate monolith. The beams are constructed from material conforming to ASTM A-992 Grade 50 steel.

5.2.10.3.2   Swing Gate Concrete Monolith and Pile Foundation

The swing gate concrete monolith and pile foundation was not designed, rather the typical T-Wall design as described in Section 5.2.5.3.13 was utilized for quantity estimation.

5.2.10.3.3   Traffic Control Devices

Each roadway gate will include guardrails meeting the requirements of LADOTD GR-200 and end treatment on all four sides of the structure. Removable Vulcan barriers will be provided as guardrails in the gate swing radius.

5.2.11   Pipeline Crossings

This section contains a summary of work for the 3 pipeline crossing T-Walls, which are part of the Larose to Lockport Reach of the Morganza to the Gulf Alignment for both the 3% AEP and the 1% AEP level of protections. Table 151 lists the structures examined.

Table 151 - Roadway Gates

Pump Station	Top Elevation (3% AEP)	Top Elevation (1% AEP)
C North Gulf South Pipeline	18.0	23.0
C North American Midstream Pipeline	18.0	23.0
C North Williams Discovery Pipeline	18.0	23.0

5.2.11.1   Physical Features

The physical features associated with the construction of the roadway gates structures are as follows:

- Utility Crossing T-Wall
- Utility Sleeve
- Cofferdam
- Tie-in T-Walls (Union Pacific Railroad Gate Only)

5.2.11.2 Construction Sequencing

A cofferdam will be constructed to construct the sleeve of the pipeline crossing through the T-Wall.

5.2.11.3 Structural Design

5.2.11.3.1 Utility Crossing T-Wall Concrete Monolith and Pile Foundation

The utility crossing concrete monolith and pile foundation was not designed, rather the typical T-Wall design as described in previous sections was utilized for quantity estimation.

5.2.11.3.2 Cofferdam

The cofferdam design as described in Paragraph 5.2.5.3.1 was used to develop quantities for the cofferdam required to construct the pipeline crossing sleeve.

5.2.11.3.3 Tie-in T-Walls

Tie-in T-Walls extend from the utility crossing T-Wall to the full levee section. See Paragraph 5.2.5.3.13 for design details.

6 RELOCATIONS

6.1 Scope and Purpose

Relocation data was collected, tabulated and detailed in this Section by the U.S. Army Corps of Engineers, New Orleans District, Engineering Division, Design Service Branch, Relocations Team, to a feasibility level of design. The Relocations Team made contact with pipeline owners to obtain detailed information on existing facilities. Approximately half of the pipeline owners responded with information. Three pipeline databases were used where information was not available from the owners, and to supplement it when it was available. The databases used were the National Pipeline Mapping System (NPMS) database, the Louisiana Department of Natural Resources (LA DNR) database, and the commercial database HTSI. Because of the number of pipelines in the Barrier Alignment Reach, duplication between databases, and some apparent errors in the

HTSI data in the Barrier Alignment Reach, the HTSI data was not used for the Barrier Alignment Reach. With the exception of Entergy, none of the other utility owners (Terrebonne Parish government, Lafourche Parish government, or the local utility and cable companies) were contacted. It should further be noted that the Relocations Team was unable to obtain Right of Entry for about half of the project alignment, including the Barrier Alignment Reach, Reach A, Reach B, Reach G, and about half of Reach L. The Relocations Team then made assumptions based on the proposed project design and project location to determine project relocation requirements. Oil wells were assumed to not be relocation items, and the levee alignment may need to be changed, or T-walls used, during the project Plans and Specifications (P&S) phase to avoid them. The cost estimates presented in this report were developed by New Orleans District, Engineering Division, Design Service Branch, Cost Engineering Team by developing cost estimates for the relocation items. These relocation costs represent a feasibility level of design and will be further refined during the development of the project P&S.

Additional relocations details will be furnished upon request.

6.2   Estimated Relocations Cost

6.2.1   General

The cost estimates presented herein are based on conceptual relocations designs. We developed these plans with the criteria input that we received from the owners (for those that submitted input), and from the pipeline databases used. Included in the construction cost estimates are the facility owners engineering design, contract supervision, and administration.

6.2.2   Methods of Relocation

 Four methods of relocating affected pipelines were investigated as follows:

a.   Above ground installation on the levee surface covered by additional fill.
b. Above ground installation, supported by temporary (until final lift is completed) pile bents.
c. Permanent Pipeline Bridge supported by pile founded piers.
d. Directional drilling.

The main conditions influencing the selection of relocation methods is the substantial subsidence anticipated by the proposed levee, and the costs associated with each

method.

6.2.2.1   Pipeline Relocation on the Levee Surface

This method of relocation is typically used for pipelines affected by main line Mississippi River and Atchafalaya Basin levees. Typically, the pipeline is installed on the levee net grade and follows the contour of the levee until it proceeds underground on the flood side and protected side of the levee. Additional fill producing the gross grade is placed over the pipeline to provide protection. This method presented problems in some places due to the anticipated subsidence of the levees on which the pipeline would rest. Since the levee provides support for the pipeline, the anticipated levee subsidence would result in undesirable stresses on the pipeline. The multiple lifts expected to meet project flood protection elevations also raised concerns about multiple relocations, a prospect which would not be welcomed by the pipeline owners. Although the cost for initial pipeline relocation by this method would probably be the least costly, the future cost of subsequent adjustments, modifications, and relocations of the pipeline would diminish this advantage.

6.2.2.2   Pipeline Relocation using Temporary Pile Bents

This method is similar to the above, but uses temporary pile bents in the levee to support the pipeline and prevent excessive settlements. Since the pile bents penetrate the levee 1-4 slope line, a sheetpile cutoff wall is required in the levee at or near the centerline.

6.2.2.3   Permanent Pipeline Bridge

 This method of relocation centers around installations of affected pipelines on bents supported by pile piers. The advantage of this method is that it would diminish the affects of the expected subsidence. However, it is possible that subsidence of the permanent pipeline bridge would also cause stress on the pipeline, requiring additional modification of the bridge. Another disadvantage would be that the pipeline would be exposed and vulnerable to vandalism. Additionally, the presence of a permanent structure over the levee would impede levee maintenance and construction of future levee lifts.

6.2.2.4   Directional Drilling

Directional drilling of pipelines under existing levees is an acceptable method of pipeline relocation, but pressures must be monitored to prevent possible fracturing of the levee from the pressure of the drilling fluid. Directional drilling is a reliable method of relocation and can be done prior to construction of the levee. From a geotechnical

perspective, the pipeline should be installed deep enough under the levee section and any berm sections to avoid stresses from the levee and berm subsidence. The advantage of this method is that it eliminates additional modification and/or relocation in the future.

Additionally, the pipeline cannot be vandalized or damaged by levee maintenance or future construction. From a negative viewpoint, the initial cost of directional drilling would be greater than that of the other two relocation methods studied. However the cost of future modifications and/or relocations resulting from the use of either of these two methods would most likely make the final relocation costs comparable to directional drilling.

6.2.3 Highways and Roads

The flood protection alignment being studied impacts several State and Parish highways and roads. Traversing the proposed flood protection requires either relocating the highways and roads over the protection, using ramps, or construction of permanent floodgates at the points of intersection of the protection and highways and roads. These structures would remain open except for times of anticipated flooding, which at that time they would be closed and remain closed until the flood threat subsided. The alternatives selected for this study were an earthen ramp for highway 182 (aka Bayou Black Drive), and floodgates for all the others.

6.2.4 Estimated Cost

6.2.4.1 Utilities

The total estimated cost for relocation of all utilities is $222,810,775.28 for the 3% AEP level of risk reduction, and $223,689,602.50 for the 1% AEP level of risk reduction. Engineering and Design (E&D) costs and Supervision and Administration (S&A) costs are included, but Contingency costs and Escalation costs are not. For pipelines, the costs are the same for both 3% AEP and 1% AEP levels of risk reduction, since the directional drill distances were taken as the same for both levels of risk reduction, because a 3% AEP Microstation dgn (electronic footprint) file was not available during the Relocations study, because the sleeve-through costs were taken as the same, and because the up-and-over relocation costs were taken as the same. Estimated 1% AEP relocation costs are summarized in Table 152 (except roads), and estimated 3% AEP relocation costs are summarized in Table 153 (except roads). Relocation costs for roads are in Table 154. All utilities affected by the project along with the associated maps will be furnished upon request. It should be noted that if a public facility owner has an interest in the land occupied by its compensable facilities that require relocation, the Government is obligated to provide substitute facilities in exchange for the owner

subordinating or releasing its interest in land to the Government. The Government usually reimburses the owner for relocation of its facilities. However, the Government may provide all or part of the substitute facilities for the owner. Any relocation subsequent to the owner subordinating its interest in the land to the Government, that is not consequential to the current project, would be at the owner's expense.

Table 152   1% AEP Relocations Cost

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
BARRIER ALIGNMENT				
BAR-P1	1023+00	$291,146.89	$0.00	$291,146.89
BAR-P2	1023+00	$16,221.92	$0.00	$16,221.92
BAR-P2	1110+28	$400,176.80	$0.00	$400,176.80
BAR-P3	1110+15	$16,648.64	$0.00	$16,648.64
BAR-P4	1109+50	$15,987.02	$0.00	$15,987.02
BAR-P5	1110+00	$15,728.63	$0.00	$15,728.63
BAR-P6	1109+10	$16,433.31	$0.00	$16,433.31
BAR-P6	1423+00	$0.00	$0.00	$0.00
BAR-P7	1023+00	$16,221.92	$0.00	$16,221.92
BAR-P7	1110+00	$16,237.56	$0.00	$16,237.56
BAR-P8	1110+78	$15,728.63	$0.00	$15,728.63
BAR-P9	1112+64	$15,728.63	$0.00	$15,728.63
BAR-P10	1176+18	$291,342.64	$0.00	$291,342.64
BAR-P11	1176+50	$16,402.02	$0.00	$16,402.02
BAR-P12	1213+10	$509,184.41	$0.00	$509,184.41
BAR-P13	1210+65	$0.00	$0.00	$0.00
BAR-P13	1333+35	$5,583,045.85	$516,878.93	$6,099,924.78
BAR-P13	1505+00	$5,583,045.85	$516,878.93	$6,099,924.78
BAR-P13	1639+00	$0.00	$0.00	$0.00
BAR-P14	1327+03	$291,557.96	$0.00	$291,557.96
BAR-P15	1327+13	$291,146.89	$0.00	$291,146.89
BAR-P16 *	n/a	$0.00	$0.00	$0.00
BAR-P17	1433+00	$0.00	$0.00	$0.00
BAR-P18	1442+24	$1,086,368.10	$0.00	$1,086,368.10
BAR-P19	1462+50	$509,185.48	$0.00	$509,185.48
BAR-P20	1484+50	$0.00	$0.00	$0.00
BAR-P21	1575+65	$613,487.09	$0.00	$613,487.09
BAR-P22	1564+00	$0.00	$0.00	$0.00
BAR-P23	1578+50	$1,644,105.22	$0.00	$1,644,105.22
BAR-P24	1585+75	$18,660.36	$8,760.66	$27,421.02

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
BAR-P25	1621+10	$509,185.48	$0.00	$509,185.48
BAR-P26	1629+40	$559,989.62	$0.00	$559,989.62
BAR-P27	1766+00	$0.00	$0.00	$0.00
BAR-P28	1780+40	$509,185.48	$0.00	$509,185.48
BAR-P29	1780+43	$559,989.62	$0.00	$559,989.62
BAR-P29	1782+90	$290,508.25	$0.00	$290,508.25
BAR-P30	1782+90	$290,387.13	$0.00	$290,387.13
BAR-P31	1410+40	$0.00	$0.00	$0.00
BAR-P32	1410+40	$0.00	$0.00	$0.00
BAR-P33	1410+40	$0.00	$0.00	$0.00
BAR-P34	1410+40	$0.00	$0.00	$0.00
REACH A				
A-P1	1836+20	$613,487.09	$0.00	$613,487.09
A-P1	1881+90	$290,658.54	$0.00	$290,658.54
A-P1	1900+55	$618,180.30	$0.00	$618,180.30
A-P2	1931+00	$2,044,997.82	$516,878.93	$2,561,876.75
A-P3	1931+00	$2,066,735.79	$516,878.93	$2,583,614.72
A-P4	1985+53	$35,751.63	$26,281.98	$62,033.61
A-P5	1985+53	$35,751.63	$26,281.98	$62,033.61
A-P6	1997+70	$559,989.62	$0.00	$559,989.62
A-P7	2010+05	$559,989.62	$0.00	$559,989.62
A-P8	2015+50	$788,602.94	$0.00	$788,602.94
A-P8	2193+10	$788,602.94	$0.00	$788,602.94
A-P9	2025+25	$509,281.14	$0.00	$509,281.14
A-P10	2025+25	$509,185.48	$0.00	$509,185.48
A-P11	2193+65	$624,593.39	$0.00	$624,593.39
REACH B				
B-P1	2340+15	$2,037,966.61	$0.00	$2,037,966.61
B-P2	2339+40	$0.00	$0.00	$0.00
B-P3	2514+60	$61,278.95	$47,557.87	$108,836.82
REACH E-2				
E2-P1	2622+30	$65,077.76	$40,048.73	$105,126.49
E2-P2	2629+30	$5,617,436.63	$516,878.93	$6,134,315.56
E2-P3	2626+30	$0.00	$0.00	$0.00
REACH E-1				
E1-P1	2751+40	$0.00	$0.00	$0.00
E1-P2	2800+00	$248,188.54	$0.00	$248,188.54
REACH F-2				

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
F2-P1	2779+05	$0.00	$0.00	$0.00
REACH F-1		0	0	0
NONE		$0.00	$0.00	$0.00
REACH G-1		0	0	0
NONE		$0.00	$0.00	$0.00
REACH G-2		0	0	0
NONE		$0.00	$0.00	$0.00
REACH G-3		0	0	0
NONE		$0.00	$0.00	$0.00
REACH H-1		0	0	0
H1-P1	3313+75	$0.00	$0.00	$0.00
H1-P2	3313+75	$1,398,516.75	$0.00	$1,398,516.75
H1-P3	3316+15	$42,075.34	$0.00	$42,075.34
REACH H-2		0	0	0
H2-P1	3457+10	$0.00	$0.00	$0.00
H2-P2	3457+10	$0.00	$0.00	$0.00
H2-P3	3457+10	$0.00	$0.00	$0.00
H2-P4	3457+10	$0.00	$0.00	$0.00
H2-P5	3457+10	$0.00	$0.00	$0.00
H2-P6	3439+10	$0.00	$0.00	$0.00
H2-P7	3439+10	$0.00	$0.00	$0.00
H2-P8	3449+10	$0.00	$0.00	$0.00
REACH H-3		0	0	0
H3-P1	3571+60	$0.00	$0.00	$0.00
H3-P2	3636+60	$0.00	$0.00	$0.00
H3-P3	3611+53	$0.00	$0.00	$0.00
H3-P4	3611+53	$0.00	$0.00	$0.00
H3-P5	3467+73	$0.00	$0.00	$0.00
H3-P6	3464+33	$0.00	$0.00	$0.00
H3-P7	3464+13	$0.00	$0.00	$0.00
H3-P8	3466+38	$0.00	$0.00	$0.00
H3-P9	3463+53	$0.00	$0.00	$0.00
H3-P10	3463+53	$0.00	$0.00	$0.00
H3-P11	3611+13	$0.00	$0.00	$0.00
H3-P12	3507+13	$0.00	$0.00	$0.00
REACH I-1		0	0	0
NONE		$0.00	$0.00	$0.00
REACH I-2		0	0	0

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
I2-P1	3808+00	$62,803.04	$47,557.87	$110,360.91
I2-P2	3807+11	$0.00	$0.00	$0.00
I2-E1	3760+60	$1,093,844.82	$83,852.03	$1,177,696.85
REACH I-3				
I3-P1	3922+60	$5,320,942.19	$516,878.93	$5,837,821.12
I3-P2	3918+13	$6,315,581.00	$516,878.93	$6,832,459.93
I3-P3	3915+67	$5,521,277.53	$516,878.93	$6,038,156.46
I3-P4	3914+50	$5,498,280.79	$516,878.93	$6,015,159.72
I3-P5	3900+90	$4,653,726.48	$516,878.93	$5,170,605.41
I3-P6	3900+10	$6,895,810.27	$516,878.93	$7,412,689.20
I3-P7	3896+89	$4,326,481.01	$516,878.93	$4,843,359.94
I3-P8	3865+54	$4,297,679.41	$516,878.93	$4,814,558.34
I3-P9	3919+20	$0.00	$0.00	$0.00
I3-W1	3928+60	$344,189.89	$103,876.40	$448,066.29
REACH J-2				
J2-P1 *	n/a	$0.00	$0.00	$0.00
J2-P2	3999+14	$0.00	$0.00	$0.00
J2-P3	4162+40	$10,478,095.41	$2,277,771.56	$12,755,866.97
J2-P4	4161+60	$10,375,194.05	$516,878.93	$10,892,072.98
J2-P5	4037+50	$12,618,256.87	$516,878.93	$13,135,135.80
J2-P6	3990+50	$0.00	$0.00	$0.00
J2-P7	3986+48	$0.00	$0.00	$0.00
J2-P8	3963+08	$9,289,042.99	$516,878.93	$9,805,921.92
J2-P9	3956+02	$5,760,341.62	$516,878.93	$6,277,220.55
J2-P10	3955+90	$999,052.03	$0.00	$999,052.03
J2-P11		$162,835.33	$103,876.40	$266,711.73
J2-P12		$0.00	$0.00	$0.00
J2-P13		$0.00	$0.00	$0.00
J2-P14		$0.00	$0.00	$0.00
J2-P15		$0.00	$0.00	$0.00
J2-E1	3984+96	$0.00	$0.00	$0.00
REACH J-1				
J1-P1	4233+75	$0.00	$0.00	$0.00
J1-P2 *	n/a	$0.00	$0.00	$0.00
J1-P3	4204+30	$16,557,557.05	$1,198,958.88	$17,756,515.93
J1-P4 *	n/a	$0.00	$0.00	$0.00
J1-P5	4330+15	$0.00	$0.00	$0.00
REACH J-3				

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
J3-P1	4431+10	$6,089,591.21	$516,878.93	$6,606,470.14
REACH K				
K-P1	4606+00	$0.00	$0.00	$0.00
REACH L				
L-P1	4777+80	$283,141.57	$0.00	$283,141.57
L-P1	4841+50	$1,353,315.09	$0.00	$1,353,315.09
L-P2	4808+30	$1,086,755.28	$0.00	$1,086,755.28
L-P2	4861+30	$1,086,368.10	$0.00	$1,086,368.10
L-P3	4866+05	$1,327,779.56	$0.00	$1,327,779.56
L-P4	4867+40	$1,223,618.66	$0.00	$1,223,618.66
L-P5	5017+90	$1,074,586.37	$0.00	$1,074,586.37
L-P6	5019+40	$2,045,788.63	$0.00	$2,045,788.63
L-P7	c. 4914+20	$563,118.43	$0.00	$563,118.43
L-P8	5019+40	$1,710,575.94	$0.00	$1,710,575.94
L-P9	c.4914+20	$24,242.90	$12,515.23	$36,758.13
L-P10	4732+35	$0.00	$0.00	$0.00
REACH LGM	**LGM PAC station values**			
LGM-P1	None, see map	$192,491.32	$0.00	$192,491.32
LGM-P2	None, see map	$192,491.32	$0.00	$192,491.32
LGM-P3	None, see map	$203,731.02	$0.00	$203,731.02
LGM-P4	None, see map	$188,735.37	$0.00	$188,735.37
LGM-P5	None, see map	$188,735.37	$0.00	$188,735.37
LGM-P6	None, see map	$0.00	$0.00	$0.00
LGM-P7	742+90	$188,735.37	$0.00	$188,735.37
LGM-P8	740+00	$188,735.37	$0.00	$188,735.37
LGM-P9	740+00	$75,581.33	$0.00	$75,581.33
LGM-W1	743+15	$92,021.61	$0.00	$92,021.61
LGM-W2	740+80	$92,319.08	$0.00	$92,319.08
LGM-W3	741+60	$92,021.61	$0.00	$92,021.61
LGM-C1	743+20	$84,971.10	$0.00	$84,971.10
LGM-C2	743+20	$3,911.01	$0.00	$3,911.01
LGM-E1	740+65	$60,029.44	$0.00	$60,029.44
LGM-E2	761+00	$58,902.38	$0.00	$58,902.38
LGM-E3	761+41	$2,563.68	$0.00	$2,563.68
LGM-E4	765+50	$22,563.01	$0.00	$22,563.01
LGM-E5	769+76	$2,957.08	$0.00	$2,957.08
LGM-P10	762+18 to	$135,336.32	$0.00	$135,336.32

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
	762+92			
LGM-P11	762+31	$188,735.37	$0.00	$188,735.37
LGM-P12	767+73	$1,511,799.78	$0.00	$1,511,799.78
LGM-C3	772+55	$0.00	$0.00	$0.00
LGM-W4	771+70	$192,491.32	$0.00	$192,491.32
LGM-W5	786+36	$192,491.32	$0.00	$192,491.32
LGM-P13	791+15	$1,392,700.49	$0.00	$1,392,700.49
LGM-P14	792+67	$1,277,371.85	$0.00	$1,277,371.85
LGM-W6	792+85	$142,172.75	$0.00	$142,172.75
LGM-C4	On Bayou Lafourche	$312,657.74	$0.00	$312,657.74
LGM-C5	On Bayou Lafourche	$312,657.74	$0.00	$312,657.74
LGM-E6	On Bayou Lafourche	$317,547.85	$0.00	$317,547.85
LGM-E7	On Bayou Lafourche	$231,708.38	$0.00	$231,708.38
LGM-E8	On Bayou Lafourche	$188,735.37	$0.00	$188,735.37
LGM-P15	On Bayou Lafourche	$1,511,799.78	$0.00	$1,511,799.78
LGM-C6	none	$0.00	$0.00	$0.00
LGM-W7	1064+70	$192,491.32	$0.00	$192,491.32
LGM-C7	1064+70	$188,735.37	$0.00	$188,735.37
LGM-C8	1064+70	$188,735.37	$0.00	$188,735.37
LGM-C9	none	$0.00	$0.00	$0.00
LGM-P16	1064+70	$192,491.32	$0.00	$192,491.32
LGM-W8	1042+16	$192,491.32	$0.00	$192,491.32
LGM-P17	941+50	$0.00	$0.00	$0.00
LGM-P18	920+87	$192,491.32	$0.00	$192,491.32
LGM-P19	905+28	$192,491.32	$0.00	$192,491.32
LGM-P20	890+77	$192,491.32	$0.00	$192,491.32
LGM-E9	868+90	$0.00	$0.00	$0.00
LGM-P21	approx 856+00	$561,662.05	$62,576.14	$624,238.19
LGM-P22	843+73	$519,201.40	$62,576.14	$581,777.54
LGM-P23	843+43	$0.00	$0.00	$0.00
LGM-P24	821+38	$607,035.00	$62,576.14	$669,611.14
LGM-P25	821+08	$519,201.40	$62,576.14	$581,777.54

Relocation Item ID	ED-L Station (100yr = 35yr)	Relocation Cost	Mitigation Cost	Total
REACH LL		0	0	0
LL-C1	2160+70	$590,534.46	$62,576.14	$653,110.60
LL-P1	2261+20	$1,117,364.05	$62,576.14	$1,179,940.19
LL-P1	2506+70	$1,073,875.84	$62,576.14	$1,136,451.98
LL-P1	2705+15	$1,412,117.52	$62,576.14	$1,474,693.66
LL-P1	2775+00 to 2778+85	$0.00	$0.00	$0.00
LL-P2	2261+20	$1,168,093.53	$62,576.14	$1,230,669.67
LL-P2	2506+70	$1,153,084.38	$62,576.14	$1,215,660.52
LL-P2	2705+15	$1,509,117.36	$62,576.14	$1,571,693.50
LL-P2	2775+00 to 2778+85	$0.00	$0.00	$0.00
LL-P3	2263+80	$2,426,909.61	$62,576.14	$2,489,485.75
LL-P3	2488+00	$2,426,909.61	$62,576.14	$2,489,485.75
LL-P3	2712+80	$2,919,743.85	$62,576.14	$2,982,319.99
LL-P3	2771+00 to 2789+00	$0.00	$0.00	$0.00
LL-P4	2269+05	$1,080,244.57	$62,576.14	$1,142,820.71
LL-P5	2495+70	$0.00	$0.00	$0.00
LL-P6	See text	$372,818.00	$0.00	$372,818.00
LL-P7	None	$0.00	$0.00	$0.00
LL-P8	2309+25	$1,953,061.80	$62,576.14	$2,015,637.94
LL-P9	2312+40	$1,073,875.84	$62,576.14	$1,136,451.98
LL-P10	2533+72 to 2717+17	$0.00	$0.00	$0.00
LL-P11	2715+35	$2,221,200.94	$62,576.14	$2,283,777.08
LL-P12	2713+50	$0.00	$0.00	$0.00
LL-P13	2753+67	$1,305,669.94	$62,576.14	$1,368,246.08
LL-P14	2753+67	$1,970,290.46	$62,576.14	$2,032,866.60
		0	0	0
Totals for all Reaches		$209,073,067.34	$14,616,535.16	$223,689,602.50

Note- 1% AEP Relocation Items and Costs (* indicates Item ID was not used). Item IDs listed more than once in an alignment Reach have multiple impact locations within that alignment Reach. Reaches are listed in the order in which they occur in the overall alignment, from West to East. Project stationing also increases from West to East. Roads affected are in a separate table.

During the development of this Feasibility Study, it was determined that if the levee

alignment were shifted slightly in a few areas, several pipelines would be avoided. The costs in this report assume the following levee shifts will take place during P&S. 1% AEP alignment stationing is used in the following:

1) Barrier Alignment shift south, about ½ alignment width, between stations 1210+00 and 1270+00, resulting in 1st impact location for pipeline BAR-P13 changing to no impact.

2) Barrier Alignment shift south, 1 alignment width or less, between stations 1422+00 and 1640+00, resulting in-

   a. Pipeline BAR-P13- 3rd impact location changing from "shift 11,800 feet of pipeline south" to "directional drill at station 1510+00".

   b. Pipeline BAR-P13- 4th impact location changing to no impact.

   c. Pipeline BAR- P6- 2nd impact location changing to no impact.

   d. Pipeline BAR-P17- changing to no impact.

   e. Pipelines BAR-P31 through BAR P34- changing to no impact.

   f. Pipeline BAR-P22- changing to no impact.

   g. Pipeline BAR-P21- changing to needing an up-and-over relocation.

3) Barrier Alignment shift south, about ¼ alignment width, between stations 1770+00 and 1795+00, resulting in pipeline BAR-P27 changing to no impact.

4) Reach J1 alignment shift with a floodside enlargement approximately retaining protected side toe location of existing levee, resulting in pipeline J1-P1 changing to no impact.

Table 153 3% AEP Relocation Costs

Relocation Item ID	35-yr Station = 100-yr Station	Relocation Cost	Mitigation Cost	Total
Reach I-2				
I2-E1		$1,093,919.91	$83,852.03	$1,177,771.94
Reach J-2				
J2-E1		$879,857.88	$83,852.03	$963,709.91
Pipelines for all Reaches	n/a	$206,220,462.33	$14,448,831.10	$220,669,293.43
Totals for all Reaches		$208,194,240.12	$14,616,535.16	$222,810,775.28

Note- Pipelines relocations cost for 1% AEP is same as for 3% AEP (see text)

6.2.4.2   Roadways

Roadways affected by this project are as follows: LA Highway 182 (Bayou Black Drive), LA Highway 315 (Bayou Dularge Road), Four Point Road, LA Highway 56, LA Highway 55, and LA Highway 665 (Pointe Aux Chenes Road). All are assumed to have a flood gate, except LA 182 which will have a ramp. For LA 182, alternate access for locals will need to be made available during the construction of the earthen ramp, which will need to be raised each time the levee is raised.

The relocation assumptions for the roadways were as follows: 3000 feet of overhead lines (poles, electric, cable, telephone) and two underground lines (2" natural gas and 12" water lines). For the floodwalls, it includes passing the gas and water lines through a sleeve. For the ramp at LA 182, it includes directionally drilling the gas and water lines. The flood gates are not considered relocation items, but the ramp is considered a relocation item. The costs include Engineering and Design, and Supervision and Administration, but not Contingency or Escalation.

Table 154   Roadway Relocation Costs

ROADWAY	REACH	1% AEP STATION	3% AEP COST	1% AEP COST
LA HWY 182 (BAYOU BLACK DR)	BARRIER ALIGN.	1003+00	$14,894,274.03	$18,479,437.33
LA HWY 315 (BAYOU DULARGE RD)	E-2	2542+15	$631,347.78	$631,321.37
FOUR POINT ROAD	G-1	3099+20	$631,347.78	$631,231.37
LA HWY 56	H-1	3316+30	$631,347.78	$631,231.37
LA HWY 55	I-2	3737+25	$631,347.78	$631,231.37
LA HWY 665 (POINTE AUX CHENES RD)	K	4437+80	$631,347.78	$631,231.37
TOTALS-			$18,051,012.93	$21,635,684.18

7   COST

7.1   1%AEP Cost Estimate

7.1.1.1   General

The project cost estimate was developed in the TRACES MII cost estimating software and used the standard approaches for a feasibility estimate structure regarding labor, equipment, materials, crews, unit prices, quotes, sub- and prime contractor markups. This philosophy was taken wherever practical within the time constraints. It was supplemented with estimating information from other sources where necessary such as

quotes, bid data, and A-E estimates. The intent was to provide or convey a "fair and reasonable" estimate that which depicts the local market conditions. The estimates assume a typical application of tiering subcontractors. Given the long time over which this project/program is to be constructed and the unknown economic status during that time, demands from non-governmental civil works projects were not considered to dampen the competition and increase prices.

7.1.2 Estimate Structure

The estimate is structured to reflect the projects performed. The estimates are subdivided by USACE feature codes and by local "reach" name.

7.1.3 Bid competition

It is assumed that there will not be an economically saturated market and that bidding competition will be present.

7.1.4 Contract Acquisition Strategy

It is assumed that the contract acquisition strategy will be similar to past projects with some negotiated contracts, focus and preference of small business/8(a), and large, unrestricted design/bid/build contracts. There is no declared contract acquisition plan/types at this time, so typical MVN goals for small business/set-aside contracts have been included on overall cost basis by assigning approximately 25% of construction dollars to the small business/set-aside contractor type.

7.1.5 Labor Shortages

It is assumed there will be a normal labor market.

7.1.6 Labor Rates

Local labor market wages are above the local Davis-Bacon Wage Determination and actual rates have been used. This is based upon local information and payroll data received from the New Orleans District Construction Representatives and estimators with experiences in past years.

7.1.7 Cost quotes

Cost quotes are used on major construction items when available. Recent quotes may include borrow material, concrete, steel and concrete piling, rock, gravel and sand, and deep soil mixing.

7.1.8 Assumptions

7.1.8.1   Materials

Materials will be purchased as part of the construction contract. The estimate does not anticipate government furnished materials. Prices include delivery of materials.

7.1.8.2   Concrete

Concrete will be purchased from commercial batch plants.

7.1.8.3   Borrow Material and Haul

Borrow material is considered the highest risk in the contracts, given the large quantities required, uncertainties of sources and materials near the many contract locations. Specific borrow sources have not been established so a conservative estimated haul distance was used when using off-site material. Borrow pits currently in use are within this distance. All borrow material is assumed Government furnished as it is a local sponsor responsibility. NO contractor furnished borrow source are used. The borrow quantity calculations followed the MVN Geotechnical guidance.

7.1.8.4   Hauled Levee

10 BCY of borrow material = 12 LCY hauled = 8 ECY compacted.

An assumed average one-way haul distance of 25 miles for 100yr was used unless a committed borrow source has been confirmed available. This decision is based upon discussions with the New Orleans District cost engineers and MTG pdt.

Haul speeds are estimated using 40 mph speed average given the long distances and rural areas.

7.1.8.5   Rock and stone

The New Orleans delta area has no rock sources. Historically, rock is barged from northern sources on the Mississippi River. This decision is based upon local knowledge, experience and supported with cost quotes.

7.1.8.6   Rates

Rates are based on the latest USACE EP-1110-1-8, Region III. Adjustments are made for fuel and facility capital cost of money (FCCM). Judicious use of owned verses rental rates was considered based on typical contractor usage and local equipment

availability. Only a few select pieces of marine\marsh equipment are considered rental. Full FCCM/Cost of Money rate is latest available; Mii program takes EP recommended discount, no other adjustments have been made to the FCCM.

7.1.8.7   Trucking

The estimate assumed independent self-employed trucking subcontractors due to the large numbers of trucks required.

7.1.8.8   Dozers

Dozers of the D-5/D-6 variety were chosen based on historical knowledge. Heavier equipment gets mired in the mud and soft soils.

7.1.8.9   Rental Rates

Rental rates were used for various pieces of marine and marsh equipment where rental is typical such as marsh backhoes. Severe equipment rates were used where appropriate.

7.1.8.10   Fuel

Gasoline, on and off-road diesel were based on local market averages for on-road and off-road for the Gulf Coast area. The Team found that fuels fluctuate irrationally; thus, used an average.

7.1.8.11   Crews

Major crew and productivity rates were developed and studied by senior USACE estimators familiar with the type of work. All of the work is typical to the New Orleans District. The crews and productivities were checked by local MVN estimators, discussions with contractors and comparisons with historical cost data. Major crews include haul, earthwork, piling, concrete, and deep soil mixing.

7.1.8.12   Unit Prices

The unit prices found within the various project estimates will fluctuate within a range between similar construction units such as floodwall concrete, earthwork, and piling. Variances are a result of differing haul distances (trucked or barged), small or large business markups, subcontracted items, designs and estimates by others.

7.1.8.13   Relocation Cost

Relocation costs are defined as the relocation of public roads, bridges, railroads, and utilities required for project purposes. In cases where potential significant impacts were known, costs were included within the cost estimate.

7.1.9 Mobilization

Contractor mobilization and demobilization are based on the assumption that most of the contractors will be coming from within the Gulf Coast/Southern region. Mob/demob costs are based on historical studies of detailed Government estimate mob/demobs which averaged 4.9 to 5% of the construction costs.   With undefined acquisition strategies and assumed individual project limits for the large number of potential contracts in this program, the estimate utilizes a more comprehensive approx. 5% value applied at each contract rather than risking minimizing mob/demob costs by detailing costs based on an assumed number of contracts. The 5% value also matches well with the 5% value previously prescribed by Walla Walla District, which has studied historical rates.

7.1.10 Field Office Overhead

The estimate used a field office overhead rate of 12% for the prime contractors at budget level development. Based on historical studies and experience, Walla Walla District has recommended typical rates ranging from 9% to 11% for large civil works projects; however, the 9-11% rate does not consider possible incentives such as camps, allowances, travel trailers, meals, etc. which have been used previously to facilitate projects. With undefined acquisition strategies and assumed individual project limits for the large number of potential contracts in this program, the estimate utilizes a more comprehensive percentage based approach applied at each contract rather than risking minimizing overhead costs by detailing costs based on an assumed number of contracts. The applied rates were previously discussed among numerous USACE District cost engineers including Walla Walla, Vicksburg, Norfolk, Huntington, St. Paul and New Orleans.

7.1.11 Overhead Assumptions

Overhead assumptions may include superintendent, office manager, pickups, periodic travel, costs, communications, temporary offices (contractor and government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp and kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project

signs, traffic control, surveys, temp fuel tank station, generators, compressors, lighting, and minor miscellaneous.

7.1.12 Home Office Overhead

Estimate percentages range based upon consideration of 8(a), small business and unrestricted prime contractors. The rates are based upon estimating and negotiating experience, and consultation with local construction representatives. Different percents are used when considering the contract acquisition strategy regarding small business 8(a), competitive small business and large business, high to low respectively. The applied rates were previously discussed among numerous USACE District cost engineers including Walla Walla, Vicksburg, Norfolk, Huntington, St. Paul and New Orleans.

7.1.13 Taxes

Local taxes will be applied, using an average between the parishes that contain the work. Reference the LA parish tax rate website: http-//www.laota.com/pta.htm

7.1.14 Bond

Bond is assumed 1% applied against the prime contractor, assuming large contracts. No differentiation was made between large and small businesses.

7.1.15 E&D and S&A

USACE Costs to manage design (PED) and construction (S&A) are based on New Orleans District Programmatic Cost Estimate guidance-

7.1.16 Planning, Engineering & Design (PED)

The PED cost includes such costs as project management, engineering, planning, designs, investigations, studies, reviews, value engineering and engineering during construction (EDC). Historically New Orleans District has used an approximate 12% rate for E&D/EDC, applied against the estimated construction costs. Other USACE civil works districts such as St. Paul, Memphis and St. Louis have reported values ranging from 10-15%. Additional costs were added for project management, engineering, planning, designs, investigations, studies, reviews, value engineering.

7.1.17 Supervision & Administration (S&A)

Historically, New Orleans District used a range from 5% to 15% depending on project size and type applied against the estimated construction costs. Other USACE civil

works districts such as St. Paul, Memphis and St. Louis report values ranging from 7.5-10%. Consideration includes that a portion of the S&A effort could be performed by contractors. Based on discussions with MVN Construction Division, an S&A cost based on contract durations was developed rather than applying a percentage.

7.1.18 Contingencies

Contingencies were developed using the USACE Cost and Schedule Risk Analysis (CSRA) process and the Crystal Ball software that evaluates schedule and cost related risks. See summary in Cost Schedule Risk Analysis (CSRA) section.

7.1.19 Escalation

Escalation used in the TPCS is based upon the US Army Corps of Engineers Engineering Manual (EM) 1110-2-1304 Civil Works Construction Cost Index System (CWCCIS) revised 30 Mar 2012.

7.1.20 HTRW

The estimate includes no costs for any potential Hazardous, Toxic, and Radioactive Waste (HTRW) concerns. Phase 1 HTRW investigations are already complete and the result of this investigation is that no further investigation is recommended.

7.1.21 Schedule

The project schedule was developed based on the construction of the individual features of work to include the entire 1% AEP Morganza to the Gulf program which includes construction of earthen levees, floodwalls, floodgates, and other structures along a 98-mile alignment south of Houma. The alignment is sub-divided into 13 main reaches (Barrier, A, B, E, F, G, H, I, J, K, L, Larose C-North, and Lockport to Larose). Final levee elevations vary from +15 to +26.5 feet NAVD88 and structure elevations range from +17 to +33 ft NAVD88. Structures include a multi-purpose lock, 22 navigable floodgates, 23 environmental water control structures, 9 road / RR gates, and fronting protection for 4 existing pumping stations. The structures located on Federally maintained navigation channels include a 110-ft wide by 800-ft long lock with an adjacent 250-ft wide sector gate on the Houma Navigation Canal and two 125-ft sector gates on the GIWW east and west of Houma. Fourteen 56-ft sector gates and five 20-to 30-ft stop log gates are located on various waterways that cross the levee system.-

The team focused the earliest construction efforts on the areas most vulnerable to storm surge inundation. Since we received authority to proceed to PED for the HNC Lock Complex ahead of the remainder of the project, we gave first priority to the HNC Lock complex, tie-in levees, and the adjacent Bayou Grand Caillou structure. From there, a

levee lift schedule was laid out that allows for a minimum of 3 years of settlement between levee lifts, minimizes adjacent/conflicting work zones, reduces the cost of interest during construction, and delivers the specified level of risk reduction (1%) by the project base year and maintains that level 50-years into the future.

7.2 3% AEP

7.2.1 General

7.2.1.1 Cost estimate development

The project cost estimate was developed in the TRACES MII cost estimating software and used the standard approaches for a feasibility estimate structure regarding labor, equipment, materials, crews, unit prices, quotes, sub- and prime contractor markups. This philosophy was taken wherever practical within the time constraints. It was supplemented with estimating information from other sources where necessary such as quotes, bid data, and A-E estimates. The intent was to provide or convey a "fair and reasonable" estimate that which depicts the local market conditions. The estimates assume a typical application of tiering subcontractors. Given the long time over which this project/program is to be constructed and the unknown economic status during that time, demands from non-governmental civil works projects were not considered to dampen the competition and increase prices.

7.2.1.2 Estimate Structure

The estimate is structured to reflect the projects performed. The estimates are subdivided by USACE feature codes and by local "reach" name.

7.2.1.3 Bid Competition

It is assumed that there will not be an economically saturated market and that bidding competition will be present.

7.2.2 Contract Acquisition Strategy

It is assumed that the contract acquisition strategy will be similar to past projects with some negotiated contracts, focus and preference of small business/8(a), and large, unrestricted design/bid/build contracts. There is no declared contract acquisition plan/types at this time, so typical MVN goals for small business/set-aside contracts have been included on overall cost basis by assigning approximately 25% of construction dollars to the small business/set-aside contractor type.

7.2.3 Labor Shortages

It is assumed there will be a normal labor market.

7.2.4 Labor Rates

Local labor market wages are above the local Davis-Bacon Wage Determination and actual rates have been used. This is based upon local information and payroll data received from the New Orleans District Construction Representatives and estimators with experiences in past years.

7.2.5 Materials

Cost quotes are used on major construction items when available. Recent quotes may include borrow material, concrete, steel and concrete piling, rock, gravel and sand, and deep soil mixing. Assumptions include materials will be purchased as part of the construction contract. The estimate does not anticipate government furnished materials. Prices include delivery of materials. Concrete will be purchased from commercial batch plants.

7.2.6 Borrow Material and Haul

Borrow material is considered the highest risk in the contracts, given the large quantities required, uncertainties of sources and materials near the many contract locations. Specific borrow sources have not been established so a conservative estimated haul distance was used when using off-site material. Borrow pits currently in use are within this distance. All borrow material is assumed Government furnished as it is a local sponsor responsibility. NO contractor furnished borrow source are used. The borrow quantity calculations followed the MVN Geotechnical guidance.

7.2.7 Hauled Levee

10 BCY of borrow material = 12 LCY hauled = 8 ECY compacted. An assumed average one-way haul distance of 20 miles for 35yr was used unless a committed borrow source has been confirmed available. This decision is based upon discussions with the New Orleans District cost engineers and MTG pdt. Haul speeds are estimated using 40 mph speed average given the long distances and rural areas.

7.2.8 Quantities

Levee degrade qtys were determined by the levee design sections. The 1st lift is dual hatting as a preload and an initial lift. The ultimate design section requires the preload/1st lift to be degraded to approx elev 4.0 and install reinforcement geotextile

prior to adding the 2nd lift. 2nd lift qty assumes construction begins at the pre-degraded 1st lift elevation. A 20% "loss" or really non-available is allowed (probably on the conservative side), but is meant more to express the resulting amount of in-place material that is felt garunteed to be credited to the re-built section (back to pre-degrade elev) and to be sure that the section used for the "existing" condition prior to the 2nd lift qty is actually there. There may be slightly different compaction levels when replaced, the material will be stockpiled nearby and some will be left behind, and some will be misplaced in the transfer out and back into the levee as the stockpiles may not be exactly adjacent to the new placement area. The lost material will be replaced with new material.

7.2.9 Rock and stone

The New Orleans delta area has no rock sources. Historically, rock is barged from northern sources on the Mississippi River. This decision is based upon local knowledge, experience and supported with cost quotes.

7.2.10 Equipment

Rates used are based from the latest USACE EP-1110-1-8, Region III. Adjustments are made for fuel and facility capital cost of money (FCCM). Judicious use of owned verses rental rates was considered based on typical contractor usage and local equipment availability. Only a few select pieces of marine \ marsh equipment are considered rental. Full FCCM/Cost of Money rate is latest available; Mii program takes EP recommended discount, no other adjustments have been made to the FCCM.

7.2.11 Trucking

The estimate assumed independent self-employed trucking subcontractors due to the large numbers of trucks required.

7.2.12 Dozers

Dozers of the D-5/D-6 variety were chosen based on historical knowledge. Heavier equipment gets mired in the mud and soft soils.

7.2.13 Severe Rates

Severe equipment rates were used for various pieces of equipment in the hydraulic dredging crews where they may come in contact with a saltwater environment.

7.2.14 Rental Rates

Rental rates were used for various pieces of marine and marsh equipment where rental is typical such as marsh backhoes.

7.2.15 Fuels

Fuels (gasoline, on and off-road diesel) were based on local market averages for on-road and off-road for the Gulf Coast area. The Team found that fuels fluctuate irrationally; thus, used an average.

7.2.16 Crews

Major crew and productivity rates were developed and studied by senior USACE estimators familiar with the type of work. All of the work is typical to the New Orleans District. The crews and productivities were checked by local MVN estimators, discussions with contractors and comparisons with historical cost data. Major crews include haul, earthwork, piling, concrete, and deep soil mixing.

Most crew work hours are assumed to be 10 hrs 6 days/wk which is typical to the area. Marine based bucket excavation/dredging operations for levee construction are assumed to work 2-12 hours shifts 7 days / week.

A 10% "markup on labor for weather delay" is selectively applied to the labor in major earthwork placing detail items and associated items that would be affected by small amounts of weather making it unsafe or difficult to place (trying to run dump trucks on a wet levee) or be detrimental/non-compliant to the work being done (trying to place/compact material in the rain). The 10% markup is to cover the common practice of paying for labor "showing up" to the job site and then being sent home due to minor weather which is part of known average weather impacts as reflected within the standard contract specifications. The markup was not applied to small quantities where this can be scheduled around.

7.2.17 Unit Prices

The unit prices found within the various project estimates will fluctuate within a range between similar construction units such as floodwall concrete, earthwork, and piling. Variances are a result of differing haul distances (trucked or barged), small or large business markups, subcontracted items, designs and estimates by others.

7.2.18 Relocation Costs

Relocation costs are defined as the relocation of public roads, bridges, railroads, and utilities required for project purposes. In cases where potential significant impacts were known, costs were included within the cost estimate.

7.2.19 Mobilization

Contractor mobilization and demobilization are based on the assumption that most of the contractors will be coming from within the Gulf Coast/Southern region. Mob/demob costs are based on historical studies of detailed Government estimate mob/demobs which averaged 4.9 to 5% of the construction costs. With undefined acquisition strategies and assumed individual project limits for the large number of potential contracts in this program, the estimate utilizes a more comprehensive approx. 5% value applied at each contract rather than risking minimizing mob/demob costs by detailing costs based on an assumed number of contracts. The 5% value also matches well with the 5% value previously prescribed by Walla Walla District, which has studied historical rates.

7.2.20 Field Office Overhead

The estimate used a field office overhead rate of 12% for the prime contractors at budget level development. Based on historical studies and experience, Walla Walla District has recommended typical rates ranging from 9% to 11% for large civil works projects; however, the 9-11% rate does not consider possible incentives such as camps, allowances, travel trailers, meals, etc. which have been used previously to facilitate projects. With undefined acquisition strategies and assumed individual project limits for the large number of potential contracts in this program, the estimate utilizes a more comprehensive percentage based approach applied at each contract rather than risking minimizing overhead costs by detailing costs based on an assumed number of contracts. The applied rates were previously discussed among numerous USACE District cost engineers including Walla Walla, Vicksburg, Norfolk, Huntington, St. Paul and New Orleans.

7.2.21 Overhead Assumptions

Overhead assumptions may include superintendent, office manager, pickups, periodic travel, costs, communications, temporary offices (contractor and government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp and kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project

signs, traffic control, surveys, temp fuel tank station, generators, compressors, lighting, and minor miscellaneous.

7.2.22 Home Office Overhead

Estimate percentages range based upon consideration of 8(a), small business and unrestricted prime contractors. The rates are based upon estimating and negotiating experience, and consultation with local construction representatives. Different percents are used when considering the contract acquisition strategy regarding small business 8(a), competitive small business and large business, high to low respectively. The applied rates were previously discussed among numerous USACE District cost engineers including Walla Walla, Vicksburg, Norfolk, Huntington, St. Paul and New Orleans.

7.2.23 Taxes

Local taxes will be applied, using an average between the parishes that contain the work. Reference the LA parish tax rate website: http-//www.laota.com/pta.htm

7.2.24 Bond

Bond is assumed 1% applied against the prime contractor, assuming large contracts. No differentiation was made between large and small businesses.
E&D and S&A: USACE Costs to manage design (PED) and construction (S&A) are based on New Orleans District Programmatic Cost Estimate guidance-

7.2.25 Planning, Engineering & Design (PED)

The PED cost includes such costs as project management, engineering, planning, designs, investigations, studies, reviews, value engineering and engineering during construction (EDC). Historically New Orleans District has used an approximate 12% rate for E&D/EDC, applied against the estimated construction costs. Other USACE civil works districts such as St. Paul, Memphis and St. Louis have reported values ranging from 10-15%. Additional costs were added for project management, engineering, planning, designs, investigations, studies, reviews, value engineering.

7.2.26 Supervision & Administration (S&A)

Historically, New Orleans District used a range from 5% to 15% depending on project size and type applied against the estimated construction costs. Other USACE civil works districts such as St. Paul, Memphis and St. Louis report values ranging from 7.5-10%. Consideration includes that a portion of the S&A effort could be performed by contractors.

Based on discussions with MVN Construction Division, an S&A cost based on contract durations was developed rather than applying a percentage.

7.2.27 Contingencies

Contingencies were developed using the USACE Cost and Schedule Risk Analysis (CSRA) process and the Crystal Ball software that evaluates schedule and cost related risks. See summary in Cost Schedule Risk Analysis (CSRA) section.

7.2.28 Escalation

Escalation used in the TPCS is based upon the US Army Corps of Engineers Engineering Manual (EM) 1110-2-1304 Civil Works Construction Cost Index System (CWCCIS) revised 30 Mar 2012.

7.2.29 HTRW

The estimate includes no costs for any potential Hazardous, Toxic, and Radioactive Waste (HTRW) concerns. Phase 1 HTRW investigations are already complete and the result of this investigation is that no further investigation is recommended.

7.2.30 Schedule

The 3% AEP project schedule was developed based on the construction of the individual features of work to include the entire Morganza to the Gulf program which includes construction of earthen levees, floodwalls, floodgates, and other structures along a 98-mile alignment south of Houma. The alignment is sub-divided into 13 main reaches (Barrier, A, B, E, F, G, H, I, J, K, L, Larose C-North, and Lockport to Larose). Final levee elevations vary from +12 to +20 feet NAVD88 and structure elevations range from +14 to +25 ft NAVD88. Structures include a multi-purpose lock, 22 navigable floodgates, 23 environmental water control structures, 9 road/RR gates, and fronting protection for 4 existing pumping stations. The structures located on Federally maintained navigation channels include a 110-ft wide by 800-ft long lock with an adjacent 250-ft wide sector gate on the Houma Navigation Canal and two 125-ft sector gates on the GIWW east and west of Houma. Fourteen 56-ft sector gates and five 20- to 30-ft stop log gates are located on various waterways that cross the levee system.

The team focused the earliest construction efforts on the areas most vulnerable to storm surge inundation. Since we received authority to proceed to PED for the HNC Lock Complex ahead of the remainder of the project, we gave first priority to the HNC Lock complex, tie-in levees, and the adjacent Bayou Grand Caillou structure. From there, a levee lift schedule was laid out that allows for a minimum of 3 years of settlement

between levee lifts, minimizes adjacent/conflicting work zones, reduces the cost of interest during construction, and delivers the specified level of risk reduction (3%) by the project base year and maintains that level 50-years into the future.

7.3 Cost Estimate

Table 155 through 160 show the baseline project cost, October 2012 cost and fully funded project cost for the 1% AEP and 3% AEP. This information is taken from the Total Project Cost Sheet (TPCS).

Table 155   1% AEP Baseline Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$282,437,000	$72,391,000	$354,828,000
02 Relocations	$230,710,000	$59,985,000	$290,695,000
05 Locks	$460,439,000	$161,154,000	$621,593,000
06 Fish & Wildlife Facilities	$719,794,000	$221,419,000	$941,213,000
11 Levees & Floodwalls	$4,225,556,000	$1,125,924,000	$5,351,000,000
15 Fldwy Control & Div Str	$791,153,000	$276,904,000	$1.068,057,000
30 PED	$780,935,000	$225,254,000	$1,006,189,000
31 Construction Management	$489,774,000	$141,271,000	$631,045,000
TOTAL	$7,980,798,000	$2,284,302,000	$10,265,100,000

Table 156   1% AEP Oct 2012 Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$282,437,000	$72,391,000	$354,828,000
02 Relocations	$230,710,000	$59,985,000	$290,695,000
05 Locks	$460,439,000	$161,154,000	$621,593,000
06 Fish & Wildlife Facilities	$719,794,000	$221,419,000	$941,213,000
11 Levees & Floodwalls	$4,225,556,000	$1,125,924,000	$5,351,000,000
15 Fldwy Control & Div Str	$791,153,000	$276,904,000	$1.068,057,000
30 PED	$780,935,000	$225,254,000	$1,006,189,000
31 Construction Management	$489,774,000	$141,271,000	$631,045,000
TOTAL	$7,980,798,000	$2,284,302,000	$10,265,100,000

Table 157   1% AEP Fully Funded Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$290,073,000	$74,331,000	$364,404,000
02 Relocations	$246,753,000	$64,156,000	$310,908,000
05 Locks	$500,269,000	$175,094,000	$675,363,000
06 Fish & Wildlife Facilities	$830,079,000	$254,747,000	$1,084,826,000
11 Levees & Floodwalls	$5,475,122,000	$1,460,444,000	$6,935,566,000
15 Fldwy Control & Div Str	$914,620,000	$320,117,000	$1,234,738,000

Feature	Cost	Contingency	Total
30 PED	$1,047,988,000	$303,076,000	$1,351,064,000
31 Construction Management	$710,533,000	$205,443,000	$915,976,000
TOTAL	$10,015,438,000	$2,857,408,000	$12,872,846,000

Table 158   3% AEP Baseline Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$270,609,000	$68,196,000	$338,805,000
02 Relocations	$226,246,000	$47,512,000	$273,758,000
05 Locks	$407,410,000	$122,223,000	$529,633,000
06 Fish & Wildlife Facilities	$493,352,000	$125,462,000	$618,814,000
11 Levees & Floodwalls	$2,021,674,000	$445,149,000	$2,466,823,000
15 Fldwy Control & Div Str	$587,261,000	$176,178,000	$763,439,000
30 PED	$459,946,000	$113,901,000	$763,439,000
31 Construction Management	$308,568,000	$76,413,000	$384,981,000
TOTAL	$4,775,066,000	$1,175,035,000	$5,950,101,000

Table 159   3% AEP October 2012 Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$270,609,000	$68,196,000	$338,805,000
02 Relocations	$226,246,000	$47,512,000	$273,758,000
05 Locks	$407,410,000	$122,223,000	$529,633,000
06 Fish & Wildlife Facilities	$493,352,000	$125,462,000	$618,814,000
11 Levees & Floodwalls	$2,021,674,000	$445,149,000	$2,466,823,000
15 Fldwy Control & Div Str	$587,261,000	$176,178,000	$763,439,000
30 PED	$459,946,000	$113,901,000	$763,439,000
31 Construction Management	$308,568,000	$76,413,000	$384,981,000
TOTAL	$4,775,066,000	$1,175,035,000	$5,950,101,000

Table 160   3% AEP Fully Funded Cost

Feature	Cost	Contingency	Total
01 Lands & Damages	$278,009,000	$70,055,000	$348,064,000
02 Relocations	$245,484,000	$51,552,000	$297,036,000
05 Locks	$430,939,000	$129,282,000	$560,220,000
06 Fish & Wildlife Facilities	$588,555,000	$148,415,000	$736,971,000
11 Levees & Floodwalls	$2,546,699,000	$563,675,000	$3,110,374,000
15 Fldwy Control & Div Str	$684,388,000	$205,316,000	$889,704,000
30 PED	$611,686,000	$152,197,000	$763,883,000
31 Construction Management	$431,494,000	$107,332,000	$538,826,000
TOTAL	$5,817,255,000	$1,427,825,000	$7,245,080,000

8   DESIGN AND CONSTRUCTION SCHEDULE

The detailed design and construction schedules for the 1% AEP and 3% AEP are available upon request.

<u>REFERENCES</u>
(**All Listed References Are Available Upon Request**)

**MORGANZA TO THE GULF OF MEXICO, LOUISIANA DRAFT PAC
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1 of 2 Reach F-2 & F-1 Plan View Station 2788+92.72 to Station 2863+83
2 of 2 Reach F-1 and G-1 Plan View Station 2863+83 to Station 3007+34.70
1 of 3 Reach G-1 Plan View Station 3007+34.70 to Station 3087+35
2 of 3 Reach G-1, G-2 Plan View Station 3087+35 to Station 3162+35
3 of 3 Reach G-1 & G-2 Plan View Station 3087+35 to Station 3162+35
1 of 4 Reach H-1 Plan View Station 3224+12.12 to Station 3313+12
2 of 4 Reach H-1 & H-2 Plan View Station 3313+12 to Station 3416+12
3 of 4 Reach H-2 & H-3 Plan View Station 3416+12 to Station 3531+12
4 of 4 Reach H-3 Plan View Station 3531+12 to Station 3640+67
1 of 3 Reach I-1 & I-2 Plan View Station 3640+67 to Station 3750+65
2 of 3 Reach I-2 & I-3 Plan View Station 3750+65 to Station 3855+70
3 of 3 Reach I-3 Plan View Station 3855+70 to Station 3941+76
1 of 5 Reach J-2 Plan View Station 3941+75.55 to Station 4048+75
2 of 5 Reach J-2 Plan View Station 4048+75 to Station 4156+69
3 of 5   Reach J-2 & J-1 Plan View Station 4156+69 to Station 4261+76
4 of 5 Reach J-1 Plan View Station 4261+76 to Station 4367+74
5 of 5 Reach J-1 Plan View Station 4367+74 to Station 4438+85.25
1 of 3 Reach K Plan View Station 4438+85+25 to Station 4543+85
2 of 3 Reach K Plan View Station 4543+85 to Station 4658+86
3 of 3 Reach K Plan View Station 4658+86 to Station 4706+98.84
1 of 3 Reach L Plan View Station 4706+98.84 to Station 4812+00
2 of 3 Reach L Plan View Station 4812+00 to Station 4942+00
3 of 3 Reach L Plan View Station 4942+00 to Station 5021+78
M2G-L-01   Plan Sta 2160+70 to Sta 2245+00
M2G-L-02   Plan Sta 2245+00 to Sta 2220+00
M2G-L-03   Plan Sta 2320+00 to Sta 2400+00
M2G-L-04   Plan Sta 2400+00 to Sta 2520+00
M2G-L-05   Plan Sta 2520+00 to Sta 2640+00
M2G-L-06   Plan Sta 2640+00 to Sta 2760+00
M2G-L-07   Plan Sta 2760+00 to Sta 2816+79
M2G-L-08   Plan Sta 1042+90.18 to Sta 925+48.62
M2G-L-09   Plan Sta 925+48.62 to Sta 798+31.98

Civil Plates 3% Design

1 of 5 Reach A Plan View Station 1828+22.13 to Station 1907+62
2 of 5 Reach A Plan View Station 1907+62 to Station 2032+60
3 of 5 Reach A Plan View Station 2032+60 to Station 2073+25
4 of 5 Reach A Plan View Station 2073+25 to Station 2183+22
5 of 5 Reach A Plan View Station 2183+22 to Station 2259+26.41
1 of 3 Reach B Plan View Station 2259+26.41 to Station 2364+25
2 of 3 Reach B Plan View Station 2364+25 to Station 2469+26
3 of 3 Reach B Plan View Station 2469+26 to Station 2526+35.62

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1 of 8 Barrier Alignment Plan View Station 1000+00 to Station 1075+00
2 of 8 Barrier Alignment Plan View Station 1075+00 to Station 1190+00
3 of 8 Barrier Alignment Plan View Station 1190+00 to Station 1305+00
4 of 8 Barrier Alignment Plan View Station 1305+00 to Station 1415+17
5 of 8 Barrier Alignment Plan View Station 1415+17 to Station 1530+17
6 of 8 Barrier Alignment Plan View Station 1530+17 to Station 1630+17
7 of 8 Barrier Alignment Plan View Station 1630+17 to Station 1745+17
8 of 8 Barrier Alignment Plan View Station 1745+17 to Station 1828+22.13
3 of 4 Reach E-2 & E-1 Plan View Station 2621+34 to Station 2726+33
4 of 4 Reach E-1 and F-2 Plan View Station 2726+33 to Station 2788+92.72
1 of 4 Reach E2 Plan View Station 2526+62 to Station 2571+43
2 of 4 Reach E-2 Plan View Station 2571+43 to Station 2621+34
1 of 2 Reach F-2 & F-1 Plan View Station 2788+92.72 to Station 2863+83
2 of 2 Reach F-1 and G-1 Plan View Station 2863+83 to Station 3007+34.70
1 of 3 Reach G-1 Plan View Station 3007+34.70 to Station 3087+35
2 of 3 Reach G-1, G-2 Plan View Station 3087+35 to Station 3162+35
3 of 3 Reach G-1 & G-2 Plan View Station 3087+35 to Station 3162+35
1 of 4 Reach H-1 Plan View Station 3224+12.12 to Station 3313+12
2 of 4 Reach H-1 & H-2 Plan View Station 3313+12 to Station 3416+12
3 of 4 Reach H-2 & H-3 Plan View Station 3416+12 to Station 3531+12
4 of 4 Reach H-3 Plan View Station 3531+12 to Station 3640+67
1 of 3 Reach I-1 & I-2 Plan View Station 3640+67 to Station 3750+65
2 of 3 Reach I-2 & I-3 Plan View Station 3750+65 to Station 3855+70
3 of 3 Reach I-3 Plan View Station 3855+70 to Station 3941+76
1 of 5 Reach J-2 Plan View Station 3941+75.55 to Station 4048+75
2 of 5 Reach J-2 Plan View Station 4048+75 to Station 4156+69
3 of 5   Reach J-2 & J-1 Plan View Station 4156+69 to Station 4261+76
4 of 5 Reach J-1 Plan View Station 4261+76 to Station 4367+74
5 of 5 Reach J-1 Plan View Station 4367+74 to Station 4438+85.25
1 of 3 Reach K Plan View Station 4438+85+25 to Station 4543+85
2 of 3 Reach K Plan View Station 4543+85 to Station 4658+86
3 of 3 Reach K Plan View Station 4658+86 to Station 4706+98.84
1 of 3 Reach L Plan View Station 4706+98.84 to Station 4812+00
2 of 3 Reach L Plan View Station 4812+00 to Station 4942+00
3 of 3 Reach L Plan View Station 4942+00 to Station 5021+78
M2G-L-01   Plan Sta 2160+70 to Sta 2245+00
M2G-L-02   Plan Sta 2245+00 to Sta 2220+00
M2G-L-03   Plan Sta 2320+00 to Sta 2400+00
M2G-L-04   Plan Sta 2400+00 to Sta 2520+00
M2G-L-05   Plan Sta 2520+00 to Sta 2640+00
M2G-L-06   Plan Sta 2640+00 to Sta 2760+00
M2G-L-07   Plan Sta 2760+00 to Sta 2816+79
M2G-L-08   Plan Sta 1042+90.18 to Sta 925+48.62

MORGANZA TO THE GULF OF MEXICO, LOUISIANA DRAFT PAC
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M2G-L-09   Plan Sta 925+48.62 to Sta 798+31.98

Levee Section Plates

A   Levee Reach A and Barrier Alignment 100 Yr LORR Base Yr Conditions with Overbuild Section
B   Levee Reach B 100 Yr LORR Base Yr Conditions with Overbuild Section
E   Levee Reach E 100 Yr LORR Base Yr Conditions with Overbuild Section
F   Levee Reach F 100 Yr LORR Base Yr Conditions with Overbuild Section
G   Levee Reach G 100 Yr LORR Base Yr Conditions with Overbuild Section
H1   Levee Reach H1 100 Yr LORR Base Yr Conditions with Overbuild Section
H2   Levee Reach H2 & H3 100 Yr LORR Base Yr Conditions with Overbuild Section
I   Levee Reach I 100 Yr LORR Base Yr Conditions with Overbuild Section
J3   Levee Reach J3 100 Yr LORR Base Yr Conditions with Overbuild Section
J   Levee Reach J 100 Yr LORR Base Yr Conditions with Overbuild Section
KL   Levee Reach K & L 100 Yr LORR Base Yr Conditions with Overbuild Section
SWL-A Lockport to Larose Reach (a) 100 Yr LORR Base Yr Conditions with Overbuild Section

Structures Plates

C-701 Bayou Black Canal 56' Sector Gate Plan Phase 1 Construction
C-702 Bayou Black Canal 56' Sector Gate Plan Phase 2 Construction
C-703 Black Bayou 56' Sector Gate Plan Final Site Plan
C-703A Black Bayou 56' Sector Gate 100 Yr Final Site Plan
C-704   Black Bayou Canal 56' Sector Gate Cross Sections
C-705 Black Bayou Canal 56' Sector Gate Cross Sections
C-707 Shell Canal West 30' Stop Log Gate Plan Phase 1 Construction
C-708 Shell Canal West 30' Stop Log Gate Plan Phase 2 Construction
C709   Shell Canal West 30' Stop Log Gate Plan Final Plan
C-709AShell Canal West 30' Stop Log Gate 100 Yr Final Plan
C-710   Shell Canal West 30' Stop Log Gate Cross Sections
C-711   Shell Canal West 35' Sector Gate Cross Sections
C-713   Shell Canal East 56' Sector Gate Plan Phase 1
C-714   Shell Canal East 56' Sector Gate Plan Phase 2
C-715   Shell Canal East 56' Sector Gate Final Plan
C-715A   Shell Canal East 56' Sector Gate 100 Yr Final Plan
C-716   Shell Canal East 56' Sector Gate Cross Sections
C-717   Shell Canal East 56' Sector Gate Cross Sections
C-719   Elliot Jones 20' Stop Log Gate Plan Phase 1 Construction
C-720   Elliot Jones 20' Stop Log Gate Plan Phase 2 Construction
C-721   Elliot Jones 20' Stop Log Gate Final Site Plan
C-721A   Elliot Jones 20' Stop Log Gate 100 Yr Final Site Plan
C-722   Elliot Jones 20' Stop Log Gate Cross Sections
C-723   Elliot Jones 20' Stop Log Gate Cross Sections

C-767	Bayou Grand Caillou 56' Sector Gate Cross Sections
C-768	Bayou Grand Caillou 56' Sector Gate Cross Sections
C-770	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Swing Gate Plan Phase 1 Construction
C-771	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Swing Gate Plan Phase 2 Construction
C-772	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Relocation Final Site Plan
C-772A	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Relocation 100 Yr Final Site Plan
C-773	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Relocation Cross Sections
C-774	Bayou Four Points 30' Stop Log Gate Plan & Four Point RD Relocation Cross Sections
C-779	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate Plan Phase 1 Construction
C-780	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate Plan Phase 2 Construction
C-781	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate Final Site Plan
C-781A	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate 100 Yr Final Site Plan
C-782	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate Cross Sections
C-783	Bayou Petit Caillou 56' Sector Gate & Hwy 56 Swing Gate Cross Sections
C-788	Placid Canal 56' Sector Gate Plan Phase 1 Construction
C-789	Placid Canal 56' Sector Gate Plan Phase 2 Construction
C-790	Placid Canal 56' Sector Gate Final Site Plan
C-790A	Placid Canal 56' Sector Gate 100 Yr Final Site Plan
C-791	Placid Canal 56' Sector Gate Cross Sections
C-792	Placid Canal 56' Sector Gate Cross Sections
C-794	Bush Canal 56' Sector Gate Plan Phase 1 Construction
C-795	Bush Canal 56' Sector Gate Plan Phase 2 Construction
C-796	Bush Canal 56' Sector Gate Final Site Plan
C-796A	Bush Canal 56' Sector Gate 100 Yr Final Site Plan
C-797	Bush Canal 56' Sector Gate Cross Sections
C-798	Bush Canal 56' Sector Gate Cross Sections
C-800	Bayou Terrebonne 56' Sector Gate & Hwy 55 Swing Gate Plan Phase 1 Construction
C-801	Bayou Terrebonne 56' Sector Gate & Hwy 55 Swing Gate Plan Phase 2 Construction
C-802	Bayou Terrebonne 56' Sector Gate & Hwy 55 Swing Gate Final Site Plan
C-802A	Bayou Terrebonne 56' Sector Gate & Hwy 55 Swing Gate 100 Yr Final Site Plan
C-803	Bayou Terrebonne 56' Sector Gate & Hwy 55 Swing Gate Cross Sections

C-842A   Environmental Control Structure Reach E Final Site Plan 100 Yr
C-843   Environmental Control Structure Reach E Cross Sections
C-844   Environmental Control Structure Reach G Final Site Plan
C-844A   Environmental Control Structure Reach G Final Site Plan 100 Yr
C-845   Environmental Control Structure Reach G Cross Sections
C-846   Environmental Control Structure Reach J Final Site Plan
C-846A   Environmental Control Structure Reach J Final Site Plan 100 Yr
C-847   Environmental Control Structure Reach J Cross Sections
C-848   Environmental Control Structure Reach K&L Final Site Plan
C-848A   Environmental Control Structure Reach K&L Final Site Plan 100 Yr
C-849   Environmental Control Structure Reach K&L Cross Sections
C-850   Environmental Control Structure Reach H Final Site Plan
C-850A   Environmental Control Structure Reach H Final Site Plan 100 Yr
C-851   Environmental Control Structure Reach H Cross Sections
C-852   Environmental Control Structure Reach A Final Site Plan
C-852A   Environmental Control Structure Reach A Final Site Plan 100 Yr
C-853   Environmental Control Structure Reach A Cross Sections
C-861   Black Bayou Pump Station Fronting Protection Final Site Plan
C-862   Black Bayou Pump Station Fronting Protection Cross Sections
C-863   Hanson Canal Pump Station Fronting Protection Final Site Plan
C-864   Hanson Canal Pump Station Fronting Protection Cross Sections
C-865   Madison Pump Station Fronting Protection Final Site Plan
C-866   Madison Canal Pump Station Fronting Protection Cross Sections
C-867   Point Aux Chenes Pump Station Fronting Protection Final Site Plan
C-868   Point Aux Chenes Pump Station Fronting Protection Cross Sections
C-900   Environmental Control Structure Larose to Lockport #1 – Final Site Plan 0
35 Year LORR
C-901   Environmental Control Structure Larose to Lockport #1 – Final Site Plan 0
100 Year LORR
C-902   Environmental Control Structure Larose to Lockport #2 – Final Site Plan 0
35 Year LORR
C-903   Environmental Control Structure Larose to Lockport #2 – Final Site Plan 0
100 Year LORR
C-904   Environmental Control Structure Larose to Lockport Cross Sections
C-905   Union Pacific Railroad Crossing Final Site Plan – 35 Year LORR
C-906   Union Pacific Railroad Crossing Final Site Plan – 100 Year LORR
C-907   Union Pacific Railroad Crossing Cross Sections
C-908   Gulf South Pipeline Crossing Final Site Plan – 35 Year LORR
C-909   Gulf South Pipeline Crossing Final Site Plan – 100 Year LORR
C-910   American Midstream Pipeline Crossing Final Site Plan – 35 Year LORR
C-911   American Midstream Pipeline Crossing Final Site Plan – 100 Year LORR
C-912   Williams Discovery Pipeline Crossing Final Site Plan – 35 Year LORR
C-913   Williams Discovery Pipeline Crossing Final Site Plan – 100 Year LORR

C-914	Larose C-North Utility Crossings Cross Sections
C-920	Larose FG 56' Sector Gate Phase 1 Construction
C-921	Larose FG 56' Sector Gate Phase 2 Construction
C-922	Larose FG 56' Sector Gate Final Site Plan View
C-923	Larose FG 56' Sector Gate Cross Sections
S-001	56' Sector Gate Isometric
S-001A	56' Sector Gate Isometric
S-002	56' Sector Gate Masonry Plan
S-002A	56' Sector Gate Masonry Plan
S-003	56' Sector Gate Masonry Elevation Transverse
S-003A	56' Sector Gate Masonry Elevation Transverse
S-004	56' Sector Gate Masonry Elevation Longitudinal
S-004A	56' Sector Gate Masonry Elevation Longitudinal
S-005	56' Sector Gate Foundation Plan
S-005A	56' Sector Gate Foundation Plan
S-006	56' Sector Gate Upper Frame
S-006A	56' Sector Gate Upper Frame
S-007	56' Sector Gate Upper Middle Frame
S-007A	56' Sector Gate Upper Middle Frame
S-008	56' Sector Gate Middle Lower Frame
S-008A	56' Sector Gate Middle Frame
S-009	56' Sector Gate Lower Frame
S-009A	56' Sector Gate Middle Lower Frame
S-010A	56' Sector Gate Lower Frame
S-011	56' Sector Gate Channel Truss
S-011A	56' Sector Gate Channel Truss
S-012	56' Sector Gate Recess Truss
S-012A	56' Sector Gate Recess Truss
S-013	56' Sector Gate Fender System
S-013A	56' Sector Gate Fender System
S-014	56' Sector Gate Hinge Assembly and Details 1 of 2
S-014A	56' Sector Gate Hinge Assembly and Details 1 of 2
S-015	56' Sector Gate Hinge Assembly and Details 2 of 2
S-015A	56' Sector Gate Hinge Assembly and Details 2 of 2
S-016	56' Sector Gate Pintle Assembly and Details
S-016A	56' Sector Gate Pintle Assembly and Details
S-018	56' Needle Girder and Support Tower
S-019	Steel Needle Plan, Elevation and Section
S-020	56' Sector Gate Needle Girder Storage Platform
S-021	56' Sector Gate Guidewall Plan
S-022	56' Sector Gate Guidewall Sections and Details
S-023	56' Sector Gate 7 Pile Cluster
S-024	56' Sector Gate Cofferdam

S-025	175' Sector Gate Isometric
S-026	125' Sector Gate Masonry Plan
S-026A	125' Sector Gate Masonry Plan
S-027	125' Sector Gate Masonry Sections
S-027A	125' Sector Gate Masonry Sections
S-029	125' Sector Gate Foundation Plan
S-029A	125' Sector Gate Foundation Plan
S-030	125' Sector Gate Upper Frame
S-030A	125' Sector Gate Upper Frame
S-031	125' Sector Gate Middle Frame
S-031A	125' Sector Gate Upper Middle Frame
S-032	125' Sector Gate Lower Frame
S-032A	125' Sector Gate Lower Middle Frame
S-033	125' Sector Gate Lower Frame
S-034	125' Sector Gate Channel Truss
S-034A	125' Sector Gate Channel Truss
S-035	125' Sector Gate Middle Channel Truss
S-035A	125' Sector Gate Middle Channel Truss
S-036	125' Sector Gate Middle Recess Truss
S-036A	125' Sector Gate Middle Recess Truss
S-037	125' Sector Gate Recess Truss
S-037A	125' Sector Gate Recess Truss
S-038	125' Sector Gate Fender
S-038A	125' Sector Gate Fender
S-040	125' Sector Gate Hinge Assembly and Details
S-040A	125' Sector Gate Hinge Assembly and Details
S-041	125' Sector Gate Pintle Assembly and Details
S-041A	125' Sector Gate Pintle Assembly and Details
S-043	125' Needle Girder and Support Towers
S-044	125' Sector Gate Dewatering Needles
S-045	125' Sector Gate Needle Girder Storage Platform
S-046	125' Sector Gate Needle Girder Storage Platform Sections
S-047	125' Sector Gate Guidewall Plan
S-048	125' Sector Gate Guidewall Section
S-049	125' Sector Gate End Cell Dolphin
S-050	125' Sector Gate Cofferdam
S-054	Sluice Gate Masonry Plan
S-054A	Sluice Gate Masonry Plan
S-055	Sluice Gate Front Elevation
S-055A	Sluice Gate Front Elevation
S-056	Sluice Gate Back Elevation
S-056A	Sluice Gate Back Elevation
S-057	Sluice Gate Section

S-057A	Sluice Gate Section
S-058	Sluice Gate Foundation Plan
S-058A	Sluice Gate Foundation Plan
S-059	Sluice Gate Dewatering Bulkhead
S-060	20' Stop Log Isometric
S-061	20' Stop Log Masonry Plan
S-061A	20' Stop Log Masonry Plan
S-062	20' Stop Log Masonry Elevation
S-062A	20' Stop Log Masonry Elevation
S-063	20' Stop Log Foundation Plan
S-063A	20' Stop Log Foundation Plan
S-064	20' Stop Log Needle Girder
S-064A	20' Stop Log Gate and Needle Girder
S-065	20' Stop Log Gate Bulkhead and Storage Platform
S-065A	20' Stop Log Gate Bulkhead and Storage Platform
S-066	20' Stop Log Gate Crane Platform
S-066A	20' Stop Log Gate Crane Platform
S-067	30' Stop Log Gate Isometric
S-068	30' Stop Log Masonry Plan
S-068A	30' Stop Log Masonry Plan
S-069	30' Stop Log Masonry Elevation
S-069A	30' Stop Log Gate Masonry Elevation
S-070	30' Stop Log Gate Foundation Plan
S-070A	30' Stop Log Gate Foundation Plan
S-071	30' Stop Log Gate and Needle Girder
S-071A	30' Stop Log Gate and Needle Girder
S-072	30' Stop Log Gate Bulkhead and Storage Platform
S-072A	30' Stop Log Gate Bulkhead and Storage Platform
S-073	30' Stop Log Gate Crane Platform
S-073A	30' Stop Log Gate Crane Platform
S-075	Highway Gate Masonry Plan & Section
S-075A	Highway Gate Masonry Plan & Section
S-076	Highway Gate Steel Plan & Section
S-076A	Highway Gate Steel Plan & Section
S-077	Highway Guardrail
S-078	Highway Excavation Plan & Section
S-081	Environmental Control Structure Masonry Plan
S-081A	Environmental Control Structure Masonry Plan
S-082	Environmental Control Structure Masonry Elevation Flood Side
S-082A	Environmental Control Structure Masonry Elevation Flood Side
S-083	Environmental Control Structure Masonry Elevation Protected Side
S-083A	Environmental Control Structure Masonry Elevation Protected Side
S-084	Environmental Control Structure Section

S-084A   Environmental Control Structure Section
S-085   Environmental Control Structure Foundation Plan
S-085A   Environmental Control Structure Foundation Plan
S-086   Environmental Control Structure Dewatering Bulkhead
S-086A   Environmental Control Structure Dewatering Bulkhead
S-087   Environmental Control Structure Wingwall Monolith Pile Layout and
                Section
S-087A   Environmental Control Structure Wingwall Monolith Pile Layout and
                Section
S-088   Environmental Control Structure Trash Rack
S-088A   Environmental Control Structure Trash Rack
S-089   Environmental Control Structure Schedule of Variables
S-089A   Environmental Control Structure Schedule of Variables
S-092   T-Wall Type 1A-1B
S-092A   T-Wall Type 1A-1B
S-093   T-Wall Type 1C-1D
S-093A   T-Wall Type 1C-1E
S-094   T-Wall Type 2A-2B
S-094A   T-Wall Type 2A-2B
S-095   T-Wall Type 2C-2D
S-095A   T-Wall Type 2C-2E
S-096   T-Wall Type 3A-3B
S-096A   T-Wall Type 3A-3B
S-097   T-Wall Type 3C-3D
S-097A   T-Wall Type 3C-3E
S-098   T-Wall Type 4A-4B
S-098A   T-Wall Type 4A-4B
S-099   T-Wall Type 4C-4D
S-099A   T-Wall Type 4C-4E
S-100   Phase 2 Cofferdam Plan
S-101   Stop Log Gates Cofferdam

Electrical Plates

E-001   56' Sector Gate Electrical
E-002   56' Sector Gate Electrical
E-003   56' Sector Gate Electrical
E-004   56' Sector Gate Electrical
E-005   175' Sector Gate Electrical
E-006   175' Sector Gate Electrical
E-007   175' Sector Gate Electrical
E-008   175' Sector Gate Electrical
E-009   175' Sector Gate Electrical

**MORGANZA TO THE GULF OF MEXICO, LOUISIANA DRAFT PAC
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E-010   175' Sector Gate Electrical

Mechanical Plates
M-001   56' Sector Gate Operating Machinery
M-002   175' Sector Gate Operating Machinery
M012   Fronting Protection Butterfly Valves

REACH A - NORTH OF G.I.W.W.
100 YEAR

END BARRIER ALIGNMENT
BEGIN REACH A STA. 1828+22.13

MATCH LINE STA. 1907+62

MATCH STA. 1828+22.13

RCH 1828+22

MINORS CANAL

BORROW PIT

PROP RW

PROTECTED SIDE

FLOOD SIDE

T17S R16E

PI 1897+54
PI 1884+69
PI 1887+27
1857+62

1900+00
1890+00
1880+00
1870+00
1840+00

S 40° 58' 11" E
S 20° 03' 04" W
S 68° 76' 75" E
S 1° 54' 46" E

N

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK DESCRIPTION DATE APPR.

SHEET
IDENTIFICATION
SHEET 2 OF 5

REACH A - 100 YEAR

N

LAKE HATCH

LAKE HATCH

MATCH LINE STA. 2032+60

MATCH LINE STA. 1907+62

BORROW PIT

PROP R/W
PROP R/W
PROP R/W
PROP R/W
PROP R/W
PROP R/W

PI 1960+92
N 89° 20' 08" E
1960+00
1960+00

PI 1941+40
N 02° 57' 58" E
PI 1953+36
S 12° 45' 48" E

PI 1912+60
MINORS CANAL

1920+00

PROTECTED SIDE
FLOOD SIDE

GULF INTRACOASTAL WATERWAY

T18S R16E

1980+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
BERM DEFLECTION POINT
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

12

11

1

2

65
64
63
62
61
60

5   4   3   2   1

D   C   B   A

MATCH LINE STA. 2073+26

MATCH LINE STA. 2032+60

REACH A - SOUTH OF G.I.W.W.
100 YEAR

N

31

32

34

35

16

12

T18S R17E

T18S R16E

PROTECTED SIDE

LAKE HATCH FLOOD SIDE

PROP R/W

PROP R/W

BORROW PIT

GULF INTRACOASTAL WATER WAY

GULF INTRACOASTAL WATERWAY

PI 2046+29

PI 1996+59

S 25° 47' 03" E

S 40° 47' 03" E

S 40° 47' 03" E

N 89° 02' 08" E

2066+00

2040+00

2020+00

2000+00

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION

US Army Corps
of Engineers®
New Orleans District

DATE: FEB. 2012
DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
CONTRACT NO:
SOLICITATION NO:
FILE NUMBER: FEB. 2012
PLOT SCALE:
FILE NAME:
SIZE: ANSI D

SHEET
IDENTIFICATION
3 of 5

MARK   DATE   DESCRIPTION   APPR.
MARK   DATE   DESCRIPTION   APPR.

MARK	APPR.	DATE	DESCRIPTION	MARK	APPR	DATE	DESCRIPTION	MARK

FILE NAME: M09100_A_4.dgn
ANSI D
SIZE:

REACH A PLAN VIEW
STATION 2073+25 TO STATION 2183+22

FILE NUMBER: HH-47041
FEB. 2012
PLOT SCALE: PLOT DATE:
JM QUATREAUX
CONTRACT NO:
SUBMITTED BY:

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT
Terrebonne Parish, LA

SOLICITATION NO:
CKD BY: T.D.T.
DWN BY: T.D.T.
D.XHETE
DATE: DESIGNED BY:
FEB. 2012 T.D.T./T.D.T.

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
STATION 2073+25 TO STATION 2183+22

SHEET IDENTIFICATION

SHEET 4 OF 5

MATCH LINE STA. 2183+22

2180+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

N

T18S R17E

18

31

PROTECTED SIDE

BAYOU DULARGE

REACH A - SOUTH OF G.I.W.W.
100 YEAR

2160+00

S. 19° 21' 05" E

FLOOD SIDE

2140+00

PROP R/W

BORROW PIT

PROP R/W

2120+00

36

35

PI 2115+62

S. 24° 18' 29" W

PI 2097+49

2100+00

S. 55° 41' 03" E

2080+00

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MATCH LINE STA. 2073+26

US Army Corps of Engineers
New Orleans District

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH A PLAN VIEW
STATION 2183+22 TO STATION 2259+26.41

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MARK	APPR	DATE	DESCRIPTION	MARK	APPR	DATE	DESCRIPTION	APPR.	DATE

SUBMITTED BY:
JIM GAUTREAUX
DWN BY: S. HUETE
CKD BY: S. HUETE
DESIGNED BY: T.D.T.

DATE: FEB. 2012
SOLICITATION NO:
CONTRACT NO:
FILE NUMBER: HH-47641
PLOT SCALE:
PLOT DATE: FEB. 2012
SIZE: ANSI D
FILE NAME: M09106_A-05.dgn

LEGEND

- BORROW PIT
- CENTERLINE OF LEVEE
- MITIGATION AREA
- RIGHT-OF-WAY
- TOE OF LEVEE SLOPE
- TOP OF LEVEE SLOPE
- EXISTING ROAD
- EXISTING WATERWAY
- PUMP STATION
- SLUICE GATE AND BOX CULVERTS

REACH A - SOUTH OF G.I.W.W.
100 YEAR

N

MARMANDE CANAL

MATCH STA. 2259+26.11
END REACH A - BEGIN REACH B

MATCH LINE STA. 2183+22

PROTECTED SIDE

FLOOD SIDE

BORROW PIT

PROP R/W

PI 2247+37

PI 2191+14

S 37° 38' 52" W

T18S R17E

T18S R16E

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

2240+00
2220+00
2200+00

23 24 25 26 27 28 29 30 31 36 37 38 25

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

REACH B 100 YEAR

MATCH LINE STA. 2364+25

S 24° 31' 24" W
S 11° 20' 37" E
S 54° 54' 44" E

PI 2335+61
PI 2300+77
PI 2289+36

PIPELINE CANAL
MARMANDE CANAL

PROTECTED SIDE
FLOOD SIDE
MITIGATION AREA
BORROW PIT
PROP. R/W

CENTERLINE REACH B
CENTERLINE REACH A

END REACH A
BEGIN REACH B STA. 2259+26.41

2360+00
2340+00
2320+00
2300+00
2280+00

T19S R17E   T19S R16E
T18S R17E   T18S R16E

59   60   61   62   63   64   24
25   26   27   28   29   36   38   1

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH B PLAN VIEW
STATION 2364+25 TO STATION 2469+26

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

US Army Corps
of Engineers
New Orleans District

MATCH LINE STA. 2469+26

MATCH LINE STA. 2364+25

REACH B - 100 YEAR

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

T19S R16E
T19S R17E

PROP R/W
BORROW PIT
MITIGATION AREA
PROTECTED SIDE

BAYOU DULARGE

S 39° 47' 19" W
S 24° 37' 24" W

PI 2407+29

2480+00
2460+00
2440+00
2420+00
2400+00
2380+00

US Army Corps
of Engineers®
New Orleans District

SHEET
IDENTIFICATION
3 3
SHEET 3 OF 3

REACH B PLAN VIEW
STATION 2469+26 TO STATION 2526+35.62

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE CROWN
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

REACH B   100 YEAR

END REACH B STA. 2526+35.62
BEGIN REACH E-2

MATCH LINE STA. 2526+35.62

MATCH LINE STA. 2469+26

BAYOU DULARGE

FALGOUT CANAL

FALGOUT CANAL

FLOOD SIDE

FLOOD SIDE

PROTECTED SIDE

PROP R/W

BORROW PIT

PROP R/W

MITIGATION
AREA

T19S R17E
T19S R16E

25
26
23
24
13
14
12
71
52
65

N

US Army Corps
of Engineers®
New Orleans District

MARK	DATE	APPR.	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	APPR.	DATE

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

DESIGNED BY:
DRAWN BY:
CKD BY:
SUBMITTED BY:
SOLICITATION NO.:
CONTRACT NO.:
PLOT SCALE:
PLOT DATE: FEB. 2012
FILE NUMBER: HH-47041
FILE NAME: M20100_Barrier_L.dgn
SIZE: ANSI D
DATE: FEB. 2012

SCALE: 1" = 400'

SHEET IDENTIFICATION
1
SHEET 1 OF 8

N

MATCH LINE STA. 1075+00

BEGIN BARRIER ALIGNMENT
STA. 1000+00.00

BARRIER ALIGNMENT

PROTECTED SIDE
FLOOD SIDE

BAYOU BLACK

BORROW PIT

PROP. R/W

PI 1041+17
PI 1000+60

S 66° 16' 13" E
S 58° 47' 13" E

1000+00
1020+00
1040+00
1060+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

400' 0 400' 800' 1200' 1600'

R14E
T16S

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

ANSI D
SIZE: D
FILE NAME:
M09100_Border_2.dgn

PLOT SCALE:
FILE NUMBER: HH-47341
PLOT DATE: FEB. 2012

CONTRACT NO.:
SUBMITTED BY: JIM GAITREAUX
AS QUANTIFIED BY:

SOLICITATION NO.:
DWN. BY:
CKD. BY:
DATE: FEB. 2012

DESIGNED BY:
T.O.L./T.C.K.
S. HVEEE
T.O.L./T.C.K.

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
BARRIER ALIGNMENT PLAN VIEW
STATION 1075+00 TO STATION 1190+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

N

MATCH LINE STA. 1190+00

MATCH LINE STA. 1075+00

BARRIER ALIGNMENT 100 YEAR

BAYOU BLACK

PROTECTED SIDE

FLOOD SIDE

PROP R/W
PROP R/W

BORROW PIT

T17S R15E
T16S R14E

S 66° 37′ 36″ E
S 27° 13′ 02″ E
S 68° 16′ 27″ E
S 66° 37′ 13″ E

PI 1149+41
PI 1121+11
PI 1109+90

SCALE: 1″ = 400′

400′ 0 400′ 800′ 1200′ 1600′

58
59
61
71
73
74
75
76
80
81
98
99
100
101
102
103
104
105

1080+00
1100+00
1120+00
1140+00
1160+00
1180+00

MARK	DESCRIPTION	DATE	APPR	MARK	DESCRIPTION	DATE	APPR

ANSID
SIZE:
FILE NAME:
MORGC_Barrier_3.dgn

SIZE: 40x61
PLOT SCALE: FILE NUMBER: H-H-47041
PLOT DATE: FEB. 2012
SUBMITTED BY:
QM GATEWAY/AX
CONTRACT NO:
CKD BY: T.D.T.
DWN BY: G. HUETE
SOLICITATION NO:
DESIGNED BY: T.D.T.
DATE: FEB. 2012

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

Terrebonne Parish, LA
HURRICANE PROTECTION PROJECT
MORGANZA, LOUISIANA TO THE GULF OF MEXICO

BARRIER ALIGNMENT PLAN VIEW
STATION 1190+00 TO STATION 1305+00

SHEET IDENTIFICATION
SHEET 3 OF 8

MATCH LINE STA. 1305+00

S 72° 30' 37" E

SHELL CANAL

PI 1286

1290+00

30

31

32

BARRIER ALIGNMENT 100 YEAR

88

89

90

91

92

93

94

95

96

97

T17S R15E

BAYOU BLACK

BAYOU BLACK MARINA

81

80

79

78

77

76

PROTECTED SIDE

FLOOD SIDE

PROP R/W

PROP R/W

BORROW PIT

S 57° 55' 30" E

PI 1281+32

1280+00

S 75° 17' 28" E

MATCH LINE STA. 1190+00

N

LEGEND

BORROW-PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MARK	APPR.	DATE	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	MARK	APPR.	DATE

SHEET
IDENTIFICATION
SHEET 4 OF 8

BARRIER ALIGNMENT
100 YEAR

BAYOU BLACK

MATCH LINE STA. 1415+17

MATCH LINE STA. 1305+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

PROTECTED SIDE

FLOOD SIDE

T17S R15E

PROP R/W
PROP R/W

S 57° 07'17" E
S 53° 35'09" E
S 66° 04'01" E
S 68° 34'04" E
S 72° 50'57" E

PI 1381+04
PI 1372+82
PI 1369+26
PI 1364+66
PI 1342+77
PI 1323+39

ENTERPRISE PRODUCTS
SHELL PIPELINE
SHELL CANAL

1400+00
1390+00
1380+00
1360+00
1340+00
1330+00
1320+00

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

N

12
13
14
15
16
17
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19
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21
22
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36
81
83
84
85
86
88

BARRIER ALIGNMENT
100 YEAR

MATCH LINE STA. 1530+17

MATCH LINE STA. 1415+17

PI 1524+43

S 73° 55' 11" E

PI 1512+45

BORROW PIT

PROP R/W

PROP R/W

S 53° 54' 08" E

FLOOD SIDE

PROTECTED SIDE

BAYOU BLACK

T17S R15E

PI 1442+11

S 57° 07' 17" E

PI 1421+84

1600+00

1500+00

1484+00

1460+00

1460+00

1420+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

US Army Corps
of Engineers®
New Orleans District

MARK   DATE   APPR.   DESCRIPTION
MARK   DATE   APPR.   DESCRIPTION

US Army Corps of Engineers
New Orleans District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

FILE NAME: M0910C Border-2.dgn
ANSI D
SIZE:
FILE NAME:
FILE NUMBER: H-H-47541
PLOT SCALE: PLOT DATE: FEB. 2012
JM QUATTLEBAUX
SUBMITTED BY:
CONTRACT NO.:
OWN BY: CKD BY:
SOLICITATION NO.: S. HUETE T.J.T.
DATE: T.D.J./T.D.K.
DESIGNED BY: FEB. 2012

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT
Terrebonne Parish, LA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
BARRIER ALIGNMENT PLAN VIEW
STATION 1630+17 TO STATION 1745+17

BARRIER ALIGNMENT
100 YEAR

MATCH LINE STA. 1745+17

MATCH LINE STA. 1630+17

T17S R16E

BAYOU BLACK

ALLIGATOR FARM

PROTECTED SIDE

FLOOD SIDE

BORROW PIT

HANSON CANAL

PROP R/W

PI 1640+62
PI 1656+79

S 35° 50' 13" E
S 45° 26' 35" E
S 56° 26' 39" E
N 88° 20'

1740+00
1720+00
1700+00
1680+00
1660+00
1640+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

US Army Corps
of Engineers®
New Orleans District

MARK	DATE	APPR.	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	APPR.	DATE

FILE NAME:
M30100_Barrier_8.dgn
SIZE: ANSI D
PLOT SCALE: PLOT DATE: FEB. 2012
FILE NUMBER: HH-47041
QUANTITIES BY: JM
CONTRACT NO: SUBMITTED BY:
CKD BY: D. XHUETE
SOLICITATION NO: DWN BY: T.O.T.
DATE: SURVEYS BY: T.O.T. / D.X.
FEB. 2012
DESIGNED BY:

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT
Terrebonne Parish, LA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT

BARRIER ALIGNMENT PLAN VIEW
STATION 1745+17 TO STATION 1828+22.13

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

MATCH LINE STA. 1828+22.13

PI 1826+26

1820+00

BARRIER ALIGNMENT
100 YEAR

END BARRIER ALIGNMENT STA. 1828+22.13
BEGIN REACH A

S 16° 41' 44" E

1810+00

PI 1793+04

BORROW PIT

PROP R/W

S 42° 04' 59" E

PROP R/W

1790+00

PROTECTED SIDE

PI 1764+47

PROP R/W

1780+00

S 50° 50' 13" E

T17S R16E

BAYOU BLACK

MATCH LINE STA. 1745+17

HANSON CANAL

FLOOD SIDE

HANSON CANAL

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

MARK	DESCRIPTION	APPR.	DATE	MARK	DESCRIPTION	APPR.	DATE
1	ATR REVIEW COMMENT	T.D.T.	10 APR 2012				

ANSI D

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT

Terrebonne Parish, LA

REACH E2 PLAN VIEW
STATION 2526+35.62 TO STATION 2571+43

SHEET
IDENTIFICATION 1
SHEET 1 OF 4

REACH E2
100 YEAR

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

T19S R17E
T19S R16E

MATCH STA. 2571+43

MITIGATION AREA

BORROW PIT

2560+00

PI STA. 2552+61

HIGHWAY 315
DU LARGE ROAD GATE

2540+00

N 85° 58' 07" E

S 62° 11' 28" E

PI STA. 2532+54

2531+34

S 18° 32' 30" E

PI STA. 2519+73

BEGIN REACH E-2
END REACH B STA. 2526+35.62

MITIGATION AREA

FLOOD SIDE

BAYOU DULARGE

FLOOD SIDE

PROTECTED SIDE

BAYOU DULARGE

FALGOUT CANAL

FALGOUT CANAL

26
25
25
24
26
14
13
15
51
52
71
72

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH E-2 PLAN VIEW
STATION 2571+43 TO STATION 2621+34

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

US Army Corps
of Engineers®
New Orleans District

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

REACH E-2
100 YEAR

MATCH LINE STA. 2621+34

MATCH STA. 2571+43

PROTECTED SIDE

FLOOD SIDE

FALGOUT CANAL

BAYOU DULARGE

BORROW PIT

MITIGATION AREA

PROP R/W

T19S R17E
T19S R16E

S 69° 03' 44" E

SCALE: 1"= 400'

0   400'   800'   1200'   1800'

MARK	DESCRIPTION	APPR.	DATE	MARK	DESCRIPTION	APPR.	DATE

LEGEND

- BORROW PIT
- CENTERLINE OF LEVEE
- MITIGATION AREA
- RIGHT-OF-WAY
- TOE OF LEVEE SLOPE
- TOP OF LEVEE SLOPE
- EXISTING ROAD
- EXISTING WATERWAY
- PUMP STATION
- SLUICE GATE AND BOX CULVERTS

REACH E-2 & E-1
100 YEAR

N

T19S R17E

PROTECTED SIDE

END REACH E-2 STA. 2646+34
BEGIN REACH E-1

MATCH LINE STA. 2621+34

MATCH LINE STA. 2726+33

BORROW PIT

MITIGATION AREA

PROP. R/W

PROP. R/W

FLOOD SIDE

S 66°14'52" E

S 69°03'44" E

PI 2632+20

RCH 2646+34

2640+00

2660+00

2680+00

2700+00

2720+00

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

11
72
13
69
29
30
28
74
27
26
25
24
83
75
18
76

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

REACH E-1 & F-2
100 YEAR

N

5
4
3
2
1

T19S R17E

CENTERLINE REACH E-1

END REACH E-1 STA. 2758+84
BEGIN REACH F-2

MATCH LINE STA. 2726+33

PROTECTED SIDE

PROP. R/W

BORROW PIT

PROP. R/W

MITIGATION AREA

FLOOD SIDE

STA. 2788+92.72

S 7° 52' 30" W

S 6° 13' 28" E

S 5° 45' 36" W

2780+00

2760+00

86
87
88
89
28
27
26
25
24
23
29
74
78
75
73
69

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

US Army Corps
of Engineers®
New Orleans District

SHEET
IDENTIFICATION 1 of 2

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO

REACH F-2 & F-1 PLAN VIEW
STATION 2788+92.72 TO STATION 2863+83

STATION 2788+92.72 TO STATION 2863+83

MARK | DATE | APPR. | DESCRIPTION | MARK | DATE | APPR. | DESCRIPTION

FILE NAME:
M0910-F2.dgn
ANSI D
SIZE:
402:1
FILE NUMBER:
HH-47841
PLOT DATE:
FEB. 2012
PLOT SCALE:
1"=400'
FILE NAME:
JIM GAUTREAUX
SUBMITTED BY:
CONTRACT NO.:
DWN BY: CKD BY:
D. XAVIER T.O.T.
SOLICITATION NO.:
DATE:
FEB. 2012
DESIGNED BY:
T.D.T./T.D.T.

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'
400'   0   400'   800'   1200'   1600'

REACH F-2 & F-1
100 YEAR

END REACH F-2 STA. 2860+35
BEGIN REACH F-1

MATCH LINE STA. 2863+83

RCH 2860+35
PI 2860+35

PROP. R/W
PROP. R/W
BORROW PIT

FLOOD SIDE

STA. 2788+92.72

T19S R17E
T20S R17E

PROTECTED SIDE

HOUMA NAVIGATIONAL CANAL

US Army Corps of Engineers
New Orleans District

MARK	DESCRIPTION	DATE	APPR.		MARK	DESCRIPTION	DATE	APPR.

ANSI D
SIZE
PLOT DATE: FEB. 2012
PLOT SCALE: 1=1.dgn
FILE NAME: M2910G-1-F1.dgn
FILE NUMBER: HH-47041
DESIGNED BY:
DRAWN BY: D. HUETE
CKD BY: T.D.T.
SUBMITTED BY: JIM GAUTREAUX
DATE: FEB. 2012
CONTRACT NO.:
SOLICITATION NO.:

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH F-1 AND G-1 PLAN VIEW
STATION 2863+83 TO STATION 3007+34.70

SHEET IDENTIFICATION 2 OF 2

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

N

REACH F-1
100 YEAR

T20S R17E

PROTECTED SIDE

MATCH LINE STA. 3007+34.70

PROPOSED STONE CLOSURE DAM

PROP. R/W

END REACH F-1
BEGIN REACH G-1 STA. 2987+35

TEMP CONST EASEMENT

HOUMA NAVIGATIONAL CANAL LOCK
CONTRUCTION OF THE LOCK TO BE DONE BY OTHERS.

N 64° 16' 17" E

TEMP CONST EASEMENT

S 25° 16' 53" E

BORROW PIT

PI 2943+91

S 6° 33' 08" W

PI 2924+55

BAYOU GRAND CAILLOU ACESSS CHANNEL

FLOOD SIDE

APPROACH CHANNEL

EXISTING GATE

PI 2906+89

S 1° 26' 33" W

PI 2894+98

S 5° 34' 09" W

BORROW PIT

PROP. R/W

MATCH LINE STA. 2863+83

2980+00
3000+00
2960+00
2940+00
2920+00
2900+00
2880+00

REACH G-1'
100 YEAR

T20S R17E

PROTECTED SIDE

15

BORROW PIT

PROP R/W

PROP R/W

MATCH LINE STA. 3087+35

S 16° 10' 03" E

PI 3

S 26° 41' 21" E

PI 3066+19

S 25° 13' 02" E

3060+00

22

PI 3043+41

3040+00

3020+00

PI 3024+85

N 65° 59' 26" E

MATCH STA. 3007+34.70

16

27

HOUMA NAVIGATIONAL CANAL

HOUMA NAVIGATIONAL CANAL LOCK
CONTRUCTION OF THE LOCK TO BE
DONE BY OTHERS.

FLOOD SIDE

21

20

N

SCALE: 1" = 400'

400' 0' 400' 800' 1200' 1600'

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

MARK	DATE	APPR.	DESCRIPTION	MARK	DATE	APPR.	DESCRIPTION

MARK	DESCRIPTION	APPR.	DATE
1	ATR REVIEW COMMENT	T.D.T.	10 APR 2012

FILE NAME: M29100_02_.dgn
ANSI D

PLOT SCALE: FEB. 2012
FILE NUMBER: HH-47041
SIZE: 4021

PLOT DATE:
SUBMITTED BY: JW GAUTREAUX

CONTRACT NO:
T.D.T.

SOLICITATION NO:
DWN BY: S.HUFFER
CKD BY: T.D.T.

DATE: FEB. 2012
DESIGNED BY: T.D.T.

REACH G-1, G-2,
100 YEAR

T20S R18E
T20S R17E

N

112
113
13
14
24
25
23
26
15
22
27

PROTECTED SIDE
FLOOD SIDE

END REACH G-1 STA. 3098+96
BEGIN REACH G-2

MATCH LINE STA. 3162+35
MATCH STA. 3087+35

PROP R/W
PROP R/W
PROP R/W
PROP R/W

BORROW PIT

PI 3127+46
PI 3105+67
PI 3092+11

FOUR POINT ROAD
ROAD GATE

RCH 3098+96

S 79° 29' 15" E
S 69° 37' 39" E
S 76° 47' 40" E

3100+00
3120+00
3140+00
3160+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH G-1 & G-2 PLAN VIEW
STATION 3087+35 TO STATION 3162+35
STATION 3087+35 STATION 3162+35

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

US Army Corps
of Engineers
New Orleans District

DESIGNED BY:
DRAWN BY: CKD BY:
DATE: FEB. 2012
T.D. / T.C.C.
S. HUETE   T.A.T.
SOLICITATION NO:
CONTRACT NO:
SUBMITTED BY:
JW GATREAUX
PLOT SCALE: PLOT DATE: FEB. 2012
FILE NUMBER: HH-47541
FILE NAME:
SIZE: ANSI D
M35100_G3.dgn

MARK | DESCRIPTION | DATE | APPR
MARK | DESCRIPTION | DATE | APPR

REACH G-3
100 YEAR

END REACH G-3 STA. 3224+12.12
BEGIN REACH H-1

MATCH STA. 3224+12.12

MATCH STA. 3162+35

T20S R18E
T20S R17E

BORROW AREA

PROP R/W
PROP R/W

S 31°02'34" E
S 33° 47' 40" E

RCH 3224+12
RCH 3180+00
RCH 3161+53
PI 3161+53
3220+00
3200+00

N

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

84   85   86   87   88
83   82   81   80   79
114   113   25   26
112   24   23

US Army Corps
of Engineers®
New Orleans District

SHEET
IDENTIFICATION
1 4
SHEET 1 OF 4

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

ANSI D
SIZE
FILE NAME: M3910C-H1.dgn

FILE NUMBER: HH-47041
PLOT SCALE: PLOT DATE: FEB. 2012
CONTRACT NO:
SOLICITATION NO:

40X1
JM QUATREAUX
SUBMITTED BY:
DWN BY: DKD
DESIGNED BY:
DATE: FEB. 2012
CKD BY: T.D.T./T.C.E.
S. HUETER T.A.T.

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

Terrebonne Parish, LA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT

REACH H-1 PLAN VIEW
STATION 3224+12.12 TO STATION 3313+12

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

MATCH LINE STA. 3313+12

PI 3309+97
S 64°02'11"E
3300+00
PI 3294+76
S 89°14'11"E
3290+00
PROP. R/W
PI 3265+
PI 3265+
BORROW PIT
PROP. R/W
MITIGATION AREA
N 71°
PI 3257+84
S 52°37'E
S 52°37'E
3240+00

MATCH STA. 3224+12.12

REACH H-1 STA. 3224+12

REACH H-1
100 YEAR

PROTECTED SIDE

FLOOD SIDE

N

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

T20S R18E
T20S R17E

T20S R18E

113
100
99
98
5
84
85
86
83
4
87
3
2
1
25
114
115
36
88
89
90
91
92

US Army Corps
of Engineers®
New Orleans District

SHEET
IDENTIFICATION 2 OF 4

SHEET 2 OF 4

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH H-1 & H-2 PLAN VIEW
STATION 3313+12 TO STATION 3416+12

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR	MARK	DESCRIPTION	DATE	APPR

DESIGNED BY:
T.D./T.C/C.
DRAWN BY:
T.A.T.
CKD BY:
S. HUETE
SUBMITTED BY:
JW QUATREVAUX
DATE:
FEB. 2012
PLOT SCALE:
FEB. 2012
PLOT DATE:
FEB. 2012
FILE NUMBER:
HH-42741
FILE NAME:
M09100_H2_dgn
SIZE:
ANSI D

SOLICITATION NO:
CONTRACT NO:

REACH H-1 & REACH H-2
100 YEAR

REACH H-1 & REACH H-2

MATCH LINE STA. 3416+12

77
78
79
80
81
82
83
84
85
86
87
88
89
90

T20S R18E

PROTECTED SIDE

RCH 3319+12

END REACH H-1 STA. 3319+12
BEGIN REACH H-2

MATCH LINE STA. 3313+12

HIGHWAY 56 ROAD GATE

PI 3325+53

3320+00

PROP. R/W

PROP. R/W

BORROW PIT

BORROW PIT

MITIGATION AREA

MITIGATION AREA

N 17° 33' 58" E

N 5° 49' 47" E

N 9° 02' 04" E

N 3° 53' 11" E

S 83° 53' 54" E

PI 3389+41

PI 3368+28

PI 3341+98

3400+00
3380+00
3360+00
3340+00

23
24
25
26
27
28
29
30
31

103
102
101
100
99
98
97

FLOOD SIDE

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

N

SCALE: 1" = 400'

400 0 400 800 1200 1600'

MATCH LINE STA. 3531+12

REACH H-2 &
100 YEAR   H-3

64
65
66
67
68
69
70
71
72
73
74
75
76
77

PROTECTED SIDE

END REACH H-2 STA. 3460+81
BEGIN REACH H-3

MATCH LINE STA. 3416+12

12
13
14
15
16
17
18
19
20
21
22
23
24

N28°12'19"E
PI 3521+81
N4°21'36"E
N6°32'40"E
N11°33'58"E

3520+00
3500+00
3480+00
3460+00
3440+00
3420+00
3400+00

PI 3460+81
PI 3442+32

BORROW PIT
PROP ROW
RCH
MITIGATION AREA

105
106
104
103

FLOOD SIDE

T20S, R18E

41
42
43
44
45
46

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE:1" = 400'

400' 0 400' 800' 1200' 1600'

US Army Corps
of Engineers®
New Orleans District

| MARK | DESCRIPTION | DATE | APPR. | MARK | DESCRIPTION | DATE | APPR. |

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH H-2 & H-3 PLAN VIEW
STATION 3416+12 TO STATION 3531+12

SHEET
IDENTIFICATION

SHEET 3 OF 4

US Army Corps
of Engineers®
New Orleans District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

FILE NAME:
M39100_H3_2.dgn
SIZE: ANSI D
FILE NUMBER: HH-47041
PLOT SCALE: PLOT DATE: FEB. 2012
CONTRACT NO.:
SUBMITTED BY: JM GAUTREAUX
SOLICITATION NO.:
OWN BY: CKD BY: DATE: FEB. 2012
DESIGNED BY:

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT
Terrebonne Parish, LA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
REACH H-3 PLAN VIEW
STATION 3531+12 TO STATION 3640+67

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

MATCH LINE STA. 3640+67

MATCH LINE STA. 3531+12

END REACH H-3 STA. 3640+67
BEGIN REACH I-1

REACH H-3
100 YEAR

T19S R18E
T20S R18E

PROTECTED SIDE

FLOOD SIDE

CENTERLINE REACH H-3

BAYOU PETIT CAILLOU

BAYOU TERREBONNE

PROP R/W
BORROW PIT
MITIGATION AREA

N33°01'02"E
N50°12'27"E
N72°12'19"E

PI 3633+75
PI 3614+52
PI 3575+66
PI 3537+50

3620+00
3600+00
3580+00
3560+00
3540+00

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

US Army Corps
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New Orleans District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

ANSI D
MG9190.1-2011.dgn
FILE NAME:
SIZE:

4021
HH-47041
FILE NUMBER:
FEB. 2012
REACH H-1 & I-2 PLAN VIEW
PLOT DATE:
PLOT SCALE:
STATION 3640+67 TO STATION 3750+65

JM QUATREAUX
CONTRACT NO:
SUBMITTED BY:

T.D.T.
DWN BY:
SOLICITATION NO:
S. HUETE
REVIEWED BY:

FEB. 2012
DATE:
T.D.T., T.D.T.
DESIGNED BY:
CKD BY:

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

HURRICANE PROTECTION PROJECT
MORGANZA, LOUISIANA TO THE GULF OF MEXICO
STATION 3640+67 TO STATION 3750+65
Terrebonne Parish, LA

SHEET
IDENTIFICATION
SHEET 1 OF 3

MATCH LINE STA. 3750+65

END REACH I-1 STA. 3733+30
BEGIN REACH I-2

END REACH H-3
BEGIN REACH I-1 STA. 3640+67

REACH I-1 & I-2
100 YEAR

PROTECTED SIDE

BAYOU TERREBONNE

BORROW PIT

MITIGATION AREA

PROP. R/W

PROP. R/W

S 64° 42' 25" E
N 41° 16' 37" E
N 34° 31' 32" E
N 42° 43' 55" E
N 34° 19' 14" E
N 28° 19' 22" E
N 19° 59' 14" E
N 19° 28' 29" E

PI 3742+23
PI 3726+55
PI 3721+46
PI 3720+13
PI 3717+08
PI 3713+74
PI 3711+40
PI 3702+47
PI 3693+11
PI 3688+66
PI 3677+47

3740+00
3720+00
3700+00
3680+00
3660+00

T19S R18E
T19S R19E
T20S R18E
T20S R19E

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

MATCH LINE STA. 3855+70

REACH I-2 & I-3
100 YEAR

N

PROTECTED SIDE

PROP. R/W

N 28° 51' 48" E

BORROW PIT

PROP. R/W 840+00

RCH 3832+26
PI 3832+26
PI 3829+89

N 19° 28' 27" E

PI 3822+55

END REACH I-2 STA. 3832+26
BEGIN REACH I-3

N 63° 72' 12" E

3820+00

PI 3803+25

3800+00

N 30° 51' 40" E

PI 3791+80

MITIGATION AREA

N 33° 47' 37" E

PI 3777+17

3780+00

N 25° 35' 12" E

BORROW PIT

PI 3763+54

3760+00

N 11° 14' 48" W

PI 3755+41

MATCH LINE STA. 3750+65

3733+30

FLOOD SIDE

T19S R18E
T19S R19E

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

REACH I-3
100 YEAR

MATCH LINE STA. 3941+75.55

HUMBLE CANAL

END REACH I-3 STA. 3941+75.55
BEGIN REACH J-2

T19S R19E

BAYOU TERREBONNE

BAYOU TERREBONNE

PROP. R/W

PROP. R/W

N 7° 58' 53" E

N 14° 19' 53" E

PI 3904+86

PI 3868+66

3940+00

3920+00

3900+00

3880+00

3860+00

BORROW PIT

MITIGATION AREA

PROTECTED SIDE

FLOOD SIDE

MATCH LINE STA. 3855+70

3

4

5

6

23

32

33

50

51

33

15

ND

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION
SLUICE GATE AND BOX CULVERTS

MARK	DATE	APPR.	DESCRIPTION	MARK	DATE	APPR.	DESCRIPTION	DATE	APPR.

US Army Corps
of Engineers ®
New Orleans District

MARK | DESCRIPTION | DATE | APPR | MARK | DESCRIPTION | DATE | APPR

FILE NAME: MORG10_J2_2.dgn
ANSI D
SIZE:
PLOT DATE: FEB. 2012
FILE NUMBER: H-8-47541
PLOT SCALE: 400:1
SUBMITTED BY:
JIM GAUTREAUX
DWN BY: CKD BY:
G. HUETE T.D.T.
SOLICITATION NO:
CONTRACT NO:
DESIGNED BY:
T.D.T./T.O.T.
DATE: FEB. 2012

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH J-2 PLAN VIEW
STATION 4048+75 TO STATION 4156+69

SHEET
IDENTIFICATION
SHEET 2 OF 5

MATCH LINE STA. 4156+69

ND

17

18

26

4140+00

T19S R19E

4120+00

PROTECTED SIDE

CENTERLINE REACH J-2

4100+00

ND

FLOOD SIDE

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

REACH J-2
100 YEAR

PROP. R/W

BORROW PIT

PROP. R/W

4080+00

MITIGATION AREA

PI 4062+45

S 49°55'08.5"

N 49°55'49" E

4060+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

MATCH LINE STA. 4048+75

N

US Army Corps
of Engineers®
New Orleans District

FILE NAME: M09100_J-1.dgn
ANSI D
SIZE:
FILE NUMBER: HH-47041
PLOT DATE: FEB. 2012
CONTRACT NO.:
SOLICITATION NO.:
SUBMITTED BY:
JIM GAUTREAUX
DATE: FEB. 2012
DESIGNED BY:
T.D.L./T.C.P.
DWN. BY: CKD BY:
S. HUETE T.A.T.

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH J-2 & J-1 PLAN VIEW
STATION 4156+69 TO STATION 4261+76

SHEET
IDENTIFICATION
SHEET 3 OF 5

ALIGNMENT 2

REACH J-2 & J-1
100 YEAR

T18S R20E

T19S R20E

ND

ND

ND

ND

ND

T18S R19E

T19S R19E

POINTE AU CHIEN BAYOU

LAFOURCHE PARISH

TERREBONNE PARISH

END REACH J-2 STA. 4202+33
BEGIN REACH J-1

MATCH LINE STA. 4261+76

PI 4256+11

4260+00

S 56°50'26" E

PI 4246+90

4240+00

PROP R/W

S 73°08'17" E

PROP R/W

BORROW PIT

PROP R/W
4220+00

PI 4202+33

MITIGATION AREA

FLOOD SIDE

4200+00

BORROW PIT

PROP R/W

PROP R/W

4180+00

MATCH LINE STA. 4156+69

PROTECTED SIDE

N

17
17
17
17

26
26
26

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK DESCRIPTION DATE APPR.
MARK DESCRIPTION DATE APPR.

US Army Corps of Engineers®
New Orleans District

| MARK | DATE | APPR. | DESCRIPTION | | MARK | DATE | APPR. | DESCRIPTION | | APPR. | DATE |

FILE NAME: M09100_J-1.dgn
ANSI D
SIZE:

FILE NAME: HH-47041
PLOT SCALE: FEB. 2012
FILE NUMBER: 4921
PLOT DATE:

SUBMITTED BY: JM QUITREAUX
CONTRACT NO.:

OWN. BY: CKD BY: S. HUETE T.D.T.
SOLICITATION NO.:

DESIGNED BY: T.D.T./D.E.
DATE: FEB. 2012

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH J-1 PLAN VIEW
STATION 4261+76 TO STATION 4367+74

REACH J-1
100 YEAR

N

MATCH LINE STA. 4367+74

MATCH LINE STA. 4261+76

ISLE DE JEAN CHARLES ROAD
ROAD GATE

RCH 4367+74

4360+00

4340+00

4320+00

4300+00

4285+80

4280+00

4278+03

BORROW PIT

PROP. R/W

PROP. R/W

MITIGATION AREA

S 18°-20'-09" E

S 42°-40'-45" E

S 08°-16'-46" E

POINTE AU CHIEN BAYOU

LAFOURCHE PARISH

TERREBONNE PARISH

PROTECTED SIDE

FLOOD SIDE

ND
T19S R19E

ND
T19S R20E

SCALE: 1" = 400'
400' 0 400' 800' 1200' 1600'

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION

SLUICE GATE AND BOX CULVERTS

US Army Corps of Engineers
New Orleans District

MARK | DATE | APPR. | DESCRIPTION
MARK | DATE | APPR. | DESCRIPTION

FIE NAME: M09100_J3.dgn
ANSI D
SIZE
FILE NUMBER: HH-47041
PLOT SCALE: PLOT DATE: FEB. 2012
DR. QUATREAUX
SUBMITTED BY:
CONTRACT NO:
T.A.T.
CKD BY:
OWN BY: D. HUETE
SOLICITATION NO:
T.O.J / T.O.K.
DESIGNED BY:
DATE: FEB. 2012

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH J-3 PLAN VIEW
STATION 4367+74 TO STATION 4438+85.25

REACH J-3
100 YEAR

END REACH J-3 STA. 4438+85.25
BEGIN REACH K

MATCH LINE STA: 4438+85.25

POINTE AU CHIEN BAYOU

HIGHWAY 665 ROAD GATE

FLOOD SIDE

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

PROTECTED SIDE

POINTE AU CHIEN BAYOU

LAFOURCHE PARISH
TERREBONNE PARISH

PROP. R/W

S 43° 50' 06" E

S 27° 04' 51" E

N 62° 46' 59" E

PI 4437+66
PI 4428+21

4420+00

PI 4367+41

4400+00

BORROW PIT

PROP. R/W 4390+50

MITIGATION AREA

MATCH LINE STA. 4367+74

END REACH J-1 STA. 4367+74
BEGIN REACH J-3

T19S R20E
ND

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

N

US Army Corps
of Engineers®
New Orleans District

MARK | DESCRIPTION | DATE | APPR. | MARK | DESCRIPTION | DATE | APPR.

FILE NAME:
M09100_K-1.dgn
SIZE: ANSI D
PLOT SCALE: 400:1
FILE NUMBER: HH-47341
PLOT DATE: FEB. 2012
DATE: FEB. 2012
AM QUATREVAUX
SUBMITTED BY:
CONTRACT NO.:
DWN BY: T.D.T.
CKD BY: T.D.T.
S. HUETE / J.M.
SOLICITATION NO.:
DESIGNED BY: T.D.T.

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA
REACH K PLAN VIEW
STATION 4438+85.25 TO STATION 4543+85

SHEET
IDENTIFICATION
1 3
SHEET 1 OF

REACH K
100 YEAR

N

MATCH LINE STA. 4543+85

PI 4537+78

4540+00

N 32° 26' 49" W

4520+00

PI 4515+00

GRAND BAYOU CANAL

PROP. R/W

PROP. R/W

BORROW PIT

N 11° 21' 06" E

PROTECTED SIDE

TERREBONNE PARISH
LAFOURCHE PARISH

T19S R20E
NA

4500+00

4480+00

4480+00

MITIGATION AREA

FLOOD SIDE

4440+00

POINTE AU CHIEN BAYOU

CENTERLINE REACH J-3

END REACH J-3
BEGIN REACH K STA. 4438+85.25

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

REACH K
100 YEAR

N

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MATCH LINE STA. 4658+86

PI 4652+26

N 37° 29' 36" E

PI 4636+76

4640+00

N 0° 58' 05" E

4620+00 GRAND BAYOU CANAL

PI 4618+77

PROP. R/W

BORROW PIT

N 19° 28' 13" E

PROP. R/W

MITIGATION AREA

4600+00

FLOOD SIDE

PI 4589+38

PI 4581+42

N 4° 37' 34" W

N 16° 12' 24" E

4580+00

PI 4565+94

GRAND BAYOU CANAL

4560+00

N 31° 27' 00" E

PI 4548+63

MATCH LINE STA. 4543+85

PROTECTED SIDE

T19S R20E   T18S R20E
NA   NA

T19S R20E
NA

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

REACH K
100 YEAR

T18S R20E

NA

PROTECTED SIDE

END REACH K STA. 4706+98.84
BEGIN REACH L

GRAND BAYOU CANAL

N 25° 33' 04" E

PROP. R/W

BORROW PIT

MITIGATION AREA

FLOOD SIDE

MATCH LINE STA. 4658+86

4700+00
4680+00
4660+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY
PUMP STATION
SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

SHEET
IDENTIFICATION
1   3
SHEET 1 OF

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MATCH LINE STA. 4812+00

PI 4810+94

N 21° 59' 43" E

REACH L
100 YEAR

GRAND BAYOU CANAL

PI 4799+77

N 21° 31' 44" E

FLOOD SIDE

PI 4782+00

N 22° 37' 10" E

T18S R20E

NA

PI 4765+80

PROTECTED SIDE

BORROW PIT

N 20° 25' 32" E

PI 4747+18

N 24° 49' 07" E

PROP. R/W

4770+00

PROP. R/W

END REACH K
BEGIN REACH L 4706+98.84

MATCH 4706+99

MATCH LINE STA. 4706+98.84

N

REACH L
100 YEAR

MATCH LINE STA. 4942+00

PI 4935+53

N

S 79° 24' 31" E

APACHE LOUISIANA MINERALS

4920+00

FLOOD SIDE

PI 4910+95

4900+00

4880+00

PROTECTED SIDE

T18S R20E
NA

PROP. R/W

S 87° 12' 24" E

PROP. R/W

4860+00

GRAND BAYOU CANAL

BAYOU BLUE

PI 4845+18

PI 4836+31

4820+00

4840+00

MATCH LINE STA. 4812+00

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
TOP OF LEVEE SLOPE
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MARK	DESCRIPTION	APPR.	MARK	DATE	DESCRIPTION	APPR.	DATE
1	ATR REVIEW COMMENT			10 APR 2012		T.Z.T.	

ANSI D
MXG109-L-249m

SIZE:
B021

FILE NAME:

PLOT SCALE: PLOT DATE:
FEB. 2012

FILE NUMBER:
HH-47041

CONTRACT NO.:

SUBMITTED BY:
JM GAUTREAUX

SOLICITATION NO.:

DWN BY:
T.R.T.

CKD BY:
C. HUETE

DATE:
FEB. 2012

DESIGNED BY:
T.G.T./T.Z.T.

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

REACH L PLAN VIEW
STATION 4812+00 TO STATION 4942+00

SHEET
IDENTIFICATION   SHEET 2 OF 3

2
3

US Army Corps
of Engineers ®
New Orleans District

SHEET
IDENTIFICATION 3 OF 3

SHEET 3 OF 3

REACH L PLAN VIEW
STATION 4942+00 TO STATION 5021+78

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
Terrebonne Parish, LA

U.S. ARMY CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
NEW ORLEANS DISTRICT

MARK	APPR	DATE	DESCRIPTION	MARK	APPR	DATE	DESCRIPTION	APPR	DATE

FILE NAME:
M20100.L-3.dgn

SIZE:
ANSI D

PLOT SCALE:
FILE NUMBER:
HH-47041

PLOT DATE:
FEB. 2012

CONTRACT NO:
SUBMITTED BY:
JIM GAUTREAUX

DWN BY:
S. HWETE

T.A.T.

DESIGNED BY:
T.O./T.D.C.

SOLICITATION NO:
CKD BY:
S. HWETE

DATE:
FEB. 2012

REACH L
100 YEAR

END REACH L STA. 5021+78

PROP. R/W

PROP. R/W

BORROW PIT

APACHE LOUISIANA MINERALS

PROTECTED SIDE

FLOOD SIDE

N 80° 07' 32" E

S 71° 36' 15" E

S 80° 21' 51" E

PI 4976+63

PI 4957+91

5020+00

5000+00

4980+00

4960+00

T18S R21E

T18S R20E

ND

MATCH LINE STA. 4942+00

N

LEGEND

BORROW PIT
CENTERLINE OF LEVEE
MITIGATION AREA
RIGHT-OF-WAY
TOE OF LEVEE SLOPE
EDGE OF LEVEE CROWN
EXISTING ROAD
EXISTING WATERWAY

PUMP STATION

SLUICE GATE AND BOX CULVERTS

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

100 YR. EVENT PLAN
B/L STA. 2160+70 TO B/L STA. 2245+00

SHEET
IDENTIFICATION
M2G-L-01

TERREBONNE PARISH, LA.

T 16 S
R 19 E

FLOOD SIDE

PROTECTED SIDE

MATCH LINE
B/L STA. 2245+00

2240+00

2220+00

2200+00

2180+00

2160+69

PI 2160+69

SECT. 65

SECT. 63

SECT. 66

SECT. 16

SECT. 18

N

LEGEND

CENTERLINE OF LEVEE/FLOODWALL

RIGHT-OF-WAY (100 YR. LORR)

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

TERREBONNE PARISH, LA.

MATCH LINE
B/L STA. 2319+58.60

PI 2319+93

SECT. 54

SECT. 37

SECT. 63

SECT. 65

SECT. 67

SECT. 16

PI 2303+73

PI 2298+15

PI 2284+47

2300+00

2280+00

2280+00

PROTECTED SIDE

FLOOD SIDE

T 16 S
R 19 E

SECT. 67

MATCH LINE
C/L STA. 2245+00

N

LEGEND

CENTERLINE OF LEVEE/FLOODWALL

RIGHT-OF-WAY (100 YR. LORR)

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA.

100 YR. EVENT PLAN
B/L STA. 2245+00 TO B/L STA. 2320+00

SHEET
IDENTIFICATION
M2G-L-02

MARK   DATE   APPR.   DESCRIPTION
MARK   DATE   APPR.   DESCRIPTION

TERREBONNE PARISH, LA.

FLOOD SIDE

PROTECTED SIDE

SECT. 67

SECT. 37

SECT. 100

SECT. 36

SECT. 63

T 16 S
R 19 E

T 16 S
R 20 E

MATCH LINE
B/L STA. 2395+74.88

MATCH LINE
B/L STA. 2319+58.60

PI 2379+02

PI 2374+57

PI 2354+20

PI 2351+54

2380+00

2360+00

2340+00

2320+00

N

LEGEND

———— CENTERLINE OF LEVEE/FLOODWALL

———— RIGHT-OF-WAY (100 YR., LORR)

US Army Corps
of Engineers®
NEW ORLEANS DISTRICT

MARK | DATE | APPR. | DESCRIPTION
MARK | DATE | APPR. | DESCRIPTION

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

100 YR. EVENT PLAN
B/L STA. 2320+00 TO B/L STA. 2400+00

SHEET IDENTIFICATION
M2G-L-03

DESIGNED BY:
DATE: 14 DEC. 2012
TYLER DAY TOWNES P.E.

DRAWN BY:
CKD BY:

SUBMITTED BY:
JAMES H. GAUTREAUX

CONTRACT NO:
SOLICITATION NO:

PLOT SCALE:
PLOT DATE: 14 DEC. 2012

FILE NUMBER:
FILE NAME: M2G_L3.dgn

ANSI D
SIZE:

SHEET
IDENTIFICATION
M2G-L-04

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

100 YR. EVENT PLAN
B/L STA. 2400+00 TO B/L STA. 2520+00

LEGEND

CENTERLINE OF LEVEE/FLOODWALL

RIGHT-OF-WAY (100 YR., LORR)

TERREBONNE PARISH, LA.

FLOOD SIDE

PROTECTED SIDE

SECT. 100

T 16 S
R 20 E

T 17 S
R 20 E

MATCH LINE
B/L STA. 2515+54.41

MATCH LINE
B/L STA. 2395+74.88

PI 2451+08
PI 2443+55
PI 2429+76
PI 2416+16

2500+00
2480+00
2460+00
2440+00
2420+00
2400+00

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

N

MARK	DATE	APPR.	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	APPR.	DATE

100 YR. EVENT PLAN
B/L STA. 2520+00 TO B/L STA. 2640+00

SHEET
IDENTIFICATION:
M2G-L-05

LEGEND

— · · — · · — CENTERLINE OF LEVEE/FLOODWALL

— · — · — RIGHT-OF-WAY (100 YR. LORR)

MATCH LINE
B/L STA. 2633+96.54

2620+00

PI 2613+22

2600+00

T 16 S
R 20 E

2580+00

FLOOD SIDE

PROTECTED SIDE

TERREBONNE PARISH, LA.

2560+00

PI 2563+48

2540+00

2520+00

MATCH LINE
B/L STA. 2515+54.41

PI 2527+91

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

100 YR. EVENT PLAN
B/L STA. 2640+00 TO B/L STA. 2760+00

SHEET
IDENTIFICATION
M2G-L-06

MATCH LINE
B/L STA. 2754+47.14

PI 2746+56

2740+00

2720+00

PI 2713+37

2700+00

2680+00

PI 2681+65

2660+00

2640+00

MATCH LINE
B/L STA. 2633+96.54

TERREBONNE PARISH, LA.

T 16 S
R 20 E

FLOOD SIDE

PROTECTED SIDE

LEGEND

———··——··— CENTERLINE OF LEVEE/FLOODWALL

————————— RIGHT-OF-WAY (100 YR. LORR)

N

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	APPR.	DATE	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	APPR.

ANSI D
SIZE:
FILE NAME:
M2G_L-07.dgn

FILE NUMBER:
4001
IN DEC. 2012
PLOT SCALE: PLOT DATE:
X-X-X

CONTRACT NO.:
X
SUBMITTED BY:
JAMES H. GAUTREAUX

SOLICITATION NO.:
X
DWN BY: CKD BY:
T.D.T. X

DATE:
14 DEC. 2012
DESIGNED BY:
TYLER DAY TOWNES, P.E.

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

100 YR. EVENT PLAN
B/L STA. 2760+00 TO B/L STA. 2816+79

SHEET
IDENTIFICATION
M2G-L-07

SECT. 100

SECT. 63

SECT. 62

SECT. 102

TERREBONNE PARISH, LA.

T 17 S
R 20 E

MATCH LINE
B/L STA. 2754+47.14

2760+00

2780+00

2800+00

LIMIT OF WORK
B/L STA. 2810+63.35

PI 2765+43

PI 2775+98

N

LEGEND

CENTERLINE OF LEVEE/FLOODWALL

RIGHT-OF-WAY (100 YR. LORR)

SCALE: 1" = 400'

0   400'   800'   1200'   1600'

400'

SECT. 63

SECT. 62

TERREBONNE PARISH, LA.

T 17 S
R 20 E

LIMIT OF WORK
B/L STA. 2810+63.35

2820+00

2840+00

FLOODWALL

Pl 2863+54

Pl 2862+92

Pl 2861+08

19+17

2860+00

Pl 2869+27
Pl 2870+34
Pl 2872+37
Pl 2873+38
Pl 2878+38
Pl 2882+07

Pl 2885+08
Pl 2884+47
Pl 2881+53

Pl 2887+18

Pl 2889+05

Pl 2893+73
Pl 2893+88

Pl 2894+77
Pl 2895+08
Pl 2896+15

Pl 2900+70

2900+00

Pl 2907+68

Pl 2910+03
Pl 2911+48
Pl 2911+30
Pl 2913+38

MATCH LINE
B/L STA. 2916+65

Pl 2914+81

LEGEND

— · · — · · — CENTERLINE OF LEVEE/FLOODWALL

— · — · — RIGHT-OF-WAY (100 YR. LORR)

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

SECT. 62

T 12 S
R 23 E

TERREBONNE PARISH, LA.

SECT. 46

SECT. 95

SECT. 01

MATCH LINE
B/L STA. 2916+65

2920+00

PI 2921+08

PI 2922+48

PI 2924+56

PI 2924+97

PI 2924+94

PI 2926+92

PI 2929+80

PI 2933+29

PI 2934+99

PI 2937+63

2940+00

FLOODWALL

PI 2946+11

PI 2946+93

PI 2953+00

2960+00

PI 2953+83

PI 2953+82

2980+00

3000+00

MATCH LINE
B/L STA. 3072+80

N

LEGEND

CENTERLINE OF LEVEE/FLOODWALL

RIGHT-OF-WAY (100 YR., LORR)

SCALE: 1" = 400'

400' 0   400'   800'   1200' 1600'

LIMIT OF WORK
B/L STA. 2948+83

PROTECTED SIDE

SECT. 3

SECT. 43

SECT. 42

T 18 S
R 21 E

MATCH LINE
B/L STA. 3072+80

2953+00
2960+00
2980+00
3000+00
3020+00
3040+00
3060+00

PI 3005+18
PI 3005+27
PI 3023+93
PI 3040+67
PI 3053+83
PI 3066+99

T 16 S
R 20 E

TERREBONNE PARISH, LA.

FLOOD SIDE

SECT. 2

SECT. 12

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

N

LEGEND

—··—··— CENTERLINE OF LEVEE/FLOODWALL

—···—···— RIGHT-OF-WAY (100 YR., LORR)

MATCH LINE
B/L STA. 3072+08

PI 3081+11
PI 3087+61
PI 3103+52
3100+00
3090+00
PI 3110+64
PI 3113+80
3120+00

TERREBONNE PARISH, LA.

SECT. 48

FLOOD SIDE

PI 3136+56
PI 3144+53
3140+00

SECT. 54

T 18 S
R 21 E

PI 3160+21
3160+00

PROTECTED SIDE

SECT. 1

SECT. 2

SECT. 3

SECT. 4

PI 3182+08
PI 3191+57
PI 3188+25
PI 3199+50
3180+00
LIMIT OF WORK
B/L STA. 3200+00
798+49.39

LEGEND

CENTERLINE OF LEVEE/FLOODWALL
RIGHT-OF-WAY (100 YR., LORR)

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

SOIL AND GEOLOGIC PROFILE

Reach I - Profile 1

MORGANZA TO THE GULF

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: T. CREASY
DRAWN BY: T. CREASY
CHECKED BY: D. BRITSCH

PLOT SCALE: 1 : 10
PLOT DATE:
DATE: 29 June 2011

PLATE G-1

AL - ARTIFICIAL LEVEE

MARSH - PREDOMINANTLY ORGANIC CLAYS, FAT CLAYS, AND PEATS WITH
OCCASIONAL SAND AND SILT LAYERS

SWAMP - PREDOMINANTLY ORGANIC CLAYS, FAT CLAYS, AND PEATS WITH
OCCASIONAL SAND AND SILT LAYERS WOOD

ABANDONED DISTRIBUTARY - INTERBEDDED LAYERS OF SANDS, SILTS, AND CLAYS

NATURAL LEVEE - PREDOMINANTLY FAT AND LEAN CLAYS AND SILTS WITH SOME SAND
LAYERS

INTRADELTA - GENERALLY COARSER MATERIAL: INTERBEDDED LAYERS OF SILTS,
SILTY SANDS, AND SANDS WITH SOME CLAYS

INTERDISTRIBUTARY - INTERBEDDED LAYERS OF FAT AND LEAN CLAYS, SILTS,
SILTY SANDS, AND SANDS

HORIZONTAL SCALE IN FEET

CH - Fat Clay
CL - Lean Clay
ML - Silt, Low Plasticity
Pt - Peat
Si - Smells
SM - Silty Sand
SP - Sand, Poorly Graded

SOIL AND GEOLOGIC PROFILE

MORGANZA TO THE GULF

Reach I - Profile 2

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: T. CREASY	PLOT SCALE:
DRAWN BY: T. CREASY	PLOT DATE:
CHECKED BY: D. BRITSCH	DATE: 29 JUNE 2011

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FILE NO.

PLATE G-2

DISTANCE BY STATIONS

South   North

MARSH - PREDOMINANTLY ORGANIC CLAYS, FAT CLAYS, AND PEATS WITH OCCASIONAL SAND AND SILT LAYERS

SWAMP - PREDOMINANTLY ORGANIC CLAYS, FAT CLAYS, AND PEATS WITH OCCASIONAL SAND AND SILT LAYERS AND WOOD

NATURAL LEVEE - PREDOMINANTLY FAT AND LEAN CLAYS AND SILTS WITH SOME SAND LAYERS

ABANDONED COURSE - INTERBEDDED LAYERS OF SANDS, SILTS, AND CLAYS

INTRADELTA - GENERALLY COARSER MATERIAL: INTERBEDDED LAYERS OF SILTS, SILTY SANDS, AND SANDS WITH SOME CLAYS

INTERDISTRIBUTARY - INTERBEDDED LAYERS OF FAT AND LEAN CLAYS, SILTS, SILTY SANDS, AND SANDS

HORIZONTAL SCALE IN FEET

CH - Fat Clay
CL - Lean Clay
ML - Silt, Low Plasticity
Pt - Peat
SC - Clayey Sand
Sh - Shells
SM - Silty Sand
SP - Sand, Poorly Graded
Wd - Wood

MAXIMUM BORING PENETRATION

COHESION
TONS / SQ.FT.

WET DENSITY
POUNDS / CU.FT.

LEGEND

MGRI-1U
MGRI-3U
MGRI-5U
MGRI-7U
MGRI-9U
MGRI-11U

⊘ UCT
▨ Q-TEST

ELEVATIONS IN FEET - N.A.V.D.

EL. -15
SM f=30
EL. -25

300
300
600
800
6.6 psf/ft
c/p=0.22

105
95
122
105
100
110

SYMBOLOGY
○ UCT
□ Q-TEST
- - - C/P Line
—— Shear Strength Line

MORGANZA TO THE GULF HURRICANE PROTECTION PROJECT
TERREBONE PARISH, LOUISIANA
LEVEE REACH I-1
BL STA 1254+00 to 1339+00
DENSITY AND SHEAR STRENGTH

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:
DRAWN BY:
CHECKED BY:
PLOT SCALE:
PLOT DATE:
CADD FILE:
FILE NO.
DATE: MAY 2011
H-8-47541

PLATE G-3

DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

LWL EL -1

Existing Bayou
Possible Borrow Pit

Possible Borrow Pit

Borrow Pit

45 FT MIN

48 FT MIN

175 FT

154 FT

VERT 1

36 FT

EL -1

EL 10

EL 14.5

EL 12

EL 22

EL 2

1V:6H

1V:15H

1V:4H

1V:10H

1V:3H

1V:4H

10'

1:4

FABRIC @ EL 4   FILL CH

CH, -1 to -10

CL, -10 to -15

CH, -15 to -25

CH, -25 to -40

CH, -40 to -70

CH, -70 to -100

EL -25

EL -40

EL -70

EL -100

EL -20

Assumes increase in C/L strengths to EL -25 due to previous lifts
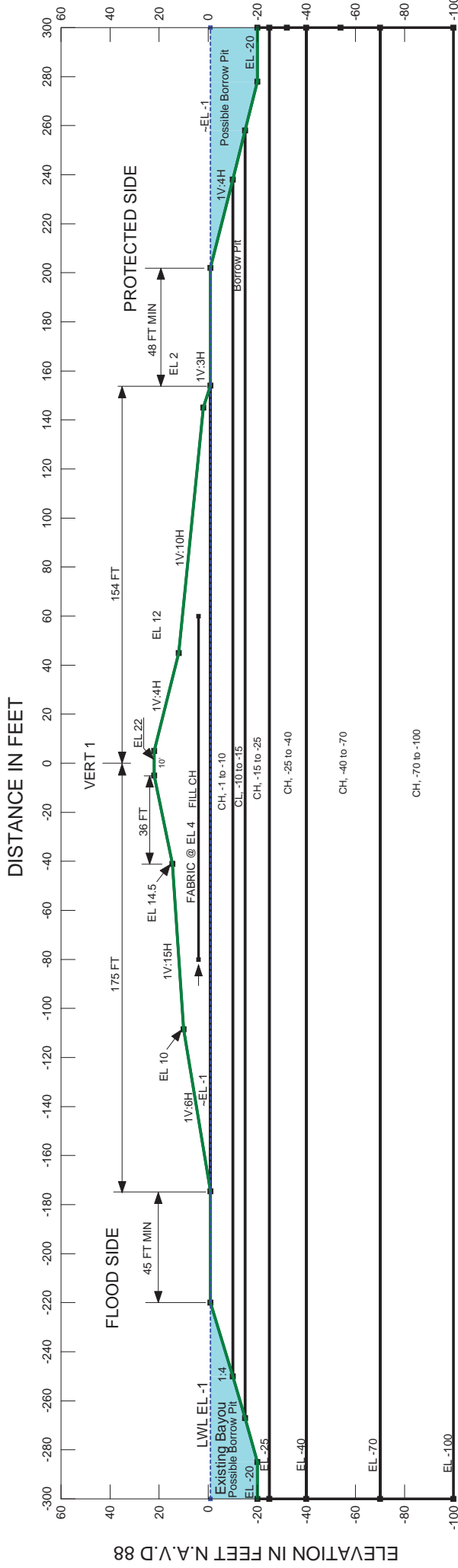
CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach A & Barrier Alignment
Terrebonne Parish, Louisiana

LWL
100-Yr Protection (Base Year plus Overbuild)
NonCircular Analysis - Around Fabric Into Borrow Pit
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach A and Barrier Alignment
100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

**US Army Corps
of Engineers®**
New Orleans District

FIGURE A_LWL-5

DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

VERT. 1 VERT. 2 VERT. 3

BORROW PIT

EL -20.0

SWL EL 13

EL -2

1V:3H

EL -2

EL 8

1V:28H

EL 9.5

1V:16H

EL 14.5

14

EL 20.5

Fabric @ EL 4

14

EL 12

14

14

EL 4.5

14

EL -2

EL -20.0

1V:26H

BORROW PIT

14

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

180 FT of 1800 lbs/in Reinforced Fabric.
100 FT Floodside of C/L
80 FT Protected Side of C/L

Assumes increase in C/L strengths due to previous lifts

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS

Morganza to the Gulf
Levee Reach B
Terrebonne Parish, Louisiana

SWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Thru Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach B, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE B_SWL-3

US Army Corps
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DISTANCE IN FEET

FLOOD SIDE

PROTECTED SIDE

VERT. 1

VERT. 2
C/L

VERT. 3

ELEVATION IN FEET N.A.V.D 88

SWL 16.5

151 FT MIN.

344 FT

381 FT

60 FT

BORROW PIT
EL. -20.0

EL. -1

EL. 4

EL 12

EL 17

EL 25

EL 15

EL. 4

EL. -1

EL. -1.5

Falgot Canal/Borrow Pit
EL. -20.0

1V:4H

1V:25H

1V:15H

1V:29H

39 FT

10

1:5

1:4

1:3

1:4

FAB @ EL -4

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

Assumes increase in C/L strengths due to previous lifts
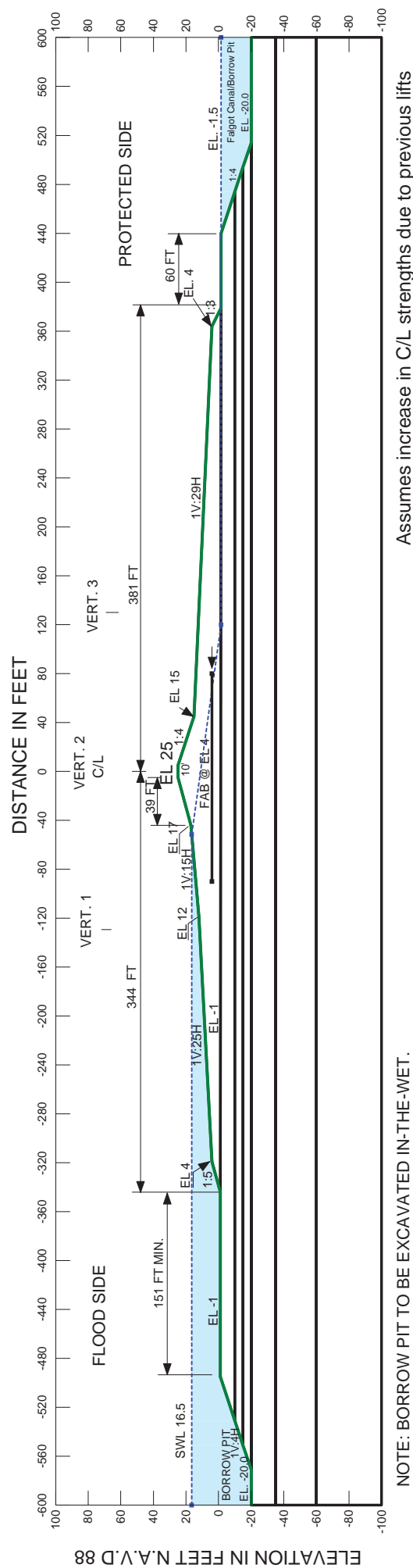
170 Feet Length of 1600 lbs/in Reinforced Geotextile laid at EL 4
From C/L Levee, offset 90 ft to Floodside and 80 ft to Protected Side

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach E
Terrebonne Parish, Louisiana

MORGANZA TO THE GULF
Levee Reach E, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

LWL
100-Yr Protection (Base Year plus Overbuild)
NonCircular Analysis - Thru Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

FIGURE E_SWL-6

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DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

210 FT

150 FT

150 FT

300 FT

VERT. 1

C/L
VERT. 2

VERT. 3

39 FT

SWL 15.5

EL 0.5

1:6

EL 5

1V:13H

1V:4H

BORROW PIT
EL. -15

EL 17

EL 25

1:4

EL 15

EL 11

1V:20H

EL 5

1:4 EL. 0

EXISTING SLOPE

Start of Houma Navigation Canal

HNC Canal
EL. -22.0

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET

* Floodside borrow pit not recommended to presence of near surface silts and sands (pervious strata)

165 Feet Length of 1950 lbs/in Reinforced Geotextile laid at EL 4
From C/L Levee, offset 85 ft to Floodside and 80 ft to Protected Side

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach F
Terrebonne Parish, Louisiana

MORGANZA TO THE GULF
Levee Reach F, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

SWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Around Fabric into Berm
Includes:
Optimization & Tension Crack

US Army Corps
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New Orleans District

FIGURE F_SWL-4

DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

VERT. 1
260 FT
75'

VERT. 2
45 FT

VERT. 3
290 FT

96 FT

69 FT

EL 16

EL 8.5

EL 16.5

EL 26
10'

EL 4

EL 5

1V ON 4H

1V ON 19H

1V ON 4H EL 16

1V ON 19H

1V ON 4H

1:6

1:6

Fabric @ EL 4

EL-1

EL-1

EL-1

EL-1

EL-20

EL-20

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET

* Floodside borrow pit not recommended to presence of near surface silts and sands (pervious strata)

140 FT of 2000 lbs/in Reinforced Fabric
80 FT Floodside of C/L
60 FT Protected Side of C/L

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach G
Terrebonne Parish, Louisiana

SWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Thru Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach G, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE G_SWL-1

US Army Corps of Engineers®
New Orleans District

DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

VERT. 1   VERT. 2   VERT. 3

BORROW PIT

SWL EL 16

EL. -10.0

1V:4H

EL. -1.0

1V:6H

EL. 9.0

1V:15H

EL. 16.5

1V:4H

EL. 25.5

1V:4H

EL. 13.0

Fabric @ EL. 4

1V:17.5H

EL. 5.0

1V:6H

EL. -1.0

BORROW PIT

1V:4H

EL. -20.0
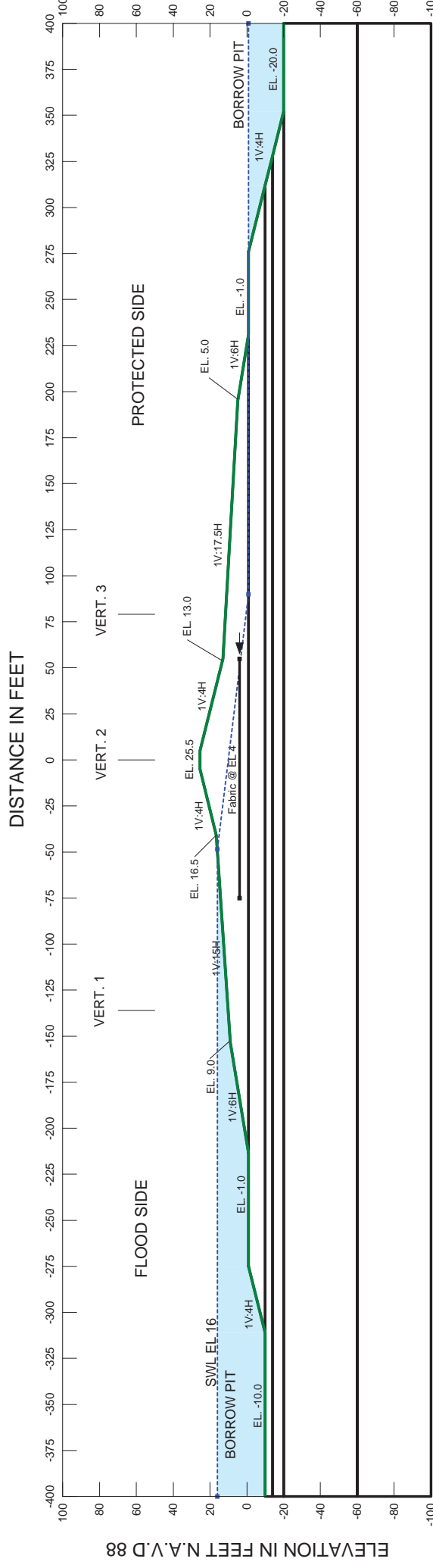
130 FT of 1550 lbs/in Reinforced Fabric
75 FT Floodside of C/L
55 FT Protected Side of C/L

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

Assumes increase in C/L strengths due to previous lifts

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach H1
Terrebonne Parish, Louisiana

SWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Around Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach H1, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE H1_SWL-2

DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

PROTECTED SIDE

FLOOD SIDE

VERT. 1
VERT. 2
VERT. 3

BORROW PIT
BORROW PIT

EL. 17.5
EL. -2.0
EL. -10.0
1:3
1V:6H
EL. 10.0
1V:15H
EL. 18.0
1V:4H
EL. 28.0
1V:4H
Fabric @ EL. 4
EL. 12.0
1V:23H
EL. 5.0
1V:6H
EL. -2.0
1V:3H
EL. 20.0

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.
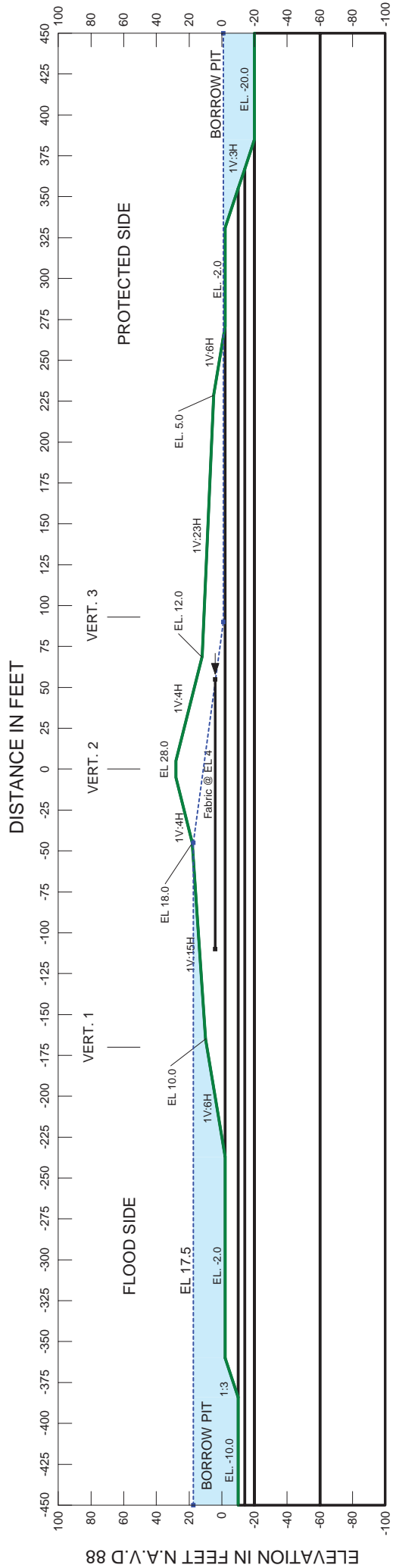
Assumes increase in C/L strengths due to previous lifts

165 FT of 1600 lbs/in Reinforced Fabric
110 FT Floodside of C/L, 55 FT Protected Side of C/L

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach H2 & H3
Terrebonne Parish, Louisiana

SWL
100-Yr Protection (Base Year plus Overbuild)
NonCircular Analysis - Thru Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach H2 & H3, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE H2_SWL-5

US Army Corps
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DISTANCE IN FEET

FLOOD SIDE   PROTECTED SIDE

ELEVATION IN FEET N.A.V.D 88

BORROW PIT*

101 FT MIN

VERT. 1   VERT. 2   VERT. 3

270 FT

155 FT

114 FT MIN

LWL EL -1.0

EL 4

1V:4H

1V:15H

EL 18

1:4

EL 28

10 FT

1:4

EL 13

1V:24H

1V:4H

EL -1

Existing Canal
EL -15

~EL -9
EL -15
EL -19
EL -25

EL -40

EL -70

EL -100

* Floodside borrow pit not recommended
due to presence of near surface silts and
sands (pervious strata)

Assumes increase in C/L strengths to EL -70 due to previous lifts

145 Feet Length of 1900 lbs/in Reinforced Geotextile laid at EL 4
From C/L Levee, offset 75 ft to Floodside and 70 ft to Protected Side

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

Morganza to the Gulf
Levee Reach I
Terrebonne Parish, Louisiana

MORGANZA TO THE GULF
Levee Reach I, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

LWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Around Fabric into Borrow Pit
Includes:
Optimization & Tension Crack

US Army Corps
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New Orleans District

FIGURE I_LWL-3

DISTANCE IN FEET

FLOOD SIDE

PROTECTED SIDE

349 FT

391 FT

VERT. 1

VERT. 2

VERT. 3

EL 28

EL 18

EL 18

1:6

1:4

10 FT

Fabric @ EL 4

EL 5

1:4

1V:20H

LWL EL -1.0

~EL -1

EL 1.5

1:4

Water EL 0

EL -2

1V:20H

ELEVATION IN FEET N.A.V.D 88

Floodside or Protected Side Borrow Pits not recommended due to presence of near surface silts and sands (pervious strata)

Assumes increase in C/L strengths to EL -30 due to previous lifts

170 Feet Length of 2000 lbs/in Reinforced Geotextile laid at EL 4
From C/L Levee, offset 100 ft to Floodside and 70 ft to Protected Side

Morganza to the Gulf
Levee Reach J3
Terrebonne Parish, Louisiana

LWL
100-Yr Protection (Base Year plus Overbuild)
Circular Analysis - Around Fabric into Berm
Includes:
Optimization & Tension Crack

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF
THE SOIL WERE BASED ON THE RESULTS OF
UNDISTURBED BORINGS AND CPT DATA. SEE
BOTH BORING AND CPT DATA PLATES.

SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.

MORGANZA TO THE GULF
Levee Reach J-3, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE J3_LWL-2

US Army Corps
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DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

PROTECTED SIDE

FLOOD SIDE

Wonder Lake
Borrow Pit

EL -1
EL -20

1V:4H

105 FT MIN

~EL -1

1V:4H

EL 4

1V:21H

359 FT

VERT. 3

EL 18

1V:4H

VERT. 2

EL 27
51 FT
10 FT

Fabric @ EL 4

EL 18

1V:15H

296 FT

VERT. 1

1V:6H   EL 4

99 FT MIN

~EL -1

LWL EL -1.0

BORROW PIT*
EL -15   1V:4H

EL -20
EL -25
EL -30

EL -60

EL -100

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

Assumes increase in C/L strengths to EL -60 due to previous lifts

* Limit Floodside borrow pit depth due to presence of near surface silts and sands (pervious strata)

Borrow Pit Availability
Borrow Land Aquisition
Borrow Pit Quantities
Environmental and Mitigation Issues
are not Geotechnical Responsibilities

130 Feet Length of 1500 lbs/in Reinforced Geotextile laid at EL 4
80 Feet Floodside of C/L
50 Feet Protected Side of C/L

LWL Case
Block Analysis - Into Berm Through Fabric
Includes:
Optimization & Tension Crack

US Army Corps
of Engineers®
New Orleans District

MORGANZA TO THE GULF
Levee Reach J, 100 YR LORR (With Overbuild)
Floodside and Protected Side Analyses
Earthen Section With Fabric
TERREBONNE PARISH, LOUISIANA

PLATE

Analysis for 100 Yr LORR (with Overbuild) Lift
Levee Reach J
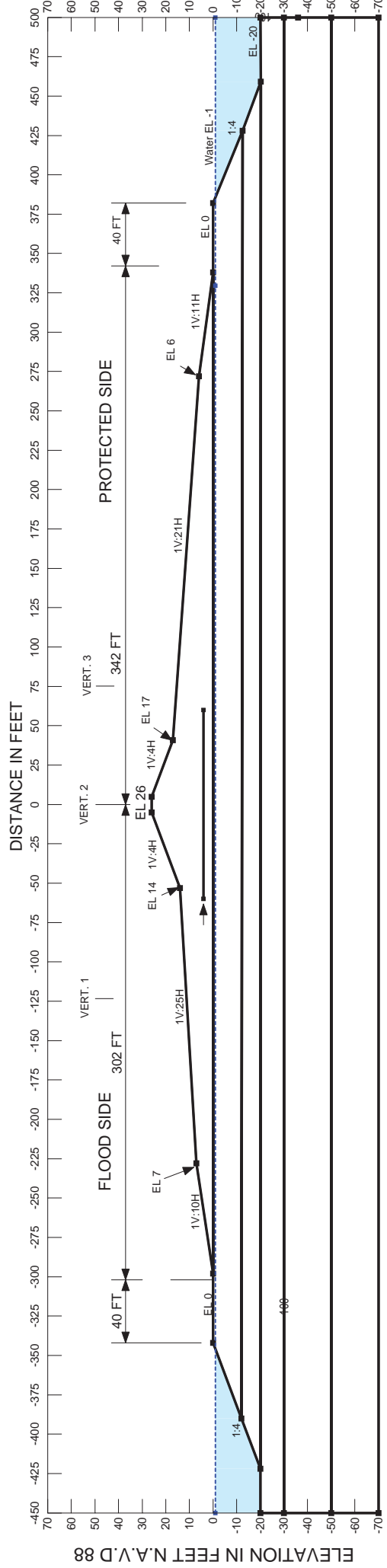
DISTANCE IN FEET

ELEVATION IN FEET N.A.V.D 88

FLOOD SIDE

PROTECTED SIDE

NOTE: BORROW PIT TO BE EXCAVATED IN-THE-WET.

Assumes increase in C/L strengths due to previous lifts

120 Feet Length of 1700 lbs/in Reinforced Geotextile laid at EL 4
From C/L Levee, offset 60 ft to Floodside and 60 ft to Protected Side

Morganza to the Gulf
Levee Reach K&L
Terrebonne Parish, Louisiana

LWL
35-Yr Protection (Base Year plus Overbuild)
NonCircular Analysis - Into Berm
Includes:
Optimization & Tension Crack

MORGANZA TO THE GULF
Levee Reach K&L, 100 YR LORR
Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Earthen Section
TERREBONNE PARISH, LOUISIANA

FIGURE KL_LWL-5

US Army Corps
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New Orleans District

SWL - NonCircular Analysis Around Fabric into Berm
100-Yr Protection (Base Year)
and 35-Yr Protection (Base Year plus Overbuild)
Includes: Optimization & Tension Crack Line

Reinforcement Fabric @ EL 4
100 foot length centered on levee
Fabric with 1200 lbs/in @ 5% Strain

MORGANZA TO THE GULF
Lockport to Larose Levee Reach (a), 100 YR
for Base Year Conditions
And Levee Reach (a), 35 YR
for Base Year Conditions w/Overbuild
Floodside and Protected Side Analyses
Fabric Reinforced Levee Section
LAFOURCHE PARISH, LOUISIANA

SWL-A-2

US Army Corps
of Engineers®
New Orleans District

LWL - NonCircular Analysis Thru Fabric into Berm
100-Yr Protection (Base Year plus Overbuild)
Includes: Optimization & Tension Crack Line

LGM - Levee Reach C-North

Floodside

LGM P/S = MTG F/S

Protected Side

LGM F/S = MTG P/S

TOE — 181 FT — C/L — 286 FT — TOE

Add 15 ft minimum for "Veg Free Zone" plus construction and maintenance zone

1.45 — 86 FT

Reinforcement Fabric @ EL 2
130 foot length
50 foot on NEW PS and 70 foot on NEW FS
Fabric with 2000 lbs/in @5% Strain

US Army Corps of Engineers®
New Orleans District

DISTANCE

ELEVATION

MTG Analysis - Levee Reach C-North (LGM)

Soil Properties Table

Layer (ELEV)	SOIL	FRIC. ANGLE	UNIT WT. (pcf)	Initial Lift			
				Centerline		Toe	
				Top	Bottom	Top	Bottom
Compacted Fill	CH	0	115	600	600	600	600
Existing Fill	CH	0	112	400	400	400	400
Uncompacted Fill	CH	0	100	120	120	120	120
GS to -10	CH	0	100	300	400	200	200
-10 to -22	ML	15	117	200	200	200	200
-22 to -36	CH	0	102	500	600	300	300
-36 to -41	ML	15	117	200	200	200	200
-41 to -52	CH	0	110	600	750	400	500
-52 to -57	ML	15	117	200	200	200	200
-57 to -100	CH	0	105	700	1000	700	1000

Soil Porperty Table Provided for Conveinerence and Actual Properties may have a small variance
For Actual Soil Properies refer to the Slope/W Input/Output Report Files

LEVEE REACH A & BARRIER ALIGNMENT

N.T.S.

NEW LEVEE FILL

EL 13.5 to 20.5

EL -10

CH   UNIT WEIGHT=100 PCF   THICKNESS =10 FEET   Cc/(1+Eo)=0.32

CH   UNIT WEIGHT=114 PCF   THICKNESS =30 FEET   Cc/(1+Eo)=0.22

EL -40

CH (-40 to -100)   UNIT WEIGHT=110 PCF   THICKNESS =60 FEET   Cc/(1+Eo)=0.29

---- DESIGN GRADE ELEVATION (100 YR LORR)

—— PREDICTED CROWN ELEVATION

LIFT SCHEDULE (YR)

X-axis: LIFT SCHEDULE (YR): 2015, 2020, 2030, 2040, 2050, 2060, 2070, 2080, 2085

Y-axis: ELEVATION (FT, NAVD88): 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

LEVEE REACH B

LEVEE FILL

EL +16 to +20.5

N.T.S.

EL -18	CH	UNIT WEIGHT=100 PCF	THICKNESS =18 FEET	Cc/(1+Eo)=0.28
EL -20	SILTY SAND		
EL -80	CH	UNIT WEIGHT=104 PCF	Cc/(1+Eo)=0.39

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

- - - - DESIGN GRADE ELEVATION (100 YR LORR)

——— PREDICTED CROWN ELEVATION

LEVEE REACH E

EL +21 to +23.5

N.T.S.
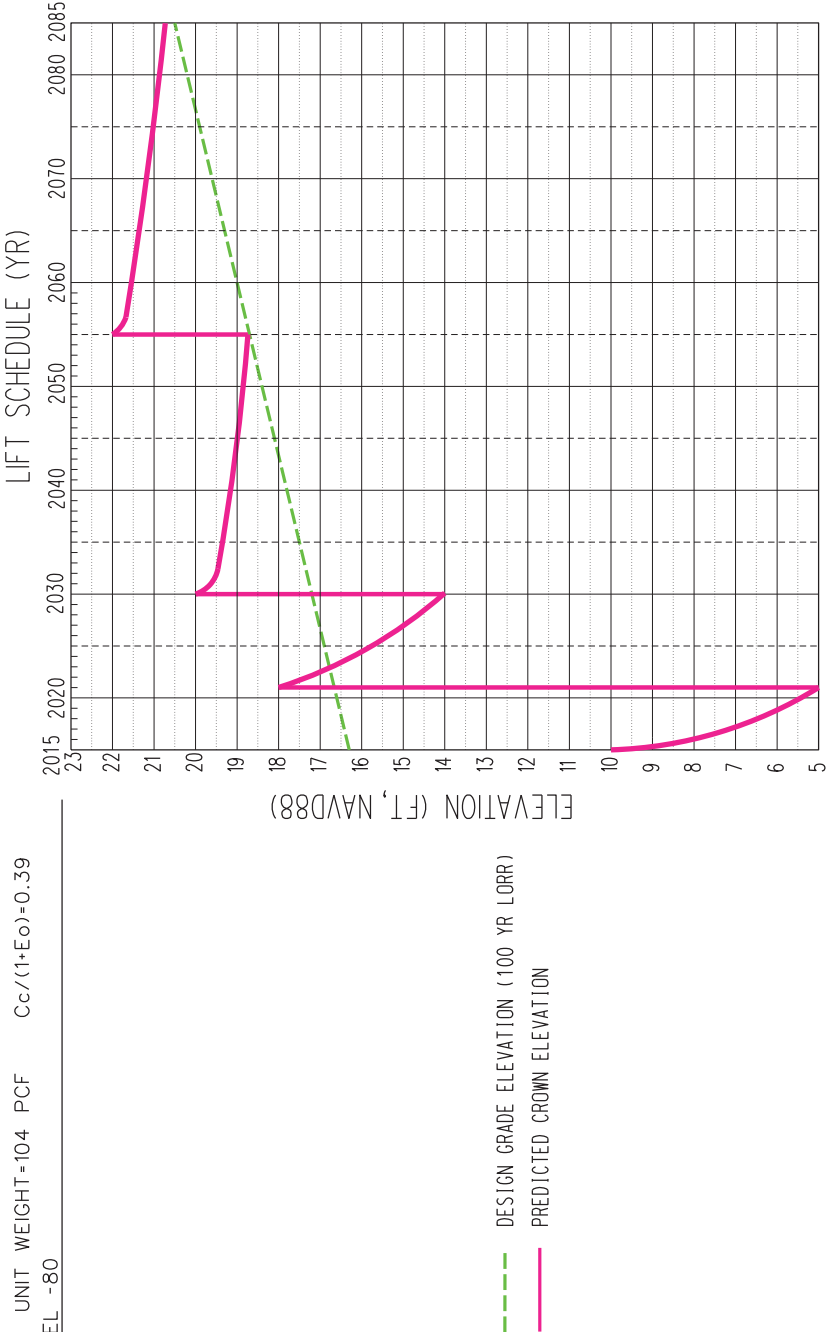
LEVEE FILL
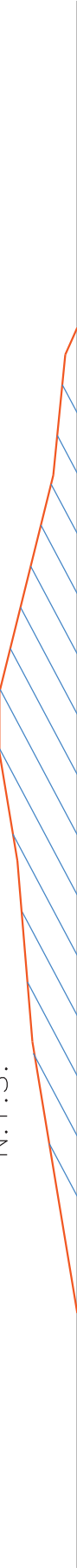
| CH | UNIT WEIGHT=90 PCF | THICKNESS =10 FEET | $Cc/(1+Eo)=0.39$ |

EL -14

EL -20   SILTY SAND

CH UNIT WEIGHT=104 PCF $Cc/(1+Eo)=0.36$

EL -85
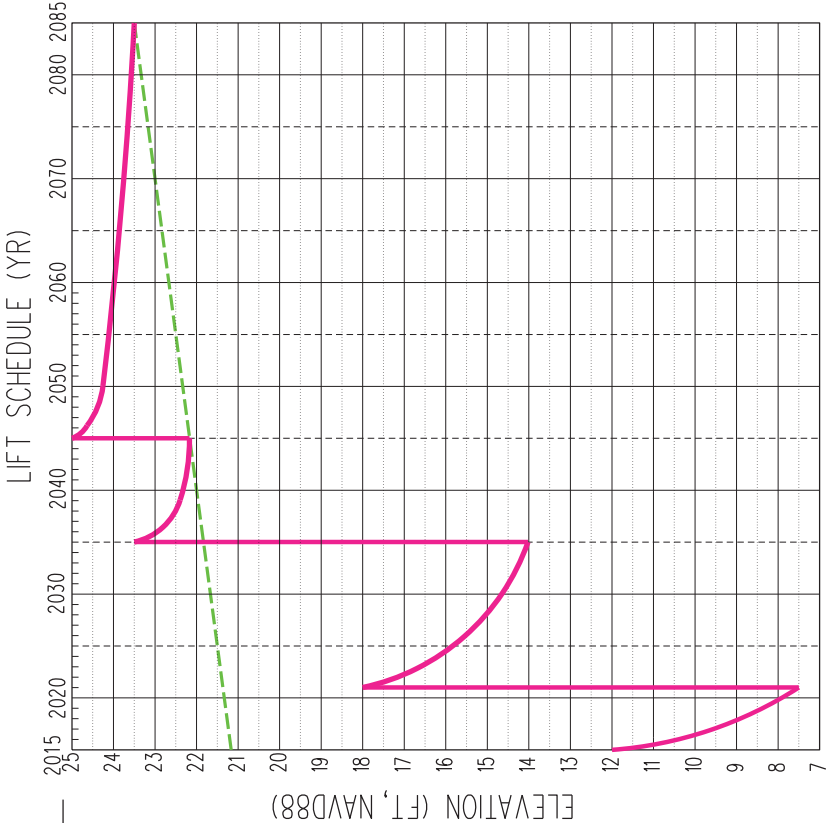
LIFT SCHEDULE (YR)

Y-axis: ELEVATION (FT, NAVD88)

X-axis values: 2015 2020 2030 2040 2050 2060 2070 2080 2085
Y-axis values: 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

- - - DESIGN GRADE ELEVATION (100 YR LORR)

——— PREDICTED CROWN ELEVATION

LEVEE REACH F

LEVEE FILL

EL +21.5 to +23.5

N.T.S.

EL -14 CH UNIT WEIGHT=95 PCF THICKNESS =15 FEET Cc/(1+Eo)=0.32

EL -20 SILTY SAND

CH UNIT WEIGHT=104 PCF Cc/(1+Eo)=0.30
EL -85

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

- - - - DESIGN GRADE ELEVATION (100 YR LORR)
———— PREDICTED CROWN ELEVATION

LEVEE REACH G

EL +22 to +24

N.T.S.
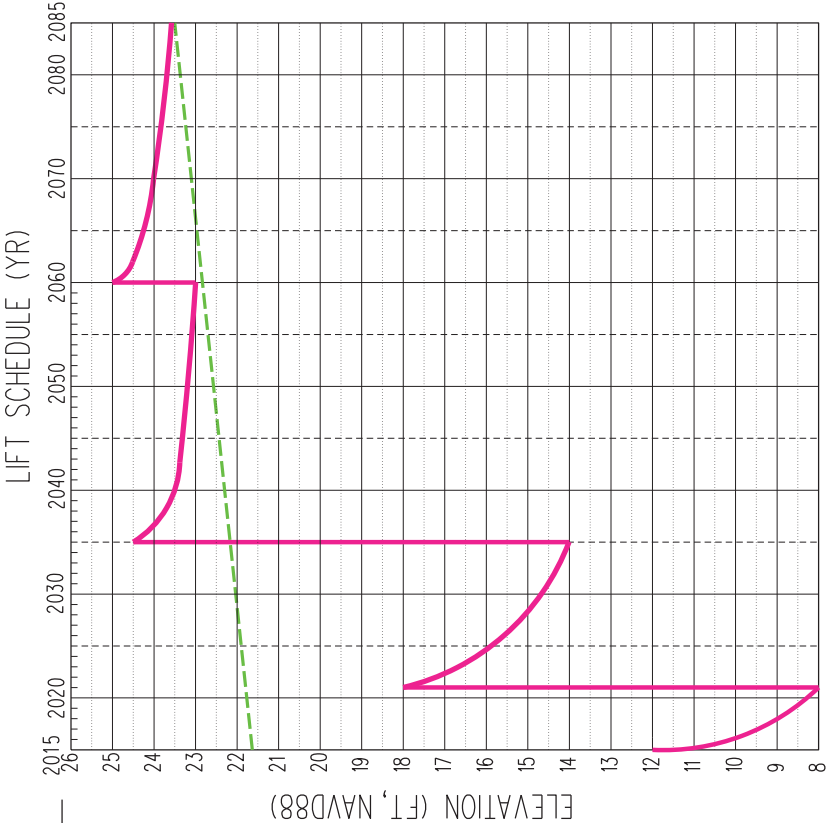
LEVEE FILL
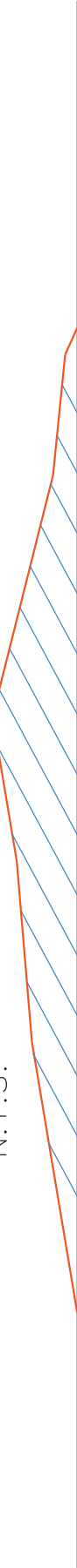
EL -13

EL -20

CH   UNIT WEIGHT=105 PCF   THICKNESS =13 FEET $Cc/(1+Eo)=0.27$

SILTY SAND

CH   UNIT WEIGHT=105 PCF   THICKNESS =60 FT $Cc/(1+Eo)=0.24$

EL -80

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

DESIGN GRADE ELEVATION (100 YR LORR)

PREDICTED CROWN ELEVATION

LEVEE REACH H

EL +23 to + 26.5

N.T.S.

EL -14

EL -20

EL -60

CH   UNIT WEIGHT=108 PCF   THICKNESS =14 FEET $Cc/(1+Eo)=0.31$

SILTY SAND

CH   UNIT WEIGHT=104 PCF $Cc/(1+Eo)=0.32$

CH (-60 to -100)

UNIT WEIGHT=108 PCF   THICKNESS =40 FEET

$Cc/(1+Eo)=0.32$

▨ LEVEE FILL

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88) vs **LIFT SCHEDULE (YR)**

X-axis: 2015 2020 2030 2040 2050 2060 2070 2080 2085

Y-axis: 28 27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10

- - - - DESIGN GRADE ELEVATION (100 YR LORR)

——— PREDICTED CROWN ELEVATION

LEVEE REACH I

LEVEE FILL

N.T.S.

EL +23 to +26.5

EL -20	
EL -25	

CH   UNIT WEIGHT=105 PCF   THICKNESS =20 FT   Cc/(1+Eo)=0.30

SILTY SAND

CH   UNIT WEIGHT=103 PCF   THICKNESS =55 FT   Cc/(1+Eo)=0.23

EL -80

EL -85

CH (-85 to -100)

UNIT WEIGHT=110 PCF   THICKNESS =15 FT   Cc/(1+Eo)=0.23

LIFT SCHEDULE (YR)

X-axis: LIFT SCHEDULE (YR) — 2015 2020 2030 2040 2050 2060 2070 2080 2085

Y-axis: ELEVATION (FT, NAVD88) — 10 to 28

- - - DESIGN GRADE ELEVATION (100 YR LORR)
——— PREDICTED CROWN ELEVATION

LEVEE REACH J

N.T.S.

LEVEE FILL

EL +23 to +26.5

CH   UNIT WEIGHT=100 PCF   THICKNESS =10 FEET   Cc/(1+Eo)=0.36

EL -11

EL -14   SILTY SAND

UNIT WEIGHT=104 PCF THICKNESS =35 FEET   Cc/(1+Eo)=0.36
        CH   EL -50

UNIT WEIGHT=104 PCF THICKNESS =30 FEET   Cc/(1+Eo)=0.41
        CH   EL -80

- - - - DESIGN GRADE ELEVATION (100 YR LORR)
———— PREDICTED CROWN ELEVATION

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88) vs **LIFT SCHEDULE (YR)**

X-axis: 2015, 2020, 2030, 2040, 2050, 2060, 2070, 2080, 2085

Y-axis: 9 to 28

LEVEE REACH K

EL +21.5 to +25.5

N.T.S.

LEVEE FILL

CH   UNIT WEIGHT=100 PCF   THICKNESS =10 FEET $Cc/(1+Eo)=0.36$

EL -10

EL -15

SILTY SAND

UNIT WEIGHT=104 PCF THICKNESS =35 FEET $Cc/(1+Eo)=0.36$
CH   EL -50

UNIT WEIGHT=104 PCF THICKNESS =30 FEET $Cc/(1+Eo)=0.41$
CH   EL -80

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

- - - - DESIGN GRADE ELEVATION (100 YR LORR)
——— PREDICTED CROWN ELEVATION

LEVEE REACH L

N.T.S.

EL +21.5 to +25.5

LEVEE FILL

CH   UNIT WEIGHT=100 PCF   THICKNESS =10 FEET   Cc/(1+Eo)=0.36

EL -10

EL -15   SILTY SAND

UNIT WEIGHT=104 PCF   THICKNESS =35 FEET   Cc/(1+Eo)=0.36
       CH   EL -50

UNIT WEIGHT=104 PCF   THICKNESS =30 FEET   Cc/(1+Eo)=0.41
       CH   EL -80

DESIGN GRADE ELEVATION (100 YR LORR)

PREDICTED CROWN ELEVATION

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

MTG -- Levee Section C-North Enlargement

N.T.S.

LEVEE FILL

EL +18 to +20.5

EXISTING LEVEE

CH

ML

CH

EL +1

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

DESIGN GRADE ELEVATION (100 YR LORR)

PREDICTED CROWN ELEVATION

PLATE: SA-11

Lockport to Larose - Levee Reach (a)

LEVEE FILL

EL +15

N.T.S.

EL +1

UNIT WEIGHT=104 PCF   THICKNESS =28 FEET   Cc/(1+Eo)=0.28

CH   EL -27

UNIT WEIGHT=106 PCF   THICKNESS =43 FEET   Cc/(1+Eo)=0.35

CH   EL -80

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

DESIGN GRADE ELEVATION (100 YR LORR)

PREDICTED CROWN ELEVATION

PLATE SA-12

Lockport to Larose - Levee Reach (b)

N.T.S.

EL +13

EL +1

LEVEE FILL

UNIT WEIGHT=104 PCF THICKNESS =28 FEET   Cc/(1+Eo)=0.28

    CH EL -27

UNIT WEIGHT=106 PCF THICKNESS =43 FEET   Cc/(1+Eo)=0.35

    CH EL -70

LIFT SCHEDULE (YR)

ELEVATION (FT, NAVD88)

— DESIGN GRADE ELEVATION (100 YR LORR)
— PREDICTED CROWN ELEVATION

TIE-IN T-WALLS, SEE SHEETS S-092 TO S-099 FOR DETAILS

C/L LEVEE

A
C-705

NEEDLE GIRDER STORAGE PLATFORM. SEE SHEET S-020 FOR DETAILS

GUIDEWALL, SEE SHEETS S-021 & S-022 FOR DETAILS

56' SECTOR GATE

DOLPHIN WITH NAVIGATION LIGHT SEE SHEET S-023 FOR DETAILS

TIE-IN T-WALLS

A
C-705

SCALE : 1" = 40'

40'   0   40'   80'   120'   160'

N

BAYOU BLACK SG 100 YR
TIE-IN SECTION
SCALE: 1" = 30'-0"
A / C-703A

TYPE 2D   TYPE 2D   TYPE 2C   TYPE 2C   TYPE 2C   TYPE 2A

EL. 20 TOP OF LEVEE
EL. 18 TOP OF SHEETS
EL. -27
PZC-13 SHEET PILE TIE-IN
9" CONCRETE SCOUR PROTECTION
EL. 8
EL. 10
EL. 2
EL. 4
EL. -6
EL. 22.0
EL. -12

30'-0"   50'-0"   50'-0"   70'-0"   30'-0"   130'-0"   55'-0"   55'-0"   130'-0"   30'-0"   70'-0"   50'-0"   50'-0"   30'-0"
280'-0"
134'-0"
280'-0"

BAYOU BLACK SG 35 YR
TIE-IN SECTION
SCALE: 1" = 30'-0"
A / C-703

TYPE 2D   TYPE 2C   TYPE 2A   TYPE 2C   TYPE 2D

EL. 15 TOP OF LEVEE
EL. 13 TOP OF SHEETS
EL. -27
PZC-13 SHEET PILE TIE-IN
9" CONCRETE SCOUR PROTECTION
EL. 6
EL. 8
EL. 0
EL. 4
EL. -6
EL. 15
EL. -12

30'-0"   35'-0"   20'-0"   50'-0"   55'-0"   55'-0"   55'-0"   50'-0"   20'-0"   35'-0"   35'-0"   30'-0"
140'-0"
134'-0"
140'-0"

NOTE:
1. FOR T-WALL GEOMETRY & PILE LAYOUT,
   SEE SHEETS S-094, S-095, S-094A, & S-095A.

MARK | DATE | APPR. | DESCRIPTION

US Army Corps
of Engineers®
NEW ORLEANS DISTRICT

MARK	DATE	APPR.	DESCRIPTION	MARK	DATE	APPR.	DESCRIPTION	DATE	APPR.

FILE NAME:
ANSI D SIZE:
PLOT SCALE: 1:1
PLOT DATE: OCT. 2011
FILE NUMBER: HBK7541
SUBMITTED BY: CHRISTOPHER DUNN, P.E.
DWN BY: CAL
CKD BY: DME
DESIGNED BY: DPL
DATE: OCT-20-2011
CONTRACT NO:
SOLICITATION NO:

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA TO THE GULF OF MEXICO LOUISIANA
REEVALUATION REPORT
TERREBONNE AND LAFOURCHE PARISHES LOUISIANA

SHELL WEST
30' STOP LOG GATE 100 YR
FINAL PLAN

SHEET IDENTIFICATION
C-709A

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

A C-711

TIE-IN T-WALLS,
SEE SHEET S-092 TO S-099
FOR DETAILS

GUIDEWALL, SEE SHEETS
S-021 & S-022 FOR DETAILS

CRANE PLATFORM
T-WALL

BULKHEAD STORAGE
PLATFORM, SEE SHEET
S-015 FOR DETAILS

30' STOP
LOG GATE

DOLPHIN WITH NAVIGATION LIGHT
SEE SHEET S-023 FOR DETAILS

A C-711

MARK | APPR. | DATE | DESCRIPTION | MARK | APPR. | DATE | DESCRIPTION

FILE NAME:
SIZE: ANSI D
FILE NUMBER: H847541
PLOT SCALE: 1:1
PLOT DATE: OCT. 2011
SUBMITTED BY: CHRISTOPHER DUNN, P.E.
DWN BY: CAD
CKD BY: CAL
DESIGNED BY:
DATE: OCT-20-2011
CONTRACT NO:
SOLICITATION NO:

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA TO THE GULF OF MEXICO LOUISIANA
REEVALUATION REPORT
TERREBONNE AND LAFOURCHE PARISHES, LOUISIANA

SHELL CANAL WEST
35' SECTOR GATE
CROSS SECTIONS

SHEET IDENTIFICATION
C-711

SHELL CANAL WEST 30' STOP LOG GATE 100 YR TIE-IN SECTION A C-709
SCALE: 1" = 30'-0"

SHELL WEST 30' STOP LOG GATE 35 YR TIE-IN SECTION A C-709
SCALE: 1" = 30'-0"

NOTE:
1. FOR T-WALL GEOMETRY & PILE LAYOUT,
   SEE SHEETS S-094, S-094A, S-095 & S-095A.

MARK	DATE	APPR.	DESCRIPTION	MARK	DATE	APPR.	DESCRIPTION	APPR.

C/L LEVEE

A
C-748

TIE-IN WALLS, SEE SHEETS
S-092 TO S-099 FOR DETAILS

6-16'X16' SLUICE GATE
STRUCTURE

NEEDLE GIRDER STORAGE
PLATFORM, SEE SHEET S-045
FOR DETAILS.

60'Ø END CELL SEE
SHEET S-049 FOR DETAILS

3-16'X16' SLUICE GATE
STRUCTURE

125' SECTOR
GATE

TIMBER GUIDWALL
SEE SHEET S-049
FOR DETAILS

A
C-748

N

SCALE: 1" = 60'

60'   0   60'   120'   180'   240'

GIWW WEST 125' SG
100 YR TIE-IN SECTION
SCALE: 1' = 20'-0"

A
C-745A

TYPE 2A · TYPE 2A · TYPE 2B · TYPE 2C · TYPE 2D · TYPE 2E

EL. 23
EL. -16
EL. -10
EL. -6
EL. 4
EL. 2
EL. 7.5
EL. 14.5
EL. 12.5

9" CONCRETE SCOUR PROTECTION

PZC-13 SHEET PILE TIE-IN

EL. 22.5 TOP OF LEVEE
EL. 20.5 TOP OF SHEETS
EL. -32

94.0 · 50.0 · 49.0 · 51.0 · 244.0 · 30.0
50'-0" · 50'-0" · 50'-0" · 50'-0" · 50'-0" · 50'-0"

4:1 · 5:1 · 6:1

GIWW WEST 125' SG
100 YR TIE-IN SECTION
SCALE: 1' = 20'-0"

A
C-745A

TYPE 2A · TYPE 2A · TYPE 2B · TYPE 2C · TYPE 2D · TYPE 2E

EL. 23
EL. -16
EL. -10
EL. -6
EL. 4
EL. 2
EL. 7.5
EL. 14.5
EL. 12.5

9" CONCRETE SCOUR PROTECTION

PZC-13 SHEET PILE TIE-IN

EL. 22.5 TOP OF LEVEE
EL. 20.5 TOP OF SHEETS
EL. -32

94.0 · 50.0 · 49.0 · 51.0 · 244.0 · 30.0
50'-0" · 50'-0" · 50'-0" · 50'-0" · 50'-0" · 50'-0"

1:4 · 1:5 · 1:6

SHEET IDENTIFICATION
C-748

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

ANSI D
SIZE:
1:1
HB47041

FILE NAME:
PLOT DATE: OCT. 2011
FILE NUMBER
PLOT SCALE:
SUBMITTED BY:
CHRISTOPHER DUNN, P.E.
CONTRACT NO:
DWN BY: CAL
CKD BY: CAL
DME
SOLICITATION NO:
DATE: OCT-20-2011
DESIGNED BY:
DPL

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA TO THE GULF OF MEXICO LOUISIANA
REEVALUATION REPORT
TERREBONNE AND LAFOURCHE PARISHES LOUISIANA

ENVIRONMENTAL CONTROL STRUCTURE
REACH E
FINAL SITE PLAN 100 YR

A
C-843

TIE-IN T-WALLS,
SEE SHEETS S-092
TO S-099 FOR DETAILS

ENVIRONMENTAL CONTROL STRUCTURE,
SEE SHEETS S-081 TO S-089 FOR DIMENSIONS
& DETAILS 2 TOTAL REACH E STRUCTURES
(TYPICAL SHOWN).

A
C-843

US Army Corps of Engineers
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	APPR.	DATE	MARK	DESCRIPTION	APPR.	DATE

ECS BARRIER 100 YR REACH E TIE-IN SECTION
SCALE: 1" = 20'-0"

A — C-842A

EL. 25.5
EL. 4.5
EL. 4
EL. 2
EL. 7
EL. 9
EL. 13
EL. 11
EL. 18
EL. 16
EL. 25 TOP OF LEVEE
EL. 23 TOP OF SHEETS
EL. -37
EL. -4.5

TYPE 4C
TYPE 4D
TYPE 4E

9" CONCRETE SCOUR PROTECTION

PZC-13 SHEET PILE TIE-IN

VARIES SEE SHEET S-082 FOR DETAILS

ECS BARRIER 100 YR REACH E TIE-IN SECTION
SCALE: 1" = 20'-0"

A — C-842A

EL. 25 TOP OF LEVEE
EL. 23 TOP OF SHEETS
EL. 16
EL. 18
EL. 11
EL. 13
EL. 9
EL. 7
EL. 4
EL. 25.5
EL. -4.5
EL. -37

TYPE 4E
TYPE 4D
TYPE 4C

9" CONCRETE SCOUR PROTECTION

PZC-13 SHEET PILE TIE-IN

ECS BARRIER 35 YR REACH E TIE-IN SECTION
SCALE: 1" = 20'-0"

A — C-842

EL. 15 TOP OF LEVEE
EL. 13 TOP OF SHEETS
EL. 5
EL. 7
EL. 2
EL. 4
EL. -4.5
EL. -46

TYPE 4D
TYPE 4C

9" CONCRETE SCOUR PROTECTION

PZC-13 SHEET PILE TIE-IN

VARIES SEE SHEET S-082 FOR DETAILS

NOTES:

1.) ALL H PILES AND SHEET PILES SHALL MEET THE REQUIREMENTS OF ATM A572, GRADE 50.

2.) ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.

3.) ALL HANDRAIL SHALL BE 2"Ø ALUMINUM PIPE ALLOY 6061-T6 WITH FLANGE FITTINGS.

LOCATION	"A"	"B"	"C"	"D"	"E"
MINORS CANAL	23.0	-9.0	-17.0	-124.0	-50
FALGOUT CANAL	23.0	-9.0	-17.0	-116.0	-62
BAYOU DULARGE	25.5	-7.0	-15.0	-116.0	-69
BAYOU GRAND CAILLOU	25.5	-12.0	-21.5	-128.5	-54
SHELL CANAL EAST	23.5	-12.0	-21.5	-111.0	-46
BAYOU BLACK	22.0	-12.0	-21.5	-111.0	-46
PLACID CANAL	31.5	-8.0	-18.5	-160.0	-69
BAYOU PETITE	30.5	-8.0	-18.0	-157.0	-79
GRAND BAYOU	29.5	-9.0	-19.0	-151.0	-63
POINT AUX CHENES	33.0	-6.0	-16.0	-137.0	-79
BAYOU TERREBONNE	33.0	-9.0	-20.0	-67.0	-73
HUMBLE CANAL	33.0	-20.0	-29.5	-129.5	-103
BUSH CANAL	33.0	-9.0	-24.0	-124.0	-71
LAROSE FG	17.0	-12.3	-19.8	-126.0	-46

56' SECTOR GATE - ELEVATIONS - TRANSVERSE

SCALE: 3/16" = 1'-0"

14' x14' PRECAST CONCRETE CONTROL HOUSE

MACHINERY RECESS

2" Ø ALUMINUM HANDRAIL

EL. "A"

3'-6"

39'-0"

56'-0"

39'-0"

EL. "B" TOP OF SLAB

EL. "C" BOTTOM OF SLAB

1' STABILIZATION SLAB

HP 14x73

TIP EL. "D"

PZC-13 SHEET PILE CUT-OFF WALL

9" EMBED. (TYP.)

TIP EL. "E"

PZC-13 SHEET PILE CUT-OFF WALL

HP 14x73

HP 14x73 BATTERED 1H ON 3 V

PLAN - PILING LAYOUT

SCALE: ³⁄₁₆" = 1'-0"

LOCATION	PILE CUTOFF EL.	PILE TIP EL.	SHEET PILE CUTOFF EL.	SHEET PILE TIP EL.
MINORS CANAL	+16.25	-124.0	+16.25	-50
FALGOUT CANAL	+16.25	-116.0	+16.25	-62
BAYOU DULARGE	-14.25	-116.0	-14.25	-69
BAYOU GRAND CAILLOU	-20.75	-128.5	-20.75	-54
SHELL CANAL EAST	-20.75	-111.0	-20.75	-46
BAYOU BLACK	-20.75	-111.0	-20.75	-46
PLACID CANAL	-17.75	-160.0	-17.75	-69
BAYOU PETITE	-17.25	-157.0	-17.25	-79
GRAND BAYOU	-18.25	-151.0	-18.25	-63
POINT AU CHENES	-15.25	-137.0	-15.25	-79
BAYOU TERREBONNE	-19.25	-167.0	-19.25	-73
HUMBLE CANAL	-19.25	-129.5	-19.25	-103
BUSH CANAL	-23.25	-124.0	-23.25	-71
LAROSE FG	-19.05	-126.0	-19.05	-46

FLOOD SIDE

SHEET PILE CUT-OFF WALL

PZC-13 SHEET PILE CUT-OFF WALL

LEGEND

HP 14x73 BATTERED 1H ON 3V

INDICATES DIRECTION OF BATTER

NOTES:
1. ALL PILES ON THE FLOOD SIDE OF THE SHEET PILE CUTOFF WALL RECEIVE TENSION ANCHORS.
2. ALL H PILES SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.

US Army Corps of Engineers
NEW ORLEANS DISTRICT

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA TO THE GULF OF MEXICO LOUISIANA
REEVALUATION REPORT
TERREBONNE AND LAFOURCHE PARISHES LOUISIANA
100 YEAR

56' SECTOR GATE
FOUNDATION PLAN

DESIGNED BY:
SUBMITTED BY: CHRISTOPHER DUNN, P.E.
DWN BY:
CKD BY:
DATE: OCT. 2011
SOLICITATION NO:
CONTRACT NO:
PLOT DATE:
PLOT SCALE:
FILE NUMBER: H-8-47561
FILE NAME: MtRT561_S-005A—.dgn
SIZE:
ANSI D

SHEET IDENTIFICATION
S-005A

125' SECTOR GATE MASONRY PLAN
SCALE: 3/16" = 1'-0"

117'-8"

B S-027
A S-027
C S-026

R=67'-6½" TO FACE OF SKIN PLATE

14' x 14' PRECAST CONCRETE CONTROL HOUSE

SUMP PIT (TYP.)

C/L HINGE & PINTLE

NEEDLE RECESS

NEEDLE GIRDER RECESS (TYP.)

310'-6⅞6"

155'-3⅝6"

125'-0"

20'-0"

20'-0"

4'-9" (TYP.)

GATE STOP (TYP.)

54'-7⅝6"

SECTION A

SCALE: 1/16"=1'-0"   S-020

EL. 23.0 GIWW WEST
EL. 21.5 GIWW EAST

10'-4"   8'-0"   12'-0"
62'-6"   62'-6"

14'x14' CONTROL HOUSE
MACHINERY BLOCK
NEEDLE GIRDER RECESS EL. 4.0
MACHINERY RECESS
EL. 18.75
EL. -16.0
EL. -28.0

PZC-13 SHEETPILE CUTOFF WALL
EMBEDMENT (TYP.) 1'-0"

TOP OF SLAB EL. -16.0
BOTTOM OF SLAB EL. -28.0

1' STABILIZATION SLAB
24" PIPE PILE BATTERED 1H:4V

6'-6"
14'x14' CONTROL HOUSE
MACHINERY BLOCK

℄ STRUCTURE

SECTION B

SCALE: 1/16"=1'-0"   S-026

EL. 23.0 GIWW WEST
EL. 21.5 GIWW EAST

20'-0"
62'-6"

HINGE RECESS EL. 11.0
THRUST BLOCK
PINTLE RECESS EL. -3.0
EL. -19.5
9'-6"
EL. -16.0

EMBEDMENT (TYP.) 1'-0"
PZC-13 SHEETPILE CUTOFF WALL
EL. -28.0

20'-0"
THRUST BLOCK
DEWATERING SUMP

TOP OF SLAB EL. -16.0
BOTTOM OF SLAB EL. -28.0

1' STABILIZATION SLAB
24" PIPE PILE BATTERED 1H:4V
14'x14' CONTROL HOUSE

℄ STRUCTURE

SECTION C

SCALE: 1/16"=1'-0"   S-026

EL. 23.0 GIWW WEST
EL. 21.5 GIWW EAST

NEEDLE GIRDER RECESS EL. 4.0

20'-7½"
6'⅞"
HINGE RECESS EL. 18.0
PINTLE RECESS EL. -3.0
1'-9"
EL. -19.5
8'-5"

14'-0"
10'-6"
8'-10"
6'-6"
8'-9½"

MACHINERY RECESS EL. 18.75
NEEDLE GIRDER RECESS EL. 4.0
3'-0"

TOP OF SLAB EL. -16.0
BOTTOM OF SLAB EL. -28.0

1' STABILIZATION SLAB
24" PIPE PILE BATTERED 1H:4V
PZC-13 SHEETPILE CUTOFF WALL
EMBEDMENT (TYP.) 1'-0"

FOUNDATION PLAN
SCALE: 3/16" = 1'-0"

PROTECTED SIDE

FLOOD SIDE

5 SPS @ 12'-0" = 60'-0"
4 SPS @ 10'-0" = 40'-0"
8 SPS @ 13'-1½" = 105'-0"
4 SPS @ 10'-0" = 40'-0"
5 SPS @ 12'-0" = 60'-0"

PZC-13 SHEET PILE CUT OFF WALL

43'-1"
35'-0"
8'-0"
15'-9"
8 SPS @ 7'-10⅝" = 63'-0"
4 SPS @ 7'-10⅝" = 31'-6"

LEGEND

24" PIPE PILE BATTERED 1H ON 4V

INDICATES DIRECTION OF BATTER

NOTES:
1. ALL PILES SHALL RECEIVE TENSION ANCHORS.
2. ALL PIPE PILES SHALL MEET THE REQUIREMENTS OF API 5L, GRADE X 52.
3. SHEET PILING SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.

LOCATION	PILE CUTOFF EL.	PILE TIP EL.	PILE LENGTH	SHEET PILE CUTOFF EL.	SHEET PILE TIP EL.
GIWW EAST	-27.0	-166.0	143.3	-26.75	-57.00
GIWW WEST	-27.0	-166.0	143.3	-26.75	-54.00

DIMENSION'S

STRUCTURE	1 "A"	"B"	"C"	"D"	"E"	2 "A"	"B"	"C"	"D"	"E"
BAYOU GRAND CAILLOU	6	117'	3'	18	6'-4"	3	117'	3'	18	6'-4"
BUSH CANAL	6	117'	3'	24	4'-9"	3	60'	3'	24	4'-9"
FALGOUT CANAL	6	117'	3'	18	6'-4"	3	60'	3'	18	6'-4"
GRAND BAYOU	6	117'	3'	24	4'-9"	3	60'	3'	24	4'-9"
GIWW WEST	6	117'	3'	18	6'-4"	3	60'	3'	18	6'-4"
GIWW EAST	3	60'	3'	9	6'-4"				9	6'-4"
PLACID CANAL	3	60'	3'	12	4'-9"				12	4'-9"
BAYOU PETITE CAILLOU	3	60'	3'	12	4'-9"				12	4'-9"

ELEVATION'S

STRUCTURE	"F"	"G"	"H"	"I"	"J"	"K"
BAYOU GRAND CAILLOU	EL. 28.0	EL. 25.5	EL. +4.0	EL. -12.0	EL. -20.0	EL. -116.0
BUSH CANAL	EL. 35.5	EL. 33.0	EL. +4.0	EL. -12.0	EL. -22.0	EL. -126.5
FALGOUT CANAL	EL. 25.5	EL. 23.0	EL. +3.0	EL. -9.0	EL. -17.0	EL. -103.0
GRAND BAYOU	EL. 32.0	EL. 29.5	EL. +3.0	EL. -9.0	EL. -19.0	EL. -151.0
GIWW WEST	EL. 25.5	EL. 23.0	EL. +3.0	EL. -13.0	EL. -21.0	EL. -130.0
GIWW EAST	EL. 24.0	EL. 21.5	EL. +2.0	EL. -10.0	EL. -18.0	EL. -131.5
PLACID CANAL	EL. 34.0	EL. 31.5	EL. +4.0	EL. -8.0	EL. -18.0	EL. -147.0
BAYOU PETITE CAILLOU	EL. 33.0	EL. 30.5	EL. +4.0	EL. -8.0	EL. -18.0	EL. -148.0

SLUICE GATE FLOOD SIDE ELEVATION

SCALE: 1/8" = 1'-0"

EL. "F"
EL. "G"
EL. "H"
EL. "I"
EL. "J"
TIP EL. "K"

3'-6"
1'-9"
2" Ø SCH. 80 ALUMINUM HANDRAILS
HP 14x89 (TYP.)
1" THICK STABILIZATION SLAB

16'x16' OR 16'x12' MANUFACTURED CAST STEEL SLUICE GATE AND THIMBLE

"A" SLUICE GATE BAYS = "B"

ELECTRIC ACTUATOR SEE NOTES

"C" (TYP.)

5'-0" (TYP.)

"D" SPACES @ "E"

16'-0" (TYP.)

1'-0" (TYP.)
1'-6"

NOTES:
1.) SLUICE GATES FOR BAYOU GRAND CAILLOU, BUSH CANAL, & GIWW WEST SHALL BE 16' x 16' W/ A BIFFI ICON U50/1440-173 ACTUATOR W/ A DIAMOND GEAR 105BG7 BEVEL GEAR W/ 4.9' DIAMETER, OR APPROVED EQUAL.
SLUICE GATES FOR FALGOUT CANAL, GRAND BAYOU, & GIWW EAST SHALL BE 16' x 12' W/ A BIFFI ICON 040/720-173 ACTUATOR W/ A DIAMOND GEAR 57BG6 BEVE GEAR W/ 3.7' DIAMETER OR APPROVED EQUAL.
2.) ALL H PILES & SHEET PILES SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.
3.) ALL CONCRETE SHALL HAVE A MIN. 28 DAY COMPRESSIVE STRENGTH OF 4,000 PSI.
4.) ALL HANDRAILS SHALL BE 2" ALUMINUM PIPE ALLOY 6061-T6 W/ FLANGE FITTINGS.

MARK	DATE	APPR.	DESCRIPTION	MARK	DATE	APPR.	DESCRIPTION	DATE	APPR.

ELEVATION'S

STRUCTURE	"A"	"B"	"C"	"D"	"E"	"F"	"G"
BAYOU GRAND CAILLOU	EL. 28.0	EL. 25.5	EL. +4.0	EL. -12	EL. -20.0	EL. -116.0	EL. -54.0
BUSH CANAL	EL. 35.5	EL. 33.0	EL. +4.0	EL. -12	EL. -22.0	EL. -126.5	EL. -71.0
FALGOUT CANAL	EL. 25.5	EL. 23.0	EL. +3.0	EL. -9	EL. -17.0	EL. -103.0	EL. -62.0
GRAND BAYOU	EL. 32.0	EL. 29.5	EL. +3.0	EL. -9	EL. -19.0	EL. -151.0	EL. -63.0
GIWW WEST	EL. 25.5	EL. 23.0	EL. +3.0	EL. -13	EL. -21.0	EL. -130.0	EL. -54.0
GIWW EAST	EL. 24.0	EL. 21.5	EL. +2.0	EL. -10	EL. -18.0	EL. -131.5	EL. -57.0
PLACID CANAL	EL. 34.0	EL. 31.5	EL. +4.0	EL. -8	EL. -18.0	EL. -147.0	EL. -69.0
BAYOU PETITE CAILLOU	EL. 33.0	EL. 30.5	EL. +4.0	EL. -8	EL. -18.0	EL. -148.0	EL. -79.0

NOTES:
1.) SLUICE GATES FOR BAYOU GRAND CAILLOU, BUSH CANAL, & GIWW WEST SHALL BE 16' x 16' W/ A BIFFI ICON 050/1440-173 ACTUATOR W/ A DIAMOND GEAR 105BG7 BEVEL GEAR W/ 4.9" DIAMETER, OR APPROVED EQUAL.
SLUICE GATES FOR FALGAUT CANAL, GRAND BAYOU, PLACID CANAL, BAYOU PETITE CAILLOU & GIWW EAST SHALL BE 16' x 12' W/ A BIFFI ICON 040/720-173 ACTUATOR W/ A DIAMOND GEAR 57BG6 BEVE GEAR W/3.7" DIAMETER OR APPROVED EQUAL.
2.) ALL H PILES & SHEET PILES SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.
3.) ALL CONCRETE SHALL HAVE A MIN. 28 DAY CPMPRESSIVE STRENGTH OF 4,000 PSI.
4.) ALL HANDRAILS SHALL BE 2" ALUMINUM PIPE ALLOY 6061-T6 W/ FLANGE FITTINGS.

FLOOD SIDE

PROTECTED SIDE

EL. 7.0

BULKHEAD RECESS

ELECTRIC ACTUATOR SEE NOTES

2" Ø SCH. 80 ALUMINUM HANDRAILS

20'-0"
50'-0"
10'-0"
20'-0"

3'-6"
9"
5'-0"
2'-0"
2'-6"

EL. "A"
EL. "B"
EL. "C"

18'-3"
18'-9"

BULKHEAD RECESS

16' x 16' MANUFACTURED CAST STEEL OR SST SLUICE GATE AND THIMBLE

"6 SPACES @ 7'-6" = 45'-0"

2'-6"
9" (TYP.)

EL. "D"
EL. "E"
EL. "F"
EL. "G"

2'-6"

HP 14x89 (TYP.)
PZC-13 SHEET PILE
1'-0" THICK STABILIZATION SLAB

SECTION A
S-054
SCALE: 1/4" = 1'-0"

SLUICE GATE PILE FOUNDATION PLAN

SCALE: $1/8$" = 1'-0"

PROTECTED SIDE

FLOOD SIDE

PZ-22 SHEET PILE CUTOFF WALL

"A" SPACES @ "B"

35'-0"   15'-0"

"C"

2'-6"   9 SPACES @ 5'-0" = 45'-0"   2'-6"

HP 14x89 (TYP.)

DIMENSIONS

STRUCTURE	1 "A"	1 "B"	1 "C"	2 "A"	2 "B"	2 "C"	PILE CUTOFF	PILE TIP EL.	PILE LENGTH	SHEET PILE CUTOFF LENGTH	SHEET PILE TIP EL.
BAYOU GRAND CAILLOU	18	6'-4"	1'-6"	9	6'-4"	1'-6"	-19.25	-116.0	101.9	-19.25	-54.0
BUSH CANAL	24	4'-9"	1'-6"	12	4'-9"	1'-6"	-21.25	-126.5	110.9	-21.25	-71.0
FALGOUT CANAL	18	6'-4"	1'-6"	9	6'-4"	1'-6"	-16.25	-103.0	91.4	-16.25	-62.0
GRAND BAYOU	24	4'-9"	1'-6"	12	4'-9"	1'-6"	-18.25	-151.0	139.9	-18.25	-63.0
GIWW WEST	18	6'-4"	1'-6"	9	6'-4"	1'-6"	-20.25	-130.0	115.7	-20.25	-54.0
GIWW EAST	9	6'-4"	1'-6"	9	6'-4"	1'-6"	-17.25	-131.5	120.4	-17.25	-57.0
PLACID CANAL	12	4'-9"	1'-6"	12	4'-9"	1'-6"	-17.25	-147.0	136.8	-17.25	-69.0
BAYOU PETITE CAILLOU	12	4'-9"	1'-6"	12	4'-9"	1'-6"	-17.25	-148.0	137.8	-17.25	-79.0

NOTES:
1.) ALL PILES ON THE FLOOD SIDE OF THE SHEET PILE CUTOFF WALL RECEIVE TENSION ANCHORS
2.) ALL H PILES & SHEET PILE SHALL MEET THE REQUIREMENTS OF ATSM A572, GRADE 50.

LEGEND

BATTERED 3V; 1H HP 14x89 STEEL PILE
BOX INDICATES DIRECTION OF BATTER

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

SHEET
IDENTIFICATION
S-061

PROTECTED SIDE

FLOOD SIDE

50'-0"

2'-6"

28'-0"

20'-0"

25'-0"

5'-0"

4'-0"

1'-3"

NEEDLE GURIDER
RECESS (TYP.)

STOP LOG SLOT

NEEDLE RECESS (TYP.)

MASONRY PLAN
SCALE: 3/8" = 1'-0"

PROTECTED SIDE

FLOOD SIDE

PZC-13 SHEET PILE CUT OFF WALL

2'-6"

2'-6"

2'-6"

2'-6"

6 SPACES @ 7'-6" = 45'-0"

4 SPACES @ 5'-9" = 23'-0"

FOUNDATION PLAN
SCALE: ⅜" = 1'-0"

LEGEND

☐ - HP 14x73
BATTERED 1H ON 3V

INDICATES DIRECTION
OF BATTER

NOTES:

1.) ALL PILES ON THE FLOOD SIDE OF THE SHEET
PILE CUT OFF WALL RECEIVE TENSION ANCHORS.

2.) ALL H-PILES AND SHEET PILES SHALL MEET THE
REQUIREMENTS OF ASTM A572, GRADE 50.

MARK | DESCRIPTION | DATE | APPR. | MARK | DESCRIPTION | DATE | APPR.

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

NOTES:

1. SEE SHEET S-089 FOR THE SCHEDULE OF VARIABLES.

2. PROVIDE A PORTABLE OPERATOR AND GENERATOR FROM THE MANUFACTURER OF THE HAND-CRANK OPERATOR, WHICH WILL BE USED TO LIFT THE SLUICE GATES. THE OPERATOR SHALL BE MANUFACTURED BY THE MANU-FACTURER OF THE SLUICE GATE. THE OPERATOR SHALL INCLUDE REVERSING FEATURES, AN OVERLOAD RELEASE CLUTCH, AND ALL NECESSARY ADAPTORS TO OPERATE THE SLUICE GATES. THE GENERATOR SHALL BE PORTABLE, GAS OPERATED AND ABLE TO HANDLE THE POWER REQUIREMENTS OF THE PORTABLE OPERATOR.

3. ALL H PILES AND SHEET PILES SHALL MEET THE REQUIREMENTS OF ATM A572, GRADE 50.

4. ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.

5. ALL HANDRAIL SHALL BE 1 1/2"Ø ALUMINUM PIPE ALLOY 6061-T6 WITH FLANGE FITTINGS.

FLOOD SIDE ELEVATION
OF DRAINAGE STRUCTURE

SCALE: 3/4" = 1'-0"

CL STEM AND GATE (TYP.)

HAND OPERATED LIFTING DEVICE, (TYP.) SEE NOTE 2.

EL. "A"

EL. "B"

EL. "C"

EL. "X"

SHEETPILE PZC-13

CL STRUCTURE

"J" NUMBER OF GATES

"E" x "F" SLUICE GATE (TYP.)

EL. "D"

HP 14x73 PILE (TYP)

3'-0" (TYP.)

"M" SPACES @ "N"

"L"

1 1/2" DIA. ALUMINUM GUARDRAIL

WELDED STEEL GRATING

W 12x40 (TYP.)

"K" NUMBER OF GATES

"E" (TYP.)

1'-0" THICK STABILIZATION SLAB

3'-6"

"L"

DRAINAGE STRUCTURE SECTION

SCALE: 3/16" = 1'-0"

PROTECTED SIDE

FLOOD SIDE

NOTES:

1. SEE SHEET S-089 FOR THE SCHEDULE OF VARIABLES.

2. PROVIDE A PORTABLE OPERATOR AND GENERATOR FROM THE MANUFACTURER OF THE HAND-CRANK OPERATOR, WHICH WILL BE USED TO LIFT THE SLUICE GATES. THE OPERATOR SHALL BE MANUFACTURED BY THE MANU- FACTURER OF THE SLUICE GATE. THE OPERATOR SHALL INCLUDE REVERSING FEATURES, AN OVERLOAD RELEASE CLUTCH, AND ALL NECESSARY ADAPTORS TO OPERATE THE SLUICE GATES. THE GENERATOR SHALL BE PORTABLE, GAS OPERATED AND ABLE TO HANDLE THE POWER REQUIREMENTS OF THE PORTABLE OPERATOR.

3. ALL H PILES AND SHEET PILES SHALL MEET THE REQUIREMENTS OF ATM A572, GRADE 50.

4. ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4000 PSI.

5. ALL HANDRAIL SHALL BE 1 1/2"Ø ALUMINUM PIPE ALLOY 6061-T6 WITH FLANGE FITTINGS.

Labels in drawing:

HAND OPERATED LIFTING DEVICE, SEE NOTE 2.

1½" DIA. ALUMINUM GUARDRAIL

STEM

STEM GUIDE

SLUICE GATE

BULKHEAD RECESS (TYP) SEE SHEET S-086

TRASH RACK RECESS (TYP) SEE SHEET S-088

1'-0" THICK STABILIZATION SLAB

HP 14x73 PILE (TYP.)

PZC-13 SHEETPILE CUTOFF

TIP EL. "X"
TIP EL. "D"
EL. "A"
EL. "B"
EL. "C"
EL. 5.5
EL. 4.5
EL. 3.5
EL. 3.75

15'-0"
13'-0"
12'-3⅛"
11'-0"
5'-0"
4'-9"
2'-6"
2'-0"
1'-9"
1'-0"
9" (TYP.)
64'-0"

"P" SPACES @ "H"
"G" SPACES @ "H"
"E"
"H"
"L"

3 / 1

DRAINAGE STRUCTURE FOUNDATION PLAN
SCALE: 3/16" = 1'-0"

NOTES:
1. SEE SHEET S-089 FOR THE SCHEDULE OF VARIABLES.
2. FOR DRAINAGE STRUCTURE SECTION SEE SHEET S-084.
3. FOR WINGWALL DETAILS SEE SHEET S-087.
4. ALL PILES ARE HP 14x73, LENGTH "O".

PROTECTED SIDE

FLOOD SIDE

LEGEND
PILE BATTER
3V ON 1H

C/L SHEET PILING (PZC-13)

SEPARATOR WALL

C/L DRAINAGE STRUCTURE

"M" SPACES @ "N"

"P" SPACES @ "H"

"G" SPACES @ "H"

"H"

2'-0"

2'-0"

64'-0"

"J"

"L"

"L"

TYPE 2A, 100 YR.
SCALE: 3/16" = 1'-0"

TYPE 2B, 100 YR.
SCALE: 3/16" = 1'-0"

PROTECTED SIDE

FLOOD SIDE

HP 14x73 (TYP.)

PZC-13 SHEET PILE CUT-OFF WALL

EL. "A"

EL. "B"

EL. "D"

5 SPACES @ 5'-0" = 25'-0"

4'-0"

20'-0"

6'-0"

5'-3"

2'-6"

PROTECTED SIDE

FLOOD SIDE

HP 14x73 (TYP.)

PZC-13 SHEET PILE CUT-OFF WALL

EL. "A"

EL. "E"

EL. "F"

EL. "G"

3 SPACES @ 5'-0" = 15'-0"

3'-0"

13'-4"

3'-8"

4'-0"

2'-6"

LOCATION	TYPE 2A NUMBER OF MONOLITHS	TYPE 2A LENGTH OF MONOLITHS	TYPE 2B NUMBER OF MONOLITHS	TYPE 2B LENGTH OF MONOLITHS	"A"	"B"	"C"	"D"	"E"	"F"	"G"
FALGOUT CANAL 56' SECTOR GATE	1	1 @ 55'	2	2 @ 55'	23.0	-9.0	-98.0	-62.0	-3.5	-110.5	-45.0
ELLIOT JONES 20' STOP LOG GATE	1	1 @ 40'	2	2 @ 45'	23.5	-5.0	-112.5	-46.0	-1.5	-128.0	-46.0
HUMPHRIES 20' STOP LOG GATE	1	1 @ 40'	2	2 @ 45'	23.5	-5.0	-112.5	-46.0	-1.5	-128.0	-46.0
MINORS CANAL 56' SECTOR GATE	N/A	N/A	2	2 @ 55'	23.0	N/A	N/A	N/A	-3.5	-128.0	-50.0
GIWW WEST 125' SECTOR GATE	4	4 @ 50'	2	2 @ 50'	23.0	-6.0,-10.0	-112.5	-54.0	-2.0	-128.0	-54.0
GIWW EAST 125' SECTOR GATE	4	2 @ 40', 2 @ 45'	2	1 @ 40', 1 @ 45'	25.0	-6.0,-10.0	-123.0	-57.0	-2.0	-136.0	-57.0
BAYOU BLACK 56' SECTOR GATE	2	2 @ 55'	N/A	N/A	22.0	-6.0	-112.5	-46.0	N/A	N/A	N/A
SHELL CANAL WEST 30' STOP LOG GATE	1	1 @ 40'	2	2 @ 45'	23.5	-6.0	-112.5	-46.0	N/A	N/A	N/A
SHELL CANAL EAST 56' SECTOR GATE	2	2 @ 55'	N/A	N/A	23.5	-6.0	-112.5	-46.0	N/A	N/A	N/A
MARMANDE CANAL 30' STOP LOG GATE	1	1 @ 40'	2	2 @ 40'	23.5	-5.0	-45.5	-46.0	-1.5	-128.0	-45.0

NOTES:
1.) ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4,000 PSI.
2.) ALL H-PILES SHEET PILES SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.
3.) THE TOP 5' OF ALL PILING SHALL BE PAINTED WITH SYSTEM NO 6, COAL TAR EPOXY.

MARK	DESCRIPTION	APPR.	DATE	MARK	DESCRIPTION	APPR.	DATE

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA TO THE GULF OF MEXICO LOUISIANA
REEVALUATION REPORT
TERREBONNE AND LAFOURCHE PARISHES LOUISIANA
100 YEAR

T-WALL TYPE 2C-2E

SHEET IDENTIFICATION
S-095A

TYPE 2C, 100 YR.
SCALE: 3/16" = 1'-0"

TYPE 2D, 100 YR.
SCALE: 3/16" = 1'-0"

TYPE 2E, 100 YR.
SCALE: 3/16" = 1'-0"

TYPE 2F, 35 YR.
SCALE: 3/16" = 1'-0"

LOCATION	TYPE 2C NUMBER OF MONOLITHS	TYPE 2C LENGTH OF MONOLITHS	TYPE 2D NUMBER OF MONOLITHS	TYPE 2D LENGTH OF MONOLITHS	TYPE 2E NUMBER OF MONOLITHS	TYPE 2E LENGTH OF MONOLITHS	"A"	"B"	"C"	"D"	"E"	"F"	"G"	"H"	"I"	"J"	"K"	"L"	"M"
FALGOUT CANAL 56' SECTOR GATE	2	2 @ 55'	6	4 @ 55', 2 @ 50'	6	1 @ 55', 5 @ 50'	23.0	2.0	-112.0	-45.0	8.0	-88.0	-39.0	13.0	-82.0	-34.0	N/A	N/A	N/A
ELLIOT JONES 20' STOP LOG GATE	6	4 @ 50', 2 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
HUMPHRIES 20' STOP LOG GATE	6	4 @ 50', 2 @ 45'	4	4 @ 50'	N/A	N/A	23.0	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
MINORS CANAL 56' SECTOR GATE	4	4 @ 50'	2	2 @ 50'	2	2 @ 50'	23.0	2.0	-130.0	-50.0	7.0	-101.0	-27.0	12.0	-94.5	-22.0	N/A	N/A	N/A
GIWW WEST 125' SECTOR GATE	2	2 @ 50'	2	2 @ 50'	2	2 @ 50'	23.0	2.0	-137.0	-57	7.5	-111.0	-35.0	12.5	-104.0	-25.0	N/A	N/A	N/A
GIWW EAST 125' SECTOR GATE	1	1 @ 45'	N/A	N/A	N/A	N/A	25.0	2.0	-130.0	-54.0	7.5	-101.0	-32.0	12.5	-94.5	-22.0	N/A	N/A	N/A
MARMANDE CANAL 30' STOP LOG GATE	4	4 @ 50'	6	6 @ 50'	6	6 @ 50'	23.0	2.0	-122.0	-32.0	8.0	-90.0	-26.0	13.0	-82.5	-21.0	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - REACH A	6	6 @ 45'	6	4 @ 50', 2 @ 45'	6	6 @ 50'	22.0	2.0	-122.0	-32.0	8.0	-90.0	-26.0	13.0	-82.5	-21.0	N/A	N/A	N/A
BAYOU BLACK 56' SECTOR GATE	6	6 @ 50'	4	4 @ 50'	N/A	N/A	22.0	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
SHELL CANAL WEST 30' STOP LOG GATE	6	2 @ 45', 4 @ 50'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
SHELL CANAL EAST 56' SECTOR GATE	6	6 @ 50'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
NAFTA T-WALL @ 36' SWING GATE	13	13 @ 50'	6	6 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 1	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 2	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 3	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 4	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 5	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 6	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ENVIRONMENTAL CONTROL STRUCTURE - BARRIER 7	8	8 @ 45'	4	4 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
BAYOU BLACK PUMPING STATION FRONTING PROTECTION	15	7 @ 50', 8 @ 45'	3	3 @ 50'	N/A	N/A	23.5	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
HANSON CANAL PUMPING STATION FRONTING PROTECTION	15	7 @ 50', 8 @ 45'	4	4@50'	N/A	N/A	23.0	3.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
C NORTH GULF SOUTH PIPELINE	3	1 @ 55', 2 @50'	4	4@50'	N/A	N/A	23.0	3.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
C NORTH ENBRIDGE PIPELINE	3	1 @ 55', 2 @50'	4	4@50'	N/A	N/A	23.0	3.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
C NORTH WILLIAMS DISCOVERY PIPELINE	3	1 @ 55', 2 @50'	4	4@50'	N/A	N/A	23.0	3.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ECS LOCKPORT TO LAROSE 1	4	4@50'	4	4@50'	N/A	N/A	18.0	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
ECS LOCKPORT TO LAROSE 1	4	4@50'	4	4@50'	N/A	N/A	18.0	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
UNION PACIFIC RAILROAD GATE	4	4@50'	4	4@50'	N/A	N/A	18.0	2.0	-130.0	-33.0	8.0	-101.0	-27.0	N/A	N/A	N/A	N/A	N/A	N/A
GIWW FLOODWALL	N/A	N/A	N/A	N/A	248	248@50'	17.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.0	-100.0	-33.0
LAROSE FLOODGATE	N/A	N/A	N/A	N/A	38	38 @ 50'	17.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.0	-100.0	-33.0

NOTES:
1.) ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4,000 PSI.
2.) ALL H-PILES SHALL MEET THE REQUIREMENTS OF ASTM A572, GRADE 50.
3.) THE TOP 5' OF ALL PILING SHALL BE PAINTED WITH SYSTEM NO 6, COAL TAR EPOXY.

FLOOD SIDE
PROTECTED SIDE
HP 14x73 (TYP.)
PZC-13 SHEET PILE CUT-OFF WALL
EL. "A"
EL. "C"
EL. "D"
EL. "K"
EL. "L"
EL. "M"

MORGANZA TO THE GULF OF MEXICO, LOUISIANA, POST-AUTHORIZATION CHANGE REPORT (PAC)

ECONOMIC APPENDIX OUTLINE

1

PART 1: BACKGROUND INFORMATION

INTRODUCTION

General. This appendix presents an economic evaluation of the two storm surge risk reduction alternatives being considered for the Morganza to the Gulf of Mexico, Louisiana evaluation area, which includes portions of two parishes in the state of Louisiana. It was prepared in accordance with Engineering Regulation (ER) 1105-2-100, Planning Guidance Notebook, and ER 1105-2-101, Planning Guidance, Risk Analysis for Flood Damage Reduction Studies. The National Economic Development Procedures Manual for Flood Risk Management and Coastal Storm Risk Management, prepared by the Water Resources Support Center, Institute for Water Resources, was also used as a reference, along with the Users Manual for the Hydrologic Engineering Center Flood Damage Analysis Model.

The economic appendix consists of a description of the methodology used to determine National Economic Development (NED) damages and benefits under existing and future conditions, projects costs, net benefits, and benefit-to-cost ratios. The evaluation reports benefits and costs at October 2011 price level. The proposed alternatives were evaluated by comparing estimated equivalent annual benefits that would accrue to the study area with estimated average annual project costs. Benefits were converted to equivalent annual values by use of the current FY 2012 Federal discount rate of 3.75 percent and a period of analysis of 50 years.   The year in which significant benefits will accrue as a result of project construction is 2026 for the 0.03 annual exceedance probability (AEP) storm surge risk reduction system alternative and 2035 for the 0.01 AEP alternative. The alternatives in the remainder of the appendix will be referred to as the 3% AEP alternative and the 1% AEP alternative.   The year 2035 was chosen as the base year for each of the alternatives as the basis for plan comparison. The alternatives that were screened to arrive at the selected alignment and alternatives are discussed in Section 4 and 5 of the Main Report.

In addition to the NED account, two other project accounts have been used to evaluate the project alternatives: Regional Economic Development (RED) and Other Social Effects (OSE). Each of these accounts will be discussed in separate appendices.

NED Benefit Categories Considered. The NED procedure manuals for coastal and urban areas recognize four primary categories of benefits for flood risk management measures: inundation reduction, intensification, location, and employment benefits. The majority of the benefits attributable to a project alternative generally result from the reduction of actual or potential damages caused by inundation. Inundation reduction, which is the only category of NED benefits addressed in this evaluation, includes the reduction of physical damages to structures, contents, and vehicles, avoidance of structure-raising costs, emergency cost reduction, agricultural benefits, water supply benefits, and safe harbor benefits.

Physical Flood Damage Reduction. Physical flood damage reduction benefits include the decrease in potential damages to residential and commercial structures, their contents, and the privately owned vehicles associated with these structures. Inundation reduction benefits were considered under both existing and future conditions. Projections of the future development expected to place in the study area during the period of analysis were included as part of the future condition analysis.

Since partial storm surge risk reduction will be provided before the base year of each project alternative, inundation reduction benefits for residential and commercial structures, their contents, and vehicles can be achieved during construction. The benefits during construction were computed by comparing the expected without-project damages to the with-project damages receiving partial risk reduction. The benefits during construction begin in the year 2024 for both of the alternatives.

Office of Management and Budget (OMB) survey forms were used to collect information on the value and placement of contents in the 24 industrial facilities located in the study area. The information from these surveys was used to develop the physical flood damage and benefits for these industrial properties. Additional information regarding the use of the OBM approved forms can be found in the final report dated May 2009 entitled *Morganza to the Gulf Post Authorization Change Report: Residential and Nonresidential Structure Inventory and Nonresidential Surveys.*

Avoidance of Structure-Raising Costs. Typically, property owners in areas that incur repetitive flooding have three options for reducing their flood risk: raise their structures in place, floodproof/retrofit their structures, or relocate to other areas. For purposes of this evaluation, only structure-raising measures were considered. The avoidance of structure-raising costs for all residential and non-residential structures that would otherwise incur repetitive flooding is considered a benefit attributable to the project alternative.

Emergency Cost Reduction Benefits. Emergency costs are those costs incurred by the community during and immediately following a major storm. They include the costs of emergency measures, such as evacuation and reoccupation activities conducted by local governments and homeowners, repair of streets, highways, and railroad tracks, and the subsequent cleanup and restoration of private, commercial, and public properties. In this evaluation, only the emergency cost reduction benefits associated with debris removal and cleanup and the reduction of damages to major and secondary highways and streets were considered.

Agricultural Benefits. NED agricultural benefits are defined as the increase in the value of the agricultural output of the area and the decrease in the cost of maintaining a given level of output attributable to a project alternative. These benefits include reductions in production costs and in associated costs, the reduction in damage costs from floods, erosion, sedimentation, inadequate drainage, or inadequate water supply, the value of increased

production of crops, and the economic efficiency of increasing the production of crops in the project area.

Agricultural benefits have not been quantified and are not included in this appendix. However, the average annual agricultural acres inundated under without-project and with-project conditions have been provided for each of the project alternatives.

Municipal Water Supply Benefits. The NED benefits from municipal water supply are defined as the willingness of a community to pay for an increase in the value of goods and services attributable to the water supply. In most cases, the marginal cost of supplying water is used to calculate the willingness of the consumers to pay for the additional water supply. However, because the marginal cost was not determined in this study, the water supply benefits were measured by comparing the reduction in the cost of treating water for municipal usage during periods of high salinity that is attributable to each of the project alternatives.

Safe Harbor Benefits for Large Recreational and Commercial Boat Fleets. The project alternatives reduce the risk of physical damage to large recreational and commercial boat fleet boats from the storm surges associated with minor storms, tropical storms, and hurricanes.   The reduction in damages to large vessels and the reduction in the cost of moving the vessels to safer areas are considered benefits attributable to the project alternatives. However, only the reduction in travel costs was considered in this evaluation.

Regional Economic Development. The RED account has been addressed in a separate appendix to evaluate the project alternatives. If the economic activity lost in the flooded region can be transferred to another area or region in the national economy, then these losses are not included in the NED account. However, the impacts on the employment, income, and output of the non-Federal or regional economy are considered part of the RED account. The input-output macroeconomic model RECONS was used to address the impacts of the construction spending associated with each of the project alternatives on the regional economy.

Other Social Effects. The OSE account has been addressed in a separate appendix to evaluate the project alternatives. OSE focuses on the health and safety impacts that each of the project alternatives has on the local population. Also, Environmental Justice (EJ) issues were investigated as part of the Environmental Impact Statement.

DESCRIPTION OF THE STUDY AREA

Geographic Location. The study area, which is located in coastal Louisiana approximately 60 miles southwest of the city of New Orleans, includes all of Terrebonne Parish and the portion of Lafourche Parish to the south and west of Bayou Lafourche. Communities located within the study area include the city of Houma, the towns of Chauvin, Dulac, and Montegut in southern Terrebonne Parish, the towns of Donner and Gibson in western Terrebonne Parish, and the towns of Gray and Schriever in northern Terrebonne Parish. Also included are the towns of Raceland, Lockport, and Pointe aux Chenes in Lafourche Parish and the portion of the city of Thibodaux south of Bayou Lafourche. The Gulf Intracoastal Waterway (GIWW) passes through the northern part of the study area in an east-west direction, and the Houma Navigation Channel (HNC) extends due south from Houma to the Gulf of Mexico. The southern extent of the study area is the alignment for the proposed hurricane protection structure that would cross the southern part of Terrebonne Parish in an east-west direction. The Morganza evaluation area was divided into 276 unique hydrologic reaches to enable an economic analysis of the project alternatives through the use of the HEC-FDA certified model. However, an inventory of residential and non-residential structures was only assembled in the 264 study area reaches that could be impacted by storm surges under the without-project condition.

Land Use. The total number of acres of developed, agricultural, and undeveloped land in Terrebonne Parish and the portion of Lafourche Parish included in the study area as of the year 2009 is shown in Table 1. The portions of Lafourche Parish north and east of Bayou Lafourche were not included in the analysis.

As shown in the table, approximately 10 percent of the total acres in the study area are currently developed.   Since there are approximately 76,000 acres of agricultural land and 2,100 acres of shrub land and grassland available for future development, there is sufficient land available to accommodate the projected residential and non-residential development through the year 2085 without impacting the wetlands in the area.

SOCIOECONOMIC SETTING

Population and Number of Households. Table 2 displays the population in each of the parishes for the years 1970, 1980, 1990, 2000, and 2010 (study year), as well as projections for the year 2035 and the year 2085, the two years that engineering inputs were modeled and used to calculate damages and benefits. Population projections are based on the Moody's County Forecast Database, which has population projections to the

year 2038. Moody's projections were extended by New Orleans District from the year 2038 to the year 2085 based on the growth rate forecasted by Moody's for the years 2018 through 2038. The slow, steady growth rate projected by Moody's during this 20-year period was consistent with the growth predicted by parish planning officials.

As shown in Table 2, both Lafourche and Terrebonne Parishes experienced a steady increase in population between 1970 and 2010. According to U.S. Census data, the population of Lafourche Parish increased from 89,974 in 2000 to 96,318 in 2010, a growth of 6,344 residents over the ten-year period. During the same period, the population of Terrebonne Parish increased from 104,503 to 111,860, an increase of 7,357 residents. The population in both parishes is projected to maintain this steady increase in population growth, with Lafourche Parish expected to have approximately 97,900 residents in 2035 and approximately 104,200 residents in the year 2085. Terrebonne Parish is expected to experience even more growth with an estimated population of approximately 120,900 in 2035 and 142,800 in 2085. Approximately 218,800 residents are projected to reside in the two-parish area in 2035, while approximately 247,000 residents are projected for the year 2085.

Table 3 displays the estimated population of the two parishes located within the inventoried portion of the study area for the year 2010 and the projected population for the years 2035 and 2085. The 2010 estimates are based on an inventory of residential and non-residential properties assembled in 2009 by field survey teams. The number of inventoried residential structures was then multiplied by 2.9, the average number of persons per household in the study area in 2010. In 2010, there were approximately 28,800 people residing in the inventoried structures in Lafourche Parish and approximately 104,900 people in Terrebonne Parish for a total of 133,700 residents. The projected population for the years 2035 and 2085 was based on the 2010 proportion of the total population residing within the inventoried area. The projected population for the years 2035 and 2085 for each parish was then multiplied by these proportions to determine the projected population for each parish. The population of Lafourche Parish is projected to total approximately 29,300 in 2035 and about 31,200 in 2085. In Terrebonne Parish, the population in this area is expected to total approximately 113,200 in 2035 and 133,800 in 2085.

Table 4 shows the total number of households in each parish for the years 1970, 1980, 1990, 2000, and 2010 and projections for the years 2035 and 2085. The projected number of households was based on the Moody's County Forecast Database and extended from the year 2038 to the year 2085 by New Orleans District based on the a growth rate forecasted by Moody's for the years 2018 through 2038.

The total number of households in Lafourche and Terrebonne Parishes experienced a steady increase between 1970 and 2010, which paralleled the growth in population. This increase, which was commensurate with the population growth experienced by the entire Gulf Coast region during the same period, can be attributed to increases in oil and gas exploration in the Gulf of Mexico and technological advancements in the industry. Similar

to the projected population growth in the two-parish area, the number of households is expected to continue increasing through the year 2085. Lafourche Parish is projected to have approximately 36,300 households in the year 2035, while Terrebonne Parish is projected to have about 43,400 households. By the year 2085, the number of households in Lafourche Parish is expected to reach approximately 38,100, while the number in Terrebonne Parish is expected to reach to approximately 50,400. In total, the two parishes are projected to have approximately 88,600 households in the year 2085.

Income. Table 5 shows the per capita personal income levels for each parish for the years 1990, 2000, 2005, 2008, and 2009, the year with the latest available data.

As shown in the table, both parishes experienced a steady increase in per capita income between 1990 and 2008. The growth in per capita income during this time reflects the increased oil and gas exploration and production activities in the Gulf of Mexico and the improvement in the economy of the state. It also reflects the improvement in the national economy that occurred from the late 1990s through the year 2008.

Between 2008 and 2009, however, both parishes experienced a slight decline in per capita income, which is likely a result of the global economic recession experienced during this time. The decline is slightly lower than the decline in per capita income seen in the state of Louisiana, which decreased from a per capita income of $38,142 in 2008 to $37,632 in 2009.

Employment.   Table 6 shows the total nonfarm employment by parish for the years 1970, 1980, 1990, 2000, 2010, and projections for the years 2035 and 2085. The employment projections were based on the Moody's County Forecast Database and extended from the year 2038 to the year 2085 by New Orleans District based on the growth rate forecasted by Moody's for the years 2018 through 2038.

Employment trends in the area have historically moved with the demand for oil and gas resources. The unemployment rate in Terrebonne and Lafourche parishes averaged approximately three percent prior to the end of 2008. The Houma-Thibodaux Metropolitan Statistical Area (MSA) continues to lead the state in jobs created and has one of the lowest unemployment rates in the state.

While the oil and gas industry pays the highest wages of all of the sectors of the economy, the services industry employs the largest number of residents. The retail sector is the second largest employer followed by government and other public agencies. The oil and gas sector in Terrebonne Parish employs slightly over 5,000 residents.

In addition to the oil and gas industry, there are three other sectors of the economy that are important to the region: commercial navigation, fisheries, and agriculture. The GIWW, the Houma Navigation Canal, and Bayou Lafourche provide key navigational channels for the energy sector. The coastal region provides a fertile spawning ground for

fisheries including shrimp, crabs, oysters, and finfish. Finally, the area grows and processes sugarcane.

Future Trends. In all portions of the study area, growth is highly dependent upon the major employment sectors. In addition, the growth in manufacturing is another major sector dependent upon the shipbuilding industry adjacent to Bayou Lafourche and the Houma Navigation Canal. The cyclical nature of the oil and gas industry has caused temporary fluctuations in the local economy since 1970. However, the overall level of growth in the population, income, and employment of the region has shown a steady increase. During the 1990s and early 2000s, technological advancements were made in the offshore oil exploration industry, such as 3D seismic drilling, which spurred exploration activity. Also during this decade, a regional cancer treatment facility was opened in the city of Houma.

The area was significantly impacted by the 2010 Deepwater Horizon British Petroleum oil spill and the decision by the Federal government to suspend the issuance of new deepwater drilling permits while safety standards were reassessed. Even though the first deep-water drilling permit since the oil spill was issued in March 2011, the area has not yet returned to the level of economic activity that it experienced prior to the oil spill and the resulting ban on drilling. According to data released by the Louisiana Department of Natural Resources at the end of 2011, there are currently 35 rigs in operation, the highest rig count off the Louisiana coast since the oil spill. The weekly rig count prior to the oil spill averaged 42 rigs. This appears to be a positive sign that the area is beginning to recover, albeit at a slow pace.

While the long term impact of the oil spill to the study area is unknown, there are other positive developments occurring in the area. During the past two decades, improvements were made in the transportation network including the opening of Interstate 310, which facilitates travel between the cities of Houma and New Orleans. The proposed I-49 highway will provide an efficient traffic route between the cities of New Orleans and Lafayette, although funding has not yet been obtained for its construction. This project may lead to increased development in the northern portion of the study area near the town of Gray in Terrebonne Parish. A proposed highway that will connect Louisiana Highway 3127 to the cities of Thibodaux and Houma could also facilitate growth in the study area.

Compliance with Policy Guidance Letter (PGL) 25 and Executive Order 11988.
Given the recent growth trends, it is reasonable to assume that development will continue to occur in the study area with or without the storm surge risk reduction system, and will not conflict with PGL 25 and EO 11988, which state that the primary objective of a flood risk reduction project is to protect existing development, rather than to make undeveloped land available for more valuable uses. With the project in place, future development may shift from the northern portions of the study area to the southern portion of the study area south of Houma. However, the overall growth rate is anticipated to be the same with or

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without the project in place. Thus, the project will not induce development, but would rather reduce the risk of the population being displaced after a major storm event.

RECENT FLOOD HISTORY

Tropical Flood Events. While Lafourche and Terrebonne parishes have periodically experienced localized flooding from excessive rainfall events, the primary cause of the flood events that have taken place in the two-parish study area has been the tidal surges from hurricanes and tropical storms. During the past 25 years, coastal Louisiana was impacted by eight major tropical events: Hurricane Juan (1985), Hurricane Andrew (1992), Tropical Storm Isidore and Hurricane Lili (2002), Hurricanes Katrina and Rita (2005), and Hurricanes Gustav and Ike (2008). While none of these storms tracked directly through the study area, the tidal surges associated with these storm events inundated structures and resulted in billions of dollars in damages to coastal Louisiana.

Table 7 provides a summary of the total FEMA flood claims paid to all Louisiana policyholders as a result of these tropical events. The table includes the number of paid losses, the total amount paid, and the average amount paid on each loss. The total and average paid losses have been converted to reflect 2011 price levels. The table only includes losses that were covered by flood insurance.

The following is a summary of each of the eight major tropical events and their effects on the two-parish area and coastal Louisiana.

Hurricane Juan.   Hurricane Juan caused extensive flooding throughout southern Louisiana due to its prolonged 5-day movement back and forth along the Louisiana coast. Rainfall totals in the area ranged from 5 inches to almost 17 inches. The storm was responsible for storm surges of 5 to 8 feet and tides of 3 to 6 above normal. According to FEMA officials, the estimated value of the residential and commercial damage and public assistance throughout coastal Louisiana totaled $112.5 million.

Over 800 homes were inundated in the coastal portion of Terrebonne Parish south of the city of Houma. Scattered pockets of flooding were also reported in the portions of Terrebonne and Lafourche Parishes north of Houma. Approximately 40 percent of the homes in the coastal areas of Lafourche Parish, including Pointe aux Chenes, were also inundated by the high tides.

Agricultural damages from the storm totaled $175 million, with 24 percent of these damages occurring in the two-parish study area. The soybean crop suffered over half of the agricultural damage, while the sugar cane crop incurred 20 percent of the damage.

Excessive rains and storm surge oversaturated the fields and caused a reduction in crop yields. The saturated fields also made it easier for the winds to topple over the cane stalks.

Hurricane Andrew. On August 26, 1992, Hurricane Andrew made landfall in St. Mary Parish, 80 miles west of Morgan City. FEMA reported that over 2,000 flood claims were filed as a result of the storm in Louisiana. These claims had a total value of over $25 million. Over 90 percent of this flood damage occurred in the Terrebonne Parish communities south of Houma, where up to six feet of water was reported. Only minor flooding in the back parts of subdivisions was reported in the city of Houma and in the areas north of the city. The unleveed portion of Lafourche Parish along its border with Terrebonne Parish, which includes the community of Pointe aux Chenes, also incurred extensive flood damage. However, most of the agricultural damage in the area occurred as the result of wind damage to the sugar cane crop.

Tropical Storm Isidore and Hurricane Lili. On October 3, 2002, one week after Tropical Storm Isidore affected the southeastern and south central coastal areas of Louisiana, Hurricane Lili made landfall on the western edge of Vermilion Bay south of the cities of Abbeville and New Iberia as a weak Category 2 hurricane. The high winds caused tidal flooding in the communities east of the eye of the storm. The ridge communities in Terrebonne Parish south of the city of Houma, including Cocodrie, Dulac, Isle de Jean Charles, and Montegut, and the community of Pointe aux Chenes in Lafourche Parish were affected by tidal flooding. The only community south of Houma that did not flood was Chauvin.

Insured flood losses from Tropical Storm Isidore and Hurricane Lili totaled nearly $600 million. Approximately $105 million of insured losses were related to Tropical Storm Isidore, while Hurricane Lili caused $471 million of insured losses. According to windshield surveys conducted by the American Red Cross, approximately 10,000 residential structures were damaged by winds and storm surges of the two storms. These surveys included both insured and uninsured structures. Tropical Storm Isidore caused damage to 2,905 structures, while Hurricane Lili caused damage to 7,356 structures.

In a revised report released in mid-November by the Louisiana State University Agricultural Center (LSU AgCenter), the estimated agricultural damages caused by Tropical Storm Isidore and Hurricane Lili totaled $454.3 million. This estimate also includes the agricultural damages caused by the continuation of rain during the month of October, which delayed the harvesting of crops. The excessive rains and storm surge flooded the agricultural fields and increased the harvest costs.

The wind and waves of Tropical Storm Isidore and Hurricane Lili caused extensive beach erosion in the barrier islands of Louisiana. These islands protect the Louisiana coastline from storm surges and provide a natural habitat for many species of wildlife. The barrier islands west of the mouth of the Mississippi River that were affected by the two storm events include the Isles Dernieres (Whiskey Bayou, Raccoon Island, Trinity Island, and East Island), Timbalier Island, East Timbalier Island, Elmer Island, and Grand Terre.

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Grand Isle incurred extensive damage along its eastern beach. Three small islands east of the mouth of the Mississippi River, Grand Gosier Island, Curlew Island, and Chandeleur Island, incurred extensive damage and beach erosion. A monetary value has not been determined for these environmental damages.

Hurricane Katrina. On August 29, 2005, Hurricane Katrina made landfall near the town of Buras in Plaquemines Parish about 50 miles east of coastal Lafourche and Terrebonne parishes. While it entered as a category 3 storm with winds in excess of 120 mile per hour. However, its storm surge of approximately 30 feet was more characteristic of a Category 5 hurricane. The majority of the damages from Hurricane Katrina occurred outside of the Morganza study area. However, if the hurricane had taken a more westerly track, the Houma area could have experienced the same magnitude of flooding as the city of New Orleans.

According to the Department of Health and Hospitals, approximately 1,400 deaths were reported following Hurricane Katrina. Approximately 1.3 million residents were displaced immediately following the storm, and 900,000 residents remained displaced as of October 5, 2005. According to the Louisiana Recovery Authority, two years after the storm, approximately 210,000 FEMA applicants still had out-of state mailing addresses, while 230,000 FEMA applicants had an in-state mailing address in a different zip code.

The storm caused more than $40.6 billion of insured losses to the homes, businesses, and vehicles in six states. Approximately two thirds of these losses, or $25.3 billion, occurred in Louisiana based on data obtained from the Insurance Information Institute. According to the LRA, approximately 150,000 housing units were damaged, and according to the Department of Environmental Quality, 350,000 vehicles, and 60,000 fishing and recreational vessels were damaged.

The storm surge from Hurricane Katrina inundated marshes and farmland throughout the coastal area including Terrebonne and Lafourche parishes. According to the LSU AgCenter, agricultural losses totaled approximately $825 million. The agricultural resources impacted by the storm include sugarcane, cotton, rice, soybeans, timber, pecans, citrus, and livestock. The losses to aquaculture (crawfish, alligators, and turtles), fisheries (shrimp, oysters, and menhaden), and wildlife and recreational resources totaled approximately $175 million.

Hurricane Rita. The hurricane made landfall along the Texas-Louisiana border on September 24, 2005, as a Category 3 storm with winds in excess of 120 miles per hour. As the hurricane passed south of the study area, its high winds pushed water north into coastal Lafourche and Terrebonne parishes. A storm surge of approximately 15 - 20 feet affected Coastal Louisiana from Terrebonne Parish to the Texas border. With estimated insured losses of approximately $3 billion, Hurricane Rita became one of the most costly natural disasters in U.S. history.

Approximately 2,000 square miles of farmland and marshes throughout the coastal area were inundated. According to the LSU AgCenter, agricultural losses totaled approximately $490 million. The agricultural resources impacted by the storm include sugarcane, cotton, rice, soybeans, timber, pecans, citrus, and livestock. The losses to aquaculture (crawfish, alligators, and turtles), fisheries (shrimp, oysters, and menhaden), and wildlife and recreational resources totaled approximately $100 million.

Hurricanes Gustav and Ike. On September 1, 2008, almost exactly three years after Hurricane Katrina, Hurricane Gustav made landfall near Cocodrie in Terrebonne Parish as a strong Category 2 hurricane. It followed a northwest path into central Louisiana, and most of the damages caused by the storm resulted from its high winds and heavy rain. Coastal flooding occurred in the low lying areas of Jefferson and Lafourche Parishes and the coastal areas of Terrebonne Parish south of the City of Houma.

Nearly 2 million residents of South Louisiana evacuated in the days before Gustav made landfall. Louisiana officials reported that emergency spending totaled approximately $500 million, which included $210 million for state agencies, $48 million for deploying the National Guard, $13.5 million for general evacuation shelters, $3 million for special-needs medical shelters, $6.1 million for transporting the medical needy, $21 million for costs of contraflow and evacuation from coastal communities and other areas, $20 million in special generators to open ice plants, pharmacies and service stations throughout the impacted areas, $5 million for state-purchased fuel, $19.7 million for ready-to-eat meals, $5.3 million for ice, and $2.5 million for water supplies. The State Department of Transportation estimated that it cost approximately $50 million to remove 1.5 million cubic yards of debris, and approximately $20 million to repair draw bridges.

Almost two weeks later, on September 12 and 13, the Louisiana coastal region incurred additional flood damages as Hurricane Ike moved along the Louisiana coast. According to estimates from the state officials, approximately 12,000 homes and businesses were flooded by the two storms. Approximately 2,500 buildings in Terrebonne Parish south of the City of Houma incurred flood damages from Hurricane Ike.

The LSU AgCenter estimated that potential lost revenues and damages to the infrastructure of the agriculture, forestry, and fisheries industries in Louisiana resulting from the two hurricanes totaled approximately $959 million. The storm surge primarily affected the cattle, rice, soybeans, and sugarcane.

FEMA Flood Claims. While Lafourche and Terrebonne parishes have periodically experienced localized flooding from excessive rainfall events, the primary cause of the flood events that have taken place in the two-parish study area has been the tidal surges from hurricanes and tropical storms. The total FEMA flood claims for the two parishes in the Morganza to the Gulf evaluation area that were paid between 1978 and September 2011 are summarized in Table 8. The table includes only those claims that were covered by flood insurance. Figure 1 shows the location of the repetitive loss properties that have had two or more FEMA flood claims during any 10-year period between 1978 and 2010.

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SCOPE OF THE STUDY

Problem Description. The study area is characterized by low, flat terrain with ridges surrounding the waterways. The terrain has made the area highly susceptible to flooding from the tidal surges of hurricanes and tropical storms. The apparent subsidence, or relative sea level rise, that has been taking place in the Morganza study area, is expected to magnify the flooding problems in the future. While the Terrebonne Levee and Conservation District is currently maintaining a system of forced drainage levees, pump stations, and flood control structures for Terrebonne Parish, an adequate overall storm surge risk reduction system is not currently available for the entire study area.

Project Alternatives. As part of the 2002 Morganza to the Gulf, Louisiana Feasibility Report, a project alignment was selected and later authorized to provide storm surge risk reduction for portions of Terrebonne and Lafourche parishes. The authorized alignment was designed to contain the pre-Katrina surge elevations associated with the 1% (100-year) annual exceedance probability (AEP) storm surge risk reduction system, and the costs were provided in 2002 price levels. Since that time, the hydrology, project design criteria, and implementation costs have changed. A Revised Project Cost Estimate (RPCE) report was developed in 2008 using post-Katrina design criteria and water surface profiles for the 1% (100-year) AEP storm surge risk reduction system. A second alternative under consideration, the 3% (approximately 35-year) AEP storm surge risk reduction system, applies pre-Katrina design criteria and authorized levee height elevations to the authorized alignment. This alignment involves the construction of new earthen levees that would run parallel to Louisiana Highway 57 south of Lake Boudreaux and north of the Falgout Canal and would connect to existing forced drainage levees. The levees will be used in conjunction with flood risk management and environmental structures and would minimize the adverse impacts to the environment, local interests, navigation, and industry. Finally, construction of a lock structure on the Houma Navigation Canal (HNC) south of Bayou Grand Caillou has been included as part of the system. Figure 2 shows the location of the study area reaches and the project alignment. The study area reaches are also shown in the 11x17 maps attached to the main report.

PART 2: ECONOMIC AND ENGINEERING INPUTS TO THE HEC-FDA MODEL

HEC-FDA MODEL

Model Overview. The Hydrologic Engineering Center Flood Damage Analysis (HEC-FDA) Version 1.2.5a Corps-certified model was used to calculate the damages and benefits for the Morganza evaluation. The economic and engineering inputs necessary for the model to calculate damages for existing conditions (2010), the first year of partial storm surge risk reduction (2024), the project base year (2035), and the final year in the period of analysis (2085) are described in this section of the report. The economic inputs include structure inventory, future development, contents-to-structure value ratios, vehicles, first floor elevations, and depth-damage relationships. The engineering inputs include ground elevations, exterior and interior relationships, local levee performance, and Federal levee performance. A separate HEC-FDA model was executed for the industrial structures in the study area for the years 2024, 2035, and 2085.

The uncertainty surrounding each of the economic and engineering variables was also entered into the model. Either a normal probability distribution, with a mean value and a standard deviation, or a triangular probability distribution, with a most likely, a maximum and a minimum value, was entered into the model to quantify the uncertainty associated with the key economic variables. A normal probability distribution was entered into the model to quantify the uncertainty surrounding the ground elevations. The number of years that stages were recorded at a given gage was entered for each study area reach to quantify the hydrologic uncertainty or error surrounding the stage-probability relationships. The uncertainty associated with the levee performance was quantified using the levee features section of the model, which related the elevation of exterior storm surges to the probability of levee failure.

ECONOMIC INPUTS TO THE HEC-FDA MODEL

Structure Inventory. Field surveys were conducted in 2009 to develop a residential and non-residential structure inventory for the economic analysis. The areas to be inventoried had been selected in 2008 based on estimates of surge elevations for this area developed as part of the Louisiana Coastal Protection and Restoration (LACPR) evaluation. Since the ground elevations in the northern portions of the evaluation area near Bayou Lafourche, including the towns of Gray and Schriever in Terrebonne Parish and the

southern portion of the city of Thibodaux in Lafourche Parish, and in the western portions of the study area near Donner and Gibson in Terrebonne Parish, were determined to be above these storm surge estimates, the structures in these areas were not included in the inventory.

Based on the structural information collected during the field surveys, the Marshall and Swift Valuation Service was used to calculate a depreciated replacement cost for all residential and non-residential structures in the study area reaches. The inventoried structures were classified as one of 14 structure types: residential one-story with slab or pier foundation, residential two-story with slab or pier foundation, mobile home, eating and recreation, grocery and gas station, multi-family residence, professional building, public and semi-public building, repairs and home use establishment, retail and personal services building, and warehouse, and contractor services building. The inventory also included 24 industrial structures that were inventoried using OMB approved interview forms. Table 9 shows the number of structures by structure category and the total number of vehicles associated with the residential structures for existing conditions (2010) for each study area reach or HEC-FDA model station number. The value of the land was not included in the analysis.

Future Development Inventory. Projections were made of the future residential and non-residential development to take place in the Morganza study area under without-project conditions. Based on historical economic trends, a total of 7,320 residential and 1,319 non-residential structures were placed on the undeveloped land within the study area reaches as part of the structure inventory for the year 2035. An additional 16,332 residential and 4,661 non-residential structures were added to the inventory for the year 2010 to obtain the structure inventory for the year 2085.

The development projected to occur in each study area reach between the year 2010 and the year 2035 was placed at an elevation equal to the stage associated with the without-project one percent annual chance exceedance (1% ACE) (100-year) event, unless the ground elevation was higher. The projected development occurring after the year 2035 was placed at an elevation equal to the stage associated with the without-project 1% ACE (100-year) event for the year 2085, unless the ground elevation was higher. The values for the projected residential and non-residential structures were assigned using the average value calculated for each structure category based on the 2010 existing development.

Table 10 shows the number of structures in each structure category and the average depreciated replacement values for (2010) existing conditions. Table 11 shows the projected number of structures in each structure category for the future years 2035 and 2085, respectively. The value of the land was not included in the analysis.

Residential and Non-Residential Content-to-Structure Value Ratios. On-site interviews were conducted with the owners of a sample of ten structures from each of the

three residential content categories (30 residential structures) and each of the eight non-residential content categories (80 non-residential structures). A CSVR was computed for each residential and non-residential structure in the sample based on the total depreciated content value developed from the surveys. An average CSVR for each of the five residential structure categories and nine commercial structure classifications was calculated as the average of the individual structure CSVRs.

Since only a limited number of field surveys were conducted for each of the residential and non-residential content categories, statistical bootstrapping was performed to address the potential error in estimating the mean and standard deviation CSVR values. Statistical bootstrapping is a method that uses re-sampling with replacement to improve the estimate of a population statistic when the sample size is insufficient for straightforward statistical inference. The bootstrapping method has the effect of increasing the sample size. Thus, bootstrapping provides a way to account for the distortions caused by the specific sample that may not be fully representative of the population.

With use of the @Risk software, a simulation using 100,000 iterations was executed for each content category. Within each iteration, a new ten-observation sample with replacement, called a bootstrap sample, was taken from the original sample of ten observations. Each observation within the original sample was given a uniform probability or chance of being selected as each one of the ten values within the bootstrap sample. The @Risk spreadsheet calculated a mean value and a standard deviation for each of the bootstrap samples, and then calculated a mean value for all of the bootstrap means and mean value of all the standard deviations.

Table 12 shows the CSVRs and standard deviations for each of the residential and non-residential structure categories derived using the statistical bootstrapping technique. The CSVRs and standard deviations were used in the HEC-FDA model along with the depth-damage relationships to calculate flood damages for residential and non-residential structures. A unique CSVR was developed for each of the 24 industrial structures in the study area based on the content values provided by the owners of the properties using OMB approved interview forms.

Vehicle Inventory. Based on 2000 Census block group data for the evaluation area, it was determined that there are an average of 1.64 vehicles associated with each household (owner occupied housing or rental unit). According to the Southeast Louisiana Evacuation Behavioral Report published in 2006 following Hurricanes Katrina and Rita, approximately 70 percent of privately owned vehicles are used for evacuation during storm events. The remaining 30 percent of the privately owned vehicles remain parked at the residences and are subject to flood damages. Using the Manheim Used Vehicle Value Index, which is based on over 4 million annual automobile transactions adjusted to reflect retail replacement value, each vehicle was assigned an average value of $12,879. Since only those vehicles not used for evacuation can be included in the damage calculations, an adjusted average vehicle value of $6,336 ($12,879 x 1.64 x .30) was assigned to each individual residential structure record in the HEC-FDA model. The

adjusted vehicle value was adjusted upward by 3.7 percent using the Manheim index from 2010 to 2011 to reflect an October 2011 price level. If an individual structure had more than one housing unit, then the adjusted vehicle value was assigned to each housing unit in a residential or multi-family structure category.

First Floor Elevations and Elevation of Vehicles. Topographical data obtained from the Light Detection and Ranging (LIDAR) digital elevation model (DEM) using the NAVD88 (2004.65 epoch) were used to determine ground elevations. Field survey teams estimated the height of each residential and non-residential structure above the ground using hand levels. The ground elevation was added to the height of the foundation of the structure above the ground in order to determine the first floor elevation of the structure. Vehicles were assigned to the ground elevation of the adjacent residential structures.

Depth-Damage Relationships. Site-specific saltwater, long duration (approximately one week) depth-damage relationships, developed by a panel of building and construction experts for the Morganza evaluation, were used in the economic analysis. These curves indicate the percentage of the total structure value that would be damaged at various depths of flooding. Damage percentages were determined for each one-half foot increment from one-half foot below first floor elevation to two feet above first floor, and for each one-foot increment from 2 feet to 15 feet above first floor elevation. The panel of experts developed depth-damage relationships for five residential structure categories and for three commercial structure categories. Depth-damage relationships were also developed for three residential content categories and eight commercial content categories. A unique depth-damage relationship was developed for the contents of each of the 24 industrial structures in the study area based on information provided by the owners of the properties using OMB approved interview forms.

The depth-damage relationships for vehicles were developed based on interviews with the owners of automobile dealerships that had experienced flood damages and were used to calculate flood damages to vehicles at the various levels of flooding.

Table 13 shows the residential and non-residential depth-damage relationships developed for structures, contents, and vehicles. More specific data regarding the depth-damage relationships can be found in the final report dated May 1997 entitled *Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios (CSVRs) in Support of the Lower Atchafalaya and Morganza to the Gulf, Louisiana, Feasibility Study*.

Uncertainty Surrounding the Economic Inputs. The uncertainty surrounding the four key economic variables was quantified and entered into the HEC-FDA model. These economic variables included structure values, contents-to-structure value ratios, first floor elevations, and depth-damage relationships. The HEC-FDA model used the uncertainty

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surrounding these variables to estimate the uncertainty surrounding the stage-damage relationships developed for each study area reach.

Structure and Vehicle Values. In order to quantify the uncertainty surrounding the values calculated for the residential and non-residential structure inventory, several survey teams valued an identical set of structures from various evaluation areas in the Gulf Coast region. The structure values calculated by each of the teams during windshield surveys were used to develop a mean value and a standard deviation for each structure in the sample.   The standard deviation was then expressed as a percentage of the mean value for that structure. The average standard deviation as a percentage of the mean for the sampled structures was then used to represent the uncertainty surrounding the structure value for all the inventoried residential and non-residential structures. The average standard deviation, which was expressed as a percentage of the mean structure value, totaled 12.15 percent for residential structures and 14.28 percent for non-residential structures.

The uncertainty surrounding the values assigned to the vehicles in the inventory was determined using a triangular probability distribution function. The Manheim vehicle value, adjusted for number of vehicles per household and for the evacuation of vehicles prior to a storm event, was used as the most likely value. The average value of a new vehicle before taxes, license, and shipping charges was used as the maximum value, while the average 10-year depreciation value of a vehicle was used as the minimum value.

Content-to-Structure Value Ratios. On-site interviews were conducted with the owners of a sample of ten structures from each of the three residential content categories (30 residential structures) and each of the eight non-residential content categories (80 non-residential structures). A CSVR was computed for each residential and non-residential structure in the sample based on the total depreciated content value developed from these interviews. A probability distribution function derived using the statistical bootstrapping method was then used to describe the distribution of these observations around the expected mean value. The mean and standard deviation values for each residential and non-residential category were entered into the HEC-FDA model. The model used a normal probability density function to describe the uncertainty surrounding the CSVR for each content category. The expected values and standard deviations are shown for each of the three residential categories and the eight non-residential categories in the final report dated May 1997 entitled *Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios (CSVRs) in Support of the Lower Atchafalaya and Morganza to the Gulf, Louisiana Feasibility Study*. Since the CSVRs for the 24 industrial structures in the study area were based on information provided by the property owners, there was no uncertainty surrounding these ratios.

First Floor Elevations. The topographical data used to estimate the first floor elevations assigned to the structure inventory contain two sources of uncertainty. The first source of uncertainty arises from the use of the 2009 LIDAR data, and the second source of uncertainty arises from the use of hand levels to determine the structure foundation

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heights above ground elevation. The error implicit in using LIDAR data to estimate the ground elevation of each of the inventoried structures is normally distributed with a mean of zero and a standard deviation of 0.219 feet. These statistics were calculated based on comparing 2,241 engineering survey points or spot elevations to the elevations determined using the 2009 LIDAR data throughout the evaluation area. According to the Hydrologic Engineering Center training manual, the uncertainty implicit in estimating foundation heights using hand levels from within 50 feet of the structure is normally distributed with a mean of zero and a standard deviation of 0.3 feet at the 95 percent level of confidence.

Based on the error surrounding the LIDAR data and the error arising from the use hand levels, the total uncertainty was estimated for each structure category at the 90 percent level of confidence. The two standard deviations (LIDAR and hand levels) were squared and then totaled. The square root of this total, 0.297 feet, represents the uncertainty surrounding the first floor elevations assigned to the structures located in the Morganza evaluation area.

Depth-Damage Relationships. A triangular probability density function was used to determine the uncertainty surrounding the damage percentage associated with each depth of flooding. A minimum, maximum and most likely damage estimate was provided by a panel of experts for each depth of flooding. The specific range of values regarding probability distributions for the depth-damage curves can be found in the final report dated May 1997 entitled *Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios (CSVRs) in Support of the Lower Atchafalaya and Morganza to the Gulf, Louisiana, Feasibility Study*.

The owners of the 24 industrial properties provided a minimum, maximum, and most likely content damage estimate for each depth of flooding using OBM approved survey forms. Copies of the OBM survey forms used to develop the depth-damage relationships can be found in the final report dated May 2009 entitled *Morganza to the Gulf Post Authorization Change Report: Residential and Nonresidential Structure Inventory and Nonresidential Surveys*.

ENGINEERING INPUTS TO THE HEC-FDA MODEL

Ground Elevations. Geospatial Engineering acquired elevation data for the Morganza study area in 2009. The LIDAR data were processed and used to create a digital elevation model (DEM) with a five-foot by five-foot horizontal grid resolution. The DEM used NAVD88 2004.65 vertical datum to determine the ground elevations for each of the residential and non-residential structures in the evaluation area.

Stage-Probability Relationships. Stage-probability relationships were provided for the existing (2010) without-project condition, and for the first year of partial storm surge reduction (2024), the base year of the project (2035), and the final year in the period of analysis (2085) under both without-project and with-project conditions for each of the 276 study area. Water surface profiles were provided for eight annual chance exceedance (ACE) events: 99% (1-year), 20% (5-year), 10% (10-year), 4% (25-year), 2% (50-year), 1% (100-year), 0.5% (200-year), and 0.2% (500-year).   The water surface profiles were based only on storm surge and did not incorporate heavy rainfall events.

The 99% ACE (1-year) event, 20% ACE (5-year) event, and 10% ACE (10-year) event water surface profiles for the year 2010 were based on gage data. For each of these ACE events, the water surface profiles for the years 2024, 2035, and 2085 were determined by adding relative sea level rise to the gage data. The water surface profiles for the 2% ACE (50-year) event through the 0.2% ACE (500-year) event were based on results from the ADCIRC model. The 4% ACE (25-year) event stages were determined by interpolation between the 10% ACE (10-year) event stages and the 2% ACE (50-year) event stages.

In cases where an analysis of the hydrology indicated that the surge elevation for a particular probability storm event would not impact a study area reach, the stages for that event were assigned using an elevation lower than the minimum ground elevation in that study area reach. This was done to ensure that flood damages due to storm surge would not be reported for these areas. The minimum ground elevations were referenced to the LIDAR data for the Morganza evaluation.

Non-Federal and Federal Levee Performance. Local levee systems provide flood risk reduction under existing conditions (2010) for over 29,000 residential and non-residential structures located within 78 of the study area reaches. A set of fragility curves, which relates specific stages in NAVD 88 (2004.65 epoch) on the exterior side of the levee to four probabilities of levee failure (zero percent, ten percent, forty-five percent, and ninety-five percent), were developed for each of the local levee systems under the without-project condition. It was assumed that there was a zero percent probability of failure at the 2-foot stage for all local levees.

The fragility curves developed for each of the local levee systems considered multiple failure modes including the slope of the levee, seepage, wave heights, overtopping, and erodability. The failure of an existing non-Federal levee typically occurs when the structural integrity of the levee is compromised by the storm surge. However, geotechnical failure analyses conducted in the evaluation area determined that there is only a one to three percent probability of failure at the top of the levee due to stability issues. Thus, overtopping and erodability were used to develop the non-Federal levee fragility curves.

The fragility curves for the non-Federal levee system were entered into the HEC-FDA model for each study area reach containing a non-Federal levee in order to assess the performance of the non-Federal levee system. Table 14 shows the non-Federal levee fragility curves and the top of levee elevation developed for each of the study area reaches containing a levee.

Federal levees will provide flood risk reduction under future conditions for approximately 52,000 residential and non-residential structures located within 233 of the study area reaches. Each of the study area reaches was assigned to one of the ten major Federal levee reaches (A, B, and E through L) based on the location of the reach and the path of the storm surge should the Federal levee fail. Fragility curves were not developed for the Federal levee system. Only a top of the Federal levee elevation was entered into the HEC-FDA model for each of the study area reaches. The top of the levee elevation in this analysis does not represent the actual height of the Federal levee; rather, it represents the still water stage elevation at which the levee is assumed to fail. At this stage, which is below the actual top of the levee, waves will overtop the Federal levee at a rate of 2.0 cubic feet per second (cfs). Table 15 shows the top of Federal levee still water stage or elevation for each of the major levee reaches for each of the project alternatives.

When existing non-Federal or Federal levees are included in the analysis, an exterior-interior stage relationship must be considered in the analysis. The exterior-interior stage relationship defines the relationship between the water surface, or stage, outside of the levee and the stage within the floodplain behind the levee. Under the with-project conditions, exterior and interior stage relationships were provided for each study area reach. In the event of a Federal levee failure, the interior surge elevation changes as the distance from the levee increases. Thus, a unique interior surge elevation curve was provided for each interior study area reach under with-project conditions. Under the without-project condition, an exterior-interior stage relationship was not provided for each study area reach. In the event of a non-Federal levee failure, the elevation of the surges within the reach is the same on both sides of the levee regardless of the distance from the levee.

Figures 2-1 and 2-2 in the main report provide a conceptual depiction of how the engineering inputs are used in the HEC-FDA model.

Uncertainty Surrounding the Engineering Inputs. The uncertainty surrounding three key engineering parameters was quantified and entered into the HEC-FDA model. These engineering variables included ground elevations, stage-probability curves, and performance of the non-Federal and Federal levees. The HEC-FDA model used the uncertainty surrounding these variables to estimate the uncertainty surrounding the elevation of the storm surges for each study area reach.

Ground Elevations. An engineering survey was conducted to estimate the uncertainty surrounding the use of the 2009 LIDAR data to estimate ground elevations in urbanized areas. The LIDAR data were compared to 2,241 spot elevations, or engineering survey points, throughout the urbanized portions of the evaluation area. The uncertainty surrounding these data was found to be normally distributed with a mean of zero and a standard deviation of 0.219 feet. (A combination of the uncertainty surrounding the ground elevations and the foundation height of a residential and non-residential structure was discussed in the first floor elevation uncertainty section of this report.)

Stage-Probability Relationships. A 50-year equivalent record length was used to quantify the uncertainty surrounding the stage-probability relationships for each study area reach. Based on this equivalent record length, the HEC-FDA model calculated the confidence limits surrounding the stage-probability functions.

Levee Performance.   The uncertainty surrounding the performance of the non-Federal levees was based on the fragility curves entered for each study area reach. The Federal levees are assumed to fail with certainty once the surge stage reaches the top of the levee height assigned to each study area reach.

PART 3: NATIONAL ECONOMIC DEVELOPMENT (NED) FLOOD DAMAGE AND BENEFIT CALCULATIONS

NED FLOOD DAMAGE AND BENEFIT CALCULATIONS FOR STRUCTURES, CONTENTS, AND VEHICLES

HEC-FDA Model Calculations. The HEC-FDA model was utilized to evaluate flood damages using risk-based analysis. Damages were reported at the index location for each of the 264 study area reaches for which a structure inventory had been conducted. A range of possible values, with a maximum and a minimum value for each economic variable (first floor elevation, structure and content values, and depth-damage relationships), was entered into the HEC-FDA model to calculate the uncertainty or error surrounding the elevation-damage, or stage-damage, relationships. The model also used the number of years that stages were recorded at a given gage to determine the hydrologic uncertainty surrounding the stage-probability relationships. Fragility curves for the non-Federal levees and top of levee elevations and exterior/interior stage relationships for Federal levees were entered into the levee features section of the model.

The possible occurrences of each variable were derived through the use of Monte Carlo simulation, which used randomly selected numbers to simulate the values of the selected variables from within the established ranges and distributions. For each variable, a sampling technique was used to select from within the range of possible values. With each sample, or iteration, a different value was selected. The number of iterations performed affects the simulation execution time and the quality and accuracy of the results. This process was conducted simultaneously for each economic and hydrologic

variable. The resulting mean value and probability distributions formed a comprehensive picture of all possible outcomes.

Stage-Damage Relationships with Uncertainty. The HEC-FDA model used the economic inputs to generate a stage-damage relationship for each structure category in each study area reach under existing (2010) and future (2024, 2035, and 2085) conditions. The possible occurrences of each economic variable were derived through the use of Monte Carlo simulation. A total of 1,000 iterations were executed by the model for the Morganza evaluation. The sum of all sampled values was divided by the number of samples to yield the expected value for a specific simulation. A mean and standard deviation was automatically calculated for the damages at each stage.

Stage-Probability Relationships with Uncertainty. The HEC-FDA model used an equivalent record length (50 years) for each study area reach to generate a stage-probability relationship with uncertainty for the without-project and the with-project alternatives under existing (2010) and future (2024, 2035, and 2085) conditions through the use of graphical analysis. The model used the eight stage-probability events together with the equivalent record length to define the full range of the stage-probability or stage-probability functions by interpolating between the data points. Confidence bands surrounding the stages for each of the probability events were also provided.

Without-Project Expected Annual Damages. The model used Monte Carlo simulation to sample from the stage-probability curve with uncertainty. For each of the iterations within the simulation, stages were simultaneously selected for the entire range of probability events. For the study area reaches without a non-Federal levee system, the Monte Carlo simulation then selects a corresponding damage value for each of the stages from the stage-damage relationships with uncertainty. For the study area reaches with a non-Federal levee system, the Monte Carlo simulation also selects a failure probability from the fragility curve developed for the non-Federal levee. If the selected stages from the stage-probability curve are below the height of the non-Federal levee, then the fragility curve is used to determine if there is levee failure. If the levee fails, then a damage estimate is sampled from the stage-damage relationship. However, if the levee does not fail, then zero damages will be selected for that iteration. If the selected stages are equal to or above the height of the non-Federal levee and the levee fails, then the Monte Carlo simulation will select a damage value from the stage-damage relationship with uncertainty for that iteration. In general, the top of the non-Federal levee elevations were set at an elevation between the stages associated with the 10% ACE (10-year) event and the 4% ACE (25-year) event. There are no exterior-interior stage probability relationships under the without-project conditions.

The sum of all damage values divided by the number of iterations run by the model yielded the expected value, or mean damage value, with confidence bands for each probability event. The probability-damage relationships are integrated by weighting the

damages corresponding to each magnitude of flooding (stage) by the percentage chance of exceedance (probability). From these weighted damages, the model determined the expected annual damages (EAD) with confidence bands (uncertainty). For the without-project alternative, the expected annual damages (EAD) were totaled for each study area reach to obtain the total without-project EAD under existing (2010) and future (2024, 2035, and 2085) conditions.

Adjusted Without-Project Expected Annual Damages. The without-project expected annual damages calculated as part of the economic analysis do not consider the behavior of property owners whose structures have incurred repetitive flood losses. The HEC-FDA model implicitly assumes that all damaged assets will be restored to their prior market value completely and instantaneously after each storm event. However, property owners could also opt to have their structures raised in place, floodproof and/or retrofit their structures, relocate within the floodplain, or permanently evacuate from the study area. The course of action selected by an individual property owner following repetitive flood losses depends upon many factors, including the degree of aversion to future anticipated flood risk by that property owner.

As shown in Table 16, unadjusted without-project expected annual damages increase approximately 184 percent between 2010 and 2085. Approximately 6 percent of this percentage increase is attributable to future development, while the remaining 178 percent is attributable to the projected rise in relative sea level. A breakdown of expected annual damages revealed that there were a significant number of structures with damage exposure from relatively frequent events. Table 17 shows that approximately 7,500 residential and non-residential structures incur flood damages from a 10% ACE (10-year) storm event in the year 2035, and Table 18 shows that approximately 1,700 residential structures would incur damages greater than or equal to 50 percent of the structural value at the 10% ACE (10-year) event. Given the number of structures at risk from frequent flooding, the magnitude of these damages, and the increased frequency which residential and non-residential structures would be exposed to flooding, adjustments to the implicit assumptions of the HEC-FDA model were deemed necessary.

Historical Response to Flood Events. The Morganza study area experienced numerous flood events during the past several decades. Historical data show that the post-flood response of property owners to the flood events prior to 2005 did not result in significant outmigration from the study area. Data from the 2000 Census show that approximately 65 percent of residents in the Lafourche and Terrebonne Parishes lived in the same housing unit as they had in 1995. This percentage ranged from a high of 81 percent in Dulac (southern portion of the study area) to a low of 54 percent in Thibodaux (northern portion of the study area). In comparison, the national percentage of the population residing in the same house in 2000 as in 1995 was 54 percent.

According to local officials, residents in low-lying communities began relocating to areas in the northern parts of the study area after Hurricanes Katrina and Rita impacted the area in 2005. Reasons for this intra-parish shift were a combination of weariness on the part of residents of having to deal with repeat flooding and the more stringent requirements to

obtain permits for rebuilding after homes were damaged. In order to rebuild, residents had to incur the cost of building to higher elevations. The ability to secure insurance at a reasonable price was also cited as a reason for the exodus.

The rate of retreat from the southern communities slowed around 2008 after Hurricane Ike impacted the area due to federal assistance, as well as the construction of local levees, which reduced damages to the area. In addition, the two parishes have also implemented elevation programs designed to raise the structures in flood-prone areas. The elevation costs have been offset by state and Federal funding and, in the case of properties with flood insurance, supplemental support in the form of FEMA Increased Cost of Compliance Grants. These programs have made structure elevation more affordable for residents.

Local officials also stated that residents prefer to remain due to the culture of the residents and the economy of the area. The economy of Terrebonne Parish is closely tied to its abundant natural resources, and many of the residents in the small communities outside of Houma are shrimpers, oystermen, crabbers, fishermen, and trappers. In Lafourche Parish, the economy is strongly tied to the production and distribution of natural gas and oil, commercial fishing, and sugar cane.

Historical data show that recent flood events have not resulted in significant outmigration from the study area, and the post-flood response of property owners in the past has been consistent with the HEC-FDA assumption that the structure inventory will remain in place throughout the period of analysis. Although the HEC-FDA certified model is a probability-based, and not an event-driven, model, the assumption that structures will be completely and immediately repaired, is rarely the case for major flood events. While it may require considerable time (months to years) to fully complete repairs, past population trends nevertheless indicate that residents and the structures in which they live have not been permanently removed from the study area. However, the manner in which property owners have responded in the past may or may not be representative of how they will respond in the future to more repetitive and more severe flood events. The more frequent and damaging that flood events become due to sea level rise, the less time property owners have to repair damaged structures prior to the next flood. Thus, adjustments were made to the 2024, 2035, and 2085 structure inventories to account for the projected rise in relative sea level.

Structure Inventory Adjustments. The adjustments were made to the structure inventory before executing the HEC-FDA model to more accurately reflect the most likely future without-project and with-project conditions. For the 2024 residential structure inventory, all properties with a first floor elevation less than or equal to the 2010 10% ACE (10-year) water surface elevation exterior to the non-Federal levee, if it exists, within each study area reach were raised to the 2010 1% ACE (100-year) plus 2 feet to account for the sea level rise projected to occur during the period of analysis. This would also ensure that the structures would not be raised more than once during the period of analysis. For

the 2035 residential structure inventory, all properties with a first floor elevation less than or equal to the 2024 10% ACE (10-year) water surface elevation exterior to the non-Federal levee were raised to the 2010 1% ACE (100-year) plus 2 feet. For the 2085 residential structure inventory, all properties with a first floor elevation less than or equal to the 2035 10% ACE (10-year) water surface elevation exterior to the non-Federal levee were raised to the 2010 1% ACE (100-year) plus 2 feet.

The non-residential structure inventory was also adjusted for repetitive flooding based on the 10% ACE (10-year) water surface elevation exterior to the non-Federal levee. If the total value of the structures in a non-residential structure category (except warehouses) was greater than or equal to 15 percent of the total value within a study area reach, then all of the structures in that category were raised based on the same criteria used for the residential structure inventory. If the total value was less than 15 percent, then the structures in that non-residential structure category were not adjusted for repetitive flooding due to their limited exposure. Warehouses were assumed to remain at their initial first floor elevation throughout the period of analysis. These structures would be difficult to elevate given the size and nature of their operations. Floodproofing measures were also not considered the most likely course of action for the owners of warehouses and other non-residential properties since these measures were deemed problematic and difficult to identify for storm surge flooding events.

The adjustments to the residential and non-residential structure inventory were made using the module feature of the HEC-FDA model. The adjusted first-floor elevations were used for the without-project inventory for the years 2024, 2035, and 2085 and for the 2024 with-project structure inventory. A separate module was created for the with-project structure inventories for the years 2035 and 2085. Since partial risk reduction will be provided by each of the project alternatives beginning in the year 2024, the first-floor elevations in these years were not adjusted under with-project conditions. It should be noted that the structures that were elevated between the years 2010 and 2024 are the only structures that were adjusted during the period of analysis under the with-project conditions.

Rationale for the Adjustments. The adjustments made to the 2024, 2035, and 2085 structure inventory were designed to account for the future behavior of property owners whose structures incur repetitive flooding. Beyond the dollar damage and disruptions associated with a flood event, a variety of considerations influence individual property owner rebuild decisions. Significant among these considerations are FEMA requirements for participation in the flood insurance program and the local permitting rules adopted by communities.

FEMA rules require that a structure located within the 1% ACE (100-year) receiving 50 percent or more structural damage from an individual flood event must elevate if it is to be rebuilt/repaired at the original location. Additionally, FEMA has requirements in place to address repetitively damaged properties. FEMA defines a repetitive flood loss property as one that incurs flood damages greater than $1,000 two or more times during a

10-year period. FEMA defines a severe repetitively flooded property as one that incurs flood damage two or more times during a ten-year period with the cumulative value of these damages exceeding the value of the structure, or one that has four claims exceeding a specifically defined amount over the same period. Thus, to be compliant with FEMA rules, severely repetitively flooded properties experiencing such damages would have to be elevated to the 1% ACE (100-year) event level. Property owners could also choose to implement an equivalent mitigation measure or face a significant increase in flood insurance premiums. Finally, the parish could enforce its own elevation requirements for properties in the high-risk flood zones that are severely damaged or are identified as repetitive flood properties, even if the owners are not National Flood Insurance Program policy holders.

As shown in Tables 17 and 18 there is a significant increase in the number of structures incurring flood damages between the 10% ACE (10-year) event and the 4% ACE (25-year) event. The inundation profiles displayed in Tables 17 and 18, along with the probabilities of repetitive flood events for individual structures, provide the basis for identifying a range of structure elevation values to be considered as the decision rule for making an adjustment the structure inventory. Evaluating repetitive flooding probabilities reveals that structures with first flood elevations at or below the 10% ACE (10-year) event have approximately a 26 percent change of being inundated two or more times over a 10-year period. For structures with first floor elevations at or below the 6.7% ACE (15-year) event and 4% ACE (25-year) event, the corresponding inundation chances fall to 14 percent and 6 percent, respectively. Note that selection of a 10-year period for computing multiple flood event probabilities should not be viewed as a definitive value for purposes of this investigation. The value of computing repetitive flooding probabilities is to provide insight regarding the decision rule for making an adjustment the structure inventory. Selection of alternative period lengths would result in different likelihoods of structures experiencing multiple flood events, but the basic relationship of probabilities across ACE events would not change. Ultimately, the adopted decision rule for structure inventory adjustment was based on the distribution of structures across ACE events, FEMA rules for rebuilding, and the expectation that the higher frequency of repetitive flooding associated with being located at the 10% ACE (10-year) event could strongly motivate property owners to take actions to reduce their exposure to flood risk and constitute the most accurate description of the most likely future.

As previously described, the structure inventory was adjusted for repetitive flooding based on the 10% ACE (10-year) water surface elevation exterior to the non-Federal levee. While non-Federal levees provide risk reduction up to the elevation associated with the 6.7% ACE (15-year) event to 5% ACE (20-year) event for approximately 60 percent of the structure inventory, these levees were not considered in this evaluation. However, as long as the non-Federal levees do not fail, structures located in the 6.7% ACE to 5% ACE (15-year to 20-year) floodplain are provided some level of risk reduction above the 10% ACE (10-year) event. This fact contributes to the rationale for using the first floor structure elevations associated with the 10% ACE (10-year) event as the adjustment point for the structure inventory.

Regarding the rationale for timing of the structure inventory adjustments, the following should be noted. Because residents are neither likely to anticipate the increase in relative sea level that is projected to occur between the years 2010 and 2024 nor take proactive mitigation measures in response, the 2024 structure inventory was adjusted based on the 10% ACE (10-year) event for the year 2010. Similarly, the 2035 structure inventory was adjusted based on the 10% ACE (10-year) event for the year 2024, and the 2085 structure inventory was adjusted based on the 10% ACE (10-year) event for the year 2035.

With-Project Expected Annual Damages. The with-project stage probability curves with uncertainty relate the stages on the exterior of the Federal levee system to each probability event. An exterior-interior stage relationship was also entered into the HEC-FDA model for each study area reach. The exterior-interior stage curve relates the stages on the outside of the Federal levee system to the stages on the inside of the Federal levee system for each study area reach. For the Morganza evaluation, the exterior stages were set equal to the water surface profiles from the with-project stage probability relationships for each reach, and the interior stages were set equal to the water surface profiles from the without-project stage-probability relationships. Additionally, since fragility curves were not developed for the Federal levee system, a top of the levee elevation was assigned and entered into the model for each study area reach. This elevation is below the actual top of the levee elevation to account for wave action above the still water stages. At stages below the top of the levee elevation, there is a 100 percent chance that the Federal levee will not fail. At stages equal to or greater than the top of the levee elevation, there is a 100 percent chance that the levee will fail.

The HEC-FDA model used Monte Carlo simulation to sample from the with-project stage-probability relationships with uncertainty for each iteration run by the model. The exterior stage randomly selected by the model was then compared to the top of the Federal levee elevation for each study area reach. If the exterior stage was below the top of the levee elevation, a zero damage value was assigned to that exterior stage. If the exterior stage selected by the model was equal to or above the height of the Federal levee, the related interior stage was used to calculate the damages from the stage-damage relationships with uncertainty. In this case, the with-project interior damages would be equal to the without-project damages for that probability event.

The sum of all damage values divided by the number of iterations run by the model yielded the expected value, or mean damage value, with confidence bands for each probability event. The probability-damage relationships were integrated by weighting the damages corresponding to each magnitude of flooding (stage) by the percentage chance of exceedance (probability). From these weighted damages, the model determined the expected annual damages (EAD) with confidence bands (uncertainty). For the with-project alternative, the expected annual damages (EAD) were totaled for each study area reach to obtain the total with-project EAD under existing (2010) and future (2024, 2035, and 2085) conditions.

Damages resulting from waves overtopping Federal levees were not calculated in this draft of the analysis. Since the top of levee elevations specified in the HEC-FDA model are less than the design top of the Federal levee, wave action above the still water stage has been incorporated into levee performance. Also, the study area reaches south of the city of Houma contain marshland that function as storage area for any excess storm surges attributable to residual wave overtopping. The exclusion of the potential damages from overtopping are not expected to be significant and does not affect plan formulation.

The performance of non-Federal levees was also not included in the calculation of with-project damages for study area reaches that are inside the Federal levee system. If the storm surge overtops the Federal levees, then it is expected that it will also overtop the non-Federal levees. The HEC-FDA model currently does not have the capability to analyze the performance of two levees simultaneously. The exclusion of non-Federal levee performance under the with-project conditions is not considered to have a significant impact on with-project damages.

For those reaches exterior to the Federal levee, the same process was used to calculate damages as was discussed under the without-project conditions. If a non-Federal levee was present in the reach, then a non-Federal levee fragility curve was used along with the with-project stage-damage relationships with uncertainty to calculate damages. If a non-Federal levee was not present in the reach, then the with-project stage-probability curves were used along with the stage-damage relationships with uncertainty to calculate damages. The with-project stages for the exterior reaches could be higher than the without-project stages for a range of probability events. The Federal levee reduces the impact of the storm surge on the interior reaches, but it elevates the stages and induces damages in all exterior reaches.

Induced Damages. The twelve study area reaches located below the proposed Federal levee system incur higher stages for various ACE storm events with the project in place for the years 2024, 2035, and 2085. The HEC-FDA model station numbers associated with these reaches are 163, 169, 175, 235, 256, 316, 340, 490, 496, 508, 604, and 631. Since these reaches experience induced damages as a direct result of the project alternatives, all of the properties in the impacted reaches, which includes 1,010 residential and non-residential structures, would be acquired and the approximately 2,500 residents would be relocated to areas outside the 100-year floodplain. The with-project induced damages, which included damages to residential and non-residential structures, their contents, and vehicles, as well as the debris removal and cleanup costs and damages to streets and highways, were removed for each of these reaches from the total damages for each of the project alternatives. The use of modules was utilized in the HEC-FDA model to remove the induced damages in the affected study area reaches. The cost of the property acquisition totaled $305 million including $249 million for residential structures and $56 million for non-residential structures. The property acquisition and relocation costs were added to the total project costs for the 3% AEP alternative and the 1% AEP alternative. A map of the impacted reaches and a more detailed discussion of the acquisition option can be found in the main report of this evaluation.

Expected Annual Inundation Reduction Benefits. The HEC-FDA model compared the without-project damages with uncertainty to the with-project damages with uncertainty to calculate the expected, benefits with uncertainty for each of the project alternatives. Benefits were calculated for the first year of partial risk reduction (2024), the project base year (2035), and future conditions (2085). Table 19 shows the expected annual without-project damages, with-project damages, and benefits for the years 2024, 2035, and 2085 for the residential and non-residential structures. The tables also show the expected annual benefits at the 25, 50, and 75 percentiles. These percentiles reflect the percentage chance that the benefits will be greater than or equal to the indicated amount.

Benefits During Construction. Construction of both the 3% AEP and 1% AEP alternatives is expected to begin in the year 2014. A closed system with all of the control structures and with at least the first levee lift in place, depending on the levee reach, is scheduled to be completed by the year 2024 for the 3% AEP alternative. A closed system with all control structures and with the first and a significant number of second levee lifts in place is scheduled to be completed by the year 2024 for the 1% AEP alternative. The construction of the 3% AEP storm damage risk reduction system is scheduled to achieve the full design elevation and full project performance in the year 2026, while the construction of the 1% AEP storm risk reduction systems is scheduled to achieve the full design elevation and full project performance in the year 2035. Completion of the initial lift of levee reaches, along with control structures, will provide partial risk reduction for the entire evaluation area. For both the 3% AEP and the 1% AEP alternatives, benefits during construction would accrue for the period 2024 to 2034.   The base year for both alternatives has been designated as 2035.

Engineering inputs, which include without-project and with-project water surface profiles, fragility curves for non-Federal levees, top of Federal levee elevation and exterior-interior stage relationships for each study area reach, were developed for the year 2024. The engineering and economic inputs incorporating uncertainty were used in the HEC-FDA model to calculate the without-project and with-project damages for the two project alternatives during the period of construction. The interim benefits that begin in the year 2024 after the completion of the initial lift of levee reaches, associated locks, and floodgates were computed by comparing the expected annual without-project damages to the with-project damages for each of the alternatives. The annual without and with-project damages were adjusted so that the benefits for each of the alternatives could remain constant through 2035, the base year, for each of the alternatives.

The benefits during construction were compounded forward to the base year, totaled, and then amortized over the 50-year period of analysis using the Federal discount rate of 3.75 percent. The calculation of the benefits during construction claimed for the 3% and 1% AEP alternatives is shown in Tables 20 and 21, respectively.

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Equivalent Annual Damages and Benefits. Damages and benefits for each of the years during the period of analysis were computed by linear interpolation between 2024 and 2084 for both the 3% AEP and the 1% AEP alternative. The FY 2012 Federal interest rate of 3.75 percent was used to compound the stream of expected annual damages and benefits before the project base year and to discount the stream of expected annual damages and benefits occurring after the base year to calculate the total present value of the damages and benefits over the period of analysis. The present value of the expected annual damages and benefits was then amortized over the period of analysis using the Federal discount rate to calculate the equivalent annual benefits. Tables 20 and 21 show the calculation of equivalent annual damages and benefits for each of the project alternatives.

Table 22 shows the equivalent annual residential and non-residential without-project damages, with-project damages, and benefits for each project alternative. Table 23 shows the equivalent annual without-project damages, with-project damages, and benefits for each project alternative for the 24 industrial properties. The tables also show the equivalent annual benefits at the 25, 50, and 75 percentiles. These percentiles reflect the percentage chance that the benefits will be greater than or equal to the indicated values.

OTHER NED BENEFIT CATEGORIES

General. In addition to the physical damages to structures, contents, and vehicles, there are five other categories of NED benefits that are attributable to the Morganza alternatives: avoidance of structure-raising costs, emergency cost reductions, agricultural benefits, safe harbor of large commercial and recreational boat fleets, and municipal water supply benefits. These benefit categories account for less than 10 percent of the total benefits associated with the project alternatives.

Avoidance of Structure-Raising Costs. Typically, property owners in areas that incur repetitive flooding have three options for reducing their flood risk: raise their structures in place, floodproof/retrofit their structures, or relocate to other areas. For purposes of this evaluation, only structure-raising measures were considered to represent the most likely response under future without-project conditions. The avoidance of structure-raising costs by owners of residential and non-residential structures that could incur repetitive flooding and the temporary relocation of the residents can be considered benefits attributable to the project alternatives.

As shown in Table 24, there were 3,092 structures in the evaluation area that have the potential for repetitive flood losses and were elevated during the period of analysis. The cost per square foot to elevate these structures was based on data obtained during interviews conducted by Corps personnel in 2008 with representatives of three shoring firms in the Metropolitan New Orleans area that specialize in the elevation of structures. An average elevation cost per square foot was derived for each one-foot increment from the original elevation of the structure for slab and pier foundation 1-story and 2-story residential structures and mobile homes. The cost of elevating a 1-story slab foundation residential structure was used for all non-residential structures. These costs were updated to October 2011 price levels using Civil Works Construction Costs Index (CWCCIS). Table 25 shows the costs per square foot of elevating each structure type for each one-foot increment of elevation up to 13 feet.

The total cost for the temporary relocation of residents during the two-month elevation process includes lodging, the labor costs associated moving personal property into and out of a POD, and the storage of these contents. The average furnished apartment rental in the Houma area was determined to be $1,200 per month based on advertised rental properties. The average POD rental, which includes pick-up and delivery, was determined to be $700 for the two-month period. The average labor cost for moving personal property into and out of the POD was determined to be $650 based on the quote from the POD company. The temporary relocation cost using October 2011 price levels totaled $3,750 for each elevated structure.

The elevation cost per square foot, based on the type of structure, number of stories, foundation type, and the number of feet elevated, was multiplied by the square footage of the footprint of each raised structure obtained from the structure inventory collected in 2009 for the evaluation area. The temporary relocation cost per structure was added to the elevation cost to derive the total structure raising cost. Table 26 shows the number of structures elevated, the average height that the structures were elevated, the total cost of elevating these structures, and the average elevation cost per structure.

The total cost of raising 703 structures between the years 2010 and 2024 was calculated to be approximately $108.3 million. The average elevation cost per year during this period was $7.2 million. The cost of raising 464 structures between the years 2025 and 2035 totaled approximately $94.9 million with an average cost per year of $8.6 million, and the cost of raising 1,855 structures between the years 2036 and 2085 totaled $238.2 million with an average cost per year of $4.8 million. The present value of these annual average costs was totaled and then amortized over the period of analysis (2024 through 2084) using the current Federal discount rate of 3.75 percent.

Emergency Cost Reduction. The NED costs associated with each of the emergency activities conducted by the public and private sectors before, during, and after storm events, and infrastructure damage to roads and utilities were estimated based on data obtained during interviews with professionals who are familiar with emergency activities and infrastructure inundation impacts. More than 100 organizations and over 150

individuals were contacted as part of the interview process, and responses were obtained from 39 experts. The interviews were conducted between December 2009 and March 2010. The information compiled as part of the interview process was used to develop depth-emergency cost and depth-infrastructure damage relationships for the Morganza evaluation area. The results can be found in the final report dated March 2012 entitled, *Development of Depth-Emergency Costs and Infrastructure Damage Relationships for Selected South Louisiana Parishes*.

The emergency costs in the report were divided into six groups: evacuation activities (evacuation, subsistence, and reoccupation), debris removal and cleanup, public utilities, infrastructure, public services patronized, and public services produced. The public utilities group was divided into five subcategories: natural gas, electricity, telecommunications, sewage and wastewater treatment, and water supply. The infrastructure group was divided into seven subcategories: streets and highways, bridges, railroads, ports, airports, land-based pipelines, and petroleum wells. The public services patronized group was divided into six subcategories: education, libraries, indoor recreation facilities, medical, eldercare, and daycare. The public services produced group was divided into five subcategories: police, fire, incarceration, judicial, and government administrative.

The damage relationships for each subcategory were generated for two flood event scenarios, along with three depths of flooding for each scenario: freshwater short duration (less than two days) and saltwater long duration (two days or more), and flooding depths of 2 feet, 5 feet, and 12 feet. However, only the saltwater long duration depth-damage relationships were applied in this analysis. The flooding event was assumed to affect a typical area occupied by 3,800 households or 10,000 residents. An individual questionnaire was developed for each of the emergency cost groups and subcategories. The experts were asked to provide a minimum, most likely, and maximum estimate for a variety of parameters required to compute the costs/damages for each of the subcategories. The experts were instructed that the range between the minimum and maximum values was not expected to represent absolute minimum and maximum values, but rather the 90th percentile of the possible outcomes.

The responses from the experts for each estimated parameter were combined and averaged to generate aggregated minimum, most likely, and maximum values. These aggregated values were used to specify a triangular probability distribution. The triangular distributions were used as inputs in an @Risk (Version 5) spreadsheet constructed to produce a distribution of results representing the cost/damage for the subcategory. The distribution fitting feature of @Risk was used to identify the probability distribution functional form that best fit the output of the @Risk spreadsheet based on the Chi-Squared statistic. In all cases the normal distribution was found to represent the best fit. (In identifying the best fit functional form, the normal and triangular distributions were considered.) Consideration was limited to these two because the ultimate use of this information was input as depth-damage functions into HEC-FDA, which is limited to these two functional forms.

The mean dollar damages at each of the three depths of flooding were converted to a percentage of the total cost/damage estimate at the 12 feet depth of flooding. In addition to the three estimated depth of flooding points, a zero damage point was also specified at 1.9 feet of flooding. (This forced damage to begin at 2 feet of flooding.) Once expressed as a percentage, mean values the four depth of flooding data points are structured in the conventional manner that is employed with HEC-FDA. The standard deviation at each depth of flooding was handled in a similar manner as the mean value, with each of the dollar value standard deviations expressed as a percentage of the mean cost/damage dollar value at 12 feet.

The cost/damage depth-damage relationships were entered into the HEC-FDA model, along with information about the structures and infrastructure obtained from an inventory compiled for the study area, including structure type, study area reach, and foundation height, and the engineering inputs (stage-probability relationships and levee fragility curves) to determine the emergency cost reduction benefits attributable to each of the project alternatives. The cost/damage value at 12 feet of flooding was used as the emergency cost or infrastructure value for each "structure inventory" record in the HEC-FDA model.

For this evaluation, only the debris removal and cleanup of the residential and non-residential structures, and the physical damages to streets and highways were quantified and included in the net benefit analysis for the project alternatives. The evacuation, subsistence, and reoccupation costs and the police and fire department relocation costs were quantified, but were not included in the net benefit analysis. These emergency cost categories were quantified in the March 2012 report.   The depth-damage results can be found in Chapter 6 of the March 2012 report. The responses to the questionnaires can be found in Attachment 2 of the report.

Debris Removal and Cleanup. Immediately after the floodwaters from a tropical event subside, the public and private sectors of the flooded community must begin the cleanup process. The first activities that typically take place include the removal of debris from roads and yards. The streets must be made accessible for use by emergency vehicles and for residents to return to their homes. Most of this type of debris is either vegetative or sediment debris left after the floodwaters subside. While these categories of debris could be a significant part of the cleanup process, they are not addressed in this analysis. This analysis has included the collection, processing, and proper disposal of the debris material from the inside of the inundated structures, which varies according to residential or non-residential occupancy type of the structure. This type of debris includes content-related debris, white goods, electronics, and hazardous waste (paints, oil, household chemicals, poisons, etc.). Hazardous debris must be properly disposed of so as to minimize the existing and future threats to human health and the environment.

Interviews were conducted with four experts in the fields of debris collection, processing, and disposal. The questionnaires used in the interview process were designed to elicit information from the experts regarding the cost of each stage of the debris cleanup process by structure occupancy type. The experts were asked to provide a minimum,

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most likely, and maximum estimate for the cleanup costs associated with the 2 feet, 5 feet, and 12 feet depths of flooding. A prototypical structure size in square feet was used for each of the five residential occupancy categories and for each of the eight non-residential occupancy categories. The experts were asked to estimate the percentage of the total cleanup caused by floodwater and to exclude any cleanup that was required by high winds.

The total amount of content-related debris estimated for each structure occupancy type was expressed in cubic yards per structure. The white goods were expressed in units per structure, and the electronics and hazardous materials were expressed in pounds of debris per structure. A minimum, most likely, and maximum cost estimate was provided for the collecting and processing of each cubic yard of content-related debris, each white goods unit, and each pound of electronics and hazardous waste for the saltwater long duration flood scenario. A minimum, most likely, and maximum cost estimate was also provided for the removal, hauling away, and disposal of the debris. The minimum, most likely, and maximum estimates from each expert were converted into aggregated values (as previously described) for each structure occupancy type and were entered into the @Risk spreadsheet as triangular distributions. Fitting of the @Risk spreadsheet output to a probability distribution functional form and conversion of the probability distribution information into HEC-FDA depth-damage input was accomplished as previously described. The mean debris removal and cleanup costs and the standard deviations for each of the three depths of flooding are shown by structure occupancy type in Table 27.

The cost/depth-damage relationships for each structure occupancy category were converted to percentages and entered into the HEC-FDA model, along with the debris and cleanup structure records (cost/damage value at 12 feet of flooding was used as the emergency cost or infrastructure value) and engineering inputs (stage-probability relationships and levee fragility curves) to calculate the expected annual without-project and with-project debris removal and cleanup costs for the years 2024, 2035, and 2085. The expected annual costs were converted to equivalent annual values using the current Federal discount rate of 3.75 percent and a 50-year period of analysis. Since the costs were initially expressed in 2010 price levels, the equivalent annual without-project and with-project values were updated to October 2011 price levels and are shown in Table 28. It should be noted that the adjusted structure inventory for repetitive flooding was used to calculate the reduction in debris removal and cleanup costs.

Damages to Infrastructure. The reduction of potential flood damages to the infrastructure (streets and highways, bridges, railroads, ports, airports, land-based pipelines, and petroleum wells) in an evaluation area can form a significant category of benefits attributable to a project alternative. For purposes of this analysis, only the damages to streets and highways were considered. Streets are defined as roadways with two lanes with relatively lower volumes of traffic and access, while major and secondary highways are defined as roadways with four lanes with relatively higher volumes of traffic and access.

GIS data were used to determine the number of miles of streets and highways in each of the study area reaches in the Morganza evaluation area. Within each study area reach, a grid was used to create individual HEC-FDA structure records. Each structure record was equal to one 1,000 feet x 1,000 feet grid unit. The NAVTEQ, Inc. database was then used to obtain the number of miles of streets and highways within each grid unit. A mean ground elevation was assigned to the grids based on LIDAR data.

Interviews were conducted with an expert in street and highway construction to determine the cost of repairing each mile of roadway. Costs were provided for three roadway components. The components of streets include street surface, street base, and street curb, while the components of major and secondary highways include road surface, road base, and road shoulder. A minimum, most likely, and maximum replacement value per mile, which included materials and labor, was assigned to each component. The expert was then asked to provide an estimate of the depreciation that has taken place in each roadway based on the age of the roadway. The value of each mile of roadway component was discounted by the estimated depreciation percentage. Finally, the expert was asked to estimate the percentage of the road components that would be damaged at the 2-feet, 5-feet, and 12-feet depths of flooding.

The damage to the streets and highways per mile was calculated by multiplying the cost of the materials and labor to replace each infrastructural component by the inverse of the depreciation percentage by the percentage damage to each component. The minimum, most likely, and maximum damages for each roadway component were used to develop a range of values for the total cost of the infrastructural damages for each mile of roadway. The triangular probability distributions were input to the @Risk model, and the probability distribution fitting feature was used to find the distribution that best fit the output. The normal distribution was found to fit the infrastructural damage outputs better than the triangular distribution. The mean value for the damages per mile and standard deviations for each of the three depths of flooding are shown for major and secondary highways and streets in Table 29.

The depth-damage relationships for major and secondary highways and streets were converted to percentages and entered into the HEC-FDA model, along with the major and secondary highways and streets structure records (damage value at 12 feet of flooding was used as the infrastructure value) and engineering inputs (stage-probability relationships and levee fragility curves) to calculate the expected annual without-project and with-project major and secondary highways and streets for the years 2024, 2035, and 2085. The expected annual costs were converted to equivalent annual values using the current Federal discount rate of 3.75 percent and a 50-year period of analysis. Since the costs were initially expressed in 2010 price levels, the equivalent annual without-project and with-project values were updated to October 2011 price levels and are shown in Table 30.

Agricultural Benefits. An economic analysis of the agricultural lands in the study area was conducted to determine the number of acres impacted in the study area. The

National Agricultural Statistical Service (NASS) geo-spatial information system for the year 2010 data were used to identify the agricultural land and crop distribution in each of the study area reaches. Agricultural activity was found in 35 of the Morganza study area reaches. The relationships for the without-project (2010, 2035, and 2085) conditions and for the with-project alternatives (2035 and 2085) conditions were used with the top of levee elevation for the non-Federal and Federal levees to determine the average annual flooded acres. Table 31 shows the average annual flooded agricultural acres by study area reach under the without-project conditions for the years 2010, 2035, and 2085 and under the with-project alternatives conditions for the years 2035 and 2085. Even if a high estimate of the net revenue generated by an average annual acre was used in the analysis, the total agricultural benefits would only equal approximately one percent of the total inundation reduction benefits to structures, contents, and vehicles for each of the project alternatives. Thus, estimates of agricultural benefits were not included in the net benefit computations.

Safe Harbor Benefits for Boat Fleets. In addition to the HNC, five bayous located in the coastal portion of the study area are used as navigational routes to and from the Gulf of Mexico: Bayous DuLarge, Grand Caillou, Petit Caillou, Terrebonne, and Pointe aux Chenes. Large commercial and recreational vessels dock along these waterways because of the proximity to the Gulf of Mexico and other fishing grounds, but these vessels must be moved upstream to safer harbors during tropical events. Storm surges could cause physical damages to moored vessels by tossing them into adjacent vessels or docks, pushing debris into the vessels, or washing them up on land. Since the project alternatives would reduce the impact that storm surges have on the waterways, vessels would not have to be moved to sheltered locations. The reduction in physical damages to the large commercial and recreational boat fleet and the reduction in the costs of moving these vessels inland to safer waterways are considered benefits attributable to the project. However, the physical damages were not quantified in this analysis.

According to data obtained from the Louisiana Department of Wildlife and Fisheries (LDWF) and the U.S. Coast Guard (USCG), there were 1,574 motorized vessels greater than 25 feet in length registered in Terrebonne Parish in 2009. These vessels were grouped into five categories: 949 were classified as commercial fishing vessels, 361 were classified as recreational boats, 140 were classified as oil and gas crew boats, 33 were used as commercial passenger vessels, and 91 were designated as other commercial vessels. Vessels less than 26 feet in length were not included in the analysis because they are typically removed from the water in advance of an approaching storm and would not benefit from the construction of the project.

Projections for the Motorized Vessel Fleet. The number of vessels in the commercial fishing and recreational fleets was projected for the years 2024, 2035, and 2085. The projections were based on both historic trends in boat registrations and economic growth patterns, and they were made for median, high growth, and low growth scenarios. The number of oil and gas related vessels was not projected because the project alternatives would not have an impact on these vessels. According to industry representatives, oil and

gas related vessels are used to evacuate crewmen from offshore oil rigs prior to tropical events, and these vessels typically leave for other ports outside of the evaluation area. The number of vessels in the commercial passenger fleet and the other vessel fleet was projected to remain constant throughout the evaluation period. Projections for the commercial fishing, recreational vessel, and commercial passenger fleets were obtained from the draft report entitled *Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets* dated March 2012.

Commercial Fishing Vessels. The commercial fishing fleet consists primarily of shrimp boats, but it also includes vessels used to harvest oysters and finfish. The number of commercial fishing vessels registered with the LDWF has been declining since the late 1990s due to an industry trend toward larger, but fewer, vessels. While the number of vessels less than 26 feet in length decreased 18.5 percent from 1,115 in 1999 to 909 in 2008, the number of vessels in the 40 to 65 feet range increased over 400 percent from 46 in 1998 to 236 in 2008. The overall number of registered commercial fishing vessels decreased 10.8 percent from 1,757 in 1997 to 1,568 in 2008, which is an average annual decline of 0.44 percent.

The projections for the commercial fishing fleet were based on the annual brown and white shrimp catch for the state of Louisiana during the 13-year period 1997 through 2009. During this period, Terrebonne Parish contributed approximately 30 percent of the total shrimp catch in the state. The Terrebonne shrimp catch totaled 25.9 million pounds in 2009 and averaged 32.9 million pounds annually between 1997 and 2009. The average catch size for the 13-year period was used as the median value in the projections. The low estimate of 21.5 million pounds was calculated by subtracting two standard deviations from the median value. The high estimate of 44.3 million pounds was calculated by adding two standard deviations to the median value.

Historical trends were used to determine the percentage of the total shrimp catch caught by vessels in each of the size categories. The percentage caught by the smaller crafts was projected to decrease through the year 2040, while the percentage caught by the larger crafts was projected to increase through the year 2040. After the year 2040, these percentages were projected to remain constant. Table 32 shows the percentage of the total shrimp catch caught by each vessel size for the years 2009, 2024, 2035, and 2085.

The 2002 LDWF report was used to calculate the median catch for each vessel size. Table 33 displays the median shrimp catch for each vessel size.

The projected number of vessels in each size category for the low, median, and high growth scenarios in the years 2024, 2035, and 2085 is shown in Table 34. These numbers are based on the annual shrimp catch, the percentage of the total catch by vessel size, and the median catch for each vessel size. As an example, the projected number of vessels over 65 feet in length for the high scenario in the year 2024 was calculated by multiplying 44.3 million pounds (the high estimate of the total shrimp

catch) by 15.9 percent (the percentage of the total catch for vessels over 60 feet in length from Table 32). This product was then divided by 69,050 (the median shrimp catch for vessels over 60 feet in length from Table 33) to estimate that there will be 102 vessels over 60 feet in length in the year 2024 under the high growth scenario.

Recreational Vessels. The large recreational vessel fleet experienced an average annual growth rate of 2.0 percent between 1999 and 2009 and an average annual growth rate of 2.9 percent between 2002 and 2009. This growth can be attributed to population growth and rising median income. Table 35 shows the annual growth in the number of recreational vessels by vessel size for the years 1999 to 2009 and 2002 to 2009.

In the median forecast, the average 10-year annual growth rate was extended through the year 2040, and a slower growth rate was used for the period between 2040 and 2085. In the low growth estimate, the size of the recreational vessel fleet is projected to remain constant during the year 2085. In the high growth estimate, the size of the recreational vessel fleet is based on the 2002 through 2009 growth rate. Table 36 shows the forecasted growth rates by vessel size for the median and high growth scenarios. Table 37 shows the projected number of recreational vessels for the years 2024, 2035, and 2085 under the low growth, median, and high growth scenarios.

Commercial Passenger Vessels. The commercial passenger fleet consists of charter fishing vessels that are similar to recreational vessels but with a different type of ownership. While commercial passenger fleet in Terrebonne Parish declined 2.9 percent annually from 68 vessels in 1997 to 56 vessels in 2003, the fleet increased 5.9 percent annually from 56 vessels in 2004 to 79 vessels in 2009. Overall, the number of commercial passenger vessels grew at an average annual growth rate of 1.2 percent. Due to the fluctuations in the number of vessels during the 13-year period, the median commercial passenger fleet was projected to remain constant at 33 vessels for the years 2024, 2035, and 2085. Of this total, 21 vessels were between 26 feet and 40 feet in length, 9 vessels were between 40 feet and 65 feet in length, and 3 vessels were over 65 feet in length. The low estimate is based on an annual decrease of 1.0 percent through the year 2040 and an annual decrease of 0.5 percent from the year 2040 through the year 2085. The high estimate is based on a 1.0 percent annual increase through the year 2040 and a 0.5 percent annual increase from the year 2040 through the year 2085. The projected number of commercial passenger vessels for the years 2024, 2035, and 2085 for the low growth, median, and high growth scenarios is shown in Table 38.

Evacuation Travel Distances. The homeports of the motorized vessel fleet under the without-project condition were determined based on interviews with experts in the Terrebonne Parish maritime industry. As shown in Table 39, 60 percent of the vessels dock along Bayous Petit Caillou and Grand Caillou, and 30 percent dock along Bayous DuLarge and Terrebonne. The remaining 10 percent dock along the HNC and Bayou Pointe aux Chenes. The homeport of 78 percent of the vessels is

located above the project alternatives, while the homeport of 22 percent of the vessels is located below the project alternatives.

The distribution of the vessel fleet along the six waterways was used to estimate the average number of nautical miles that a vessel would travel to evacuate from storms more intense than the 10% ACE (10-year) event. Under the without-project condition, all vessels would have to travel north of their homeport to seek shelter during tropical events. Under the with-project conditions, only the vessels with homeports below the proposed alternatives would have to evacuate to safer locations. The weighted average number of nautical miles each vessel would have to travel in advance of an approaching storm event was determined to be 10.94 nautical miles under without-project conditions and 1.14 nautical miles with the proposed alternatives in place. The distances traveled to flee an approaching storm under without and with-project conditions are shown in Table 40. The total distances traveled to flee approaching storms were calculated under the without-project and the with-project conditions by multiplying the number of vessels in each vessel type and size category by the weighted average travel distances.

Travel Costs per Nautical Mile. The cost per nautical mile by vessel size category for commercial fishing, recreational/commercial passenger (charter fishing), and other commercial vessels was estimated using fuel and crew costs. The average speed and fuel consumption for each size category by vessel type was determined based on the characteristics of used vessels available for sale on the websites www.MaritimeSales.com and www.YachtWorld.com. The cost of diesel fuel was based on the 3-year average of Gulf Coast monthly fuel costs during the period March 2009 through February 2012 from the U.S. Energy Information Agency (EIA). Crew costs were estimated at $15 per hour for crew members and $20 per hour for captains. Crew sizes varied by vessel type and size, with larger vessels requiring more manpower. Table 40 show the calculation of the travel costs per nautical mile for the commercial fishing vessels, recreational/commercial passenger vessels, and other vessels by size category.

Travel costs were calculated using October 2011 price levels for the without-project and the with-project conditions by multiplying the projected number of vessels in the fleet by the weighted average travel distance from by the average operating cost per nautical mile. Table 41 shows the without-project and with-project total vessel travel costs for each vessel size for the low growth, median, and high growth scenarios for the years 2024, 2035, and 2085. The difference between the without-project travel costs and the travel costs with the project alternatives in place is considered the travel cost reduction benefit attributable to the project alternatives.

Expected Annual Travel Costs Reduction. The without-project and with-project travel costs were integrated by weighting the travel costs by the percentage chance of exceedance (probability) for those ACE events equal to and more intense than the 10% ACE (10-year) event. From these weighted travel costs, the expected annual travel costs were calculated for the without-project and with-project conditions for the low growth, median, and high growth scenarios for the years 2024, 2035, and 2085. The difference between the without-project and the with-project expected annual travel costs is

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considered the benefit attributable to the project alternatives. The expected annual travel costs reductions are shown in Table 43.

Expected Annual Physical Damage Reduction for the Vessel Fleet. A depth-damage curve relating the height of the storm surge above normal sea level at one-foot increments to the percentage of the vessel damaged was developed for the vessel fleet as part of the draft report entitled *Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets* dated March 2012. The damage percentages were based on data collected in the Louisiana coastal region following Hurricanes Katrina and Rita. However, because sufficient documentation supporting the development of the depth-damage relationships was not available, physical damages to the vessel fleet were not calculated used in the net benefit analysis for the Morganza PAC Report.

Municipal Water Supply Benefits.   The Terrebonne Water District Number 1 is responsible for supplying drinking water to the residents of Terrebonne Parish. The city of Houma and the town of Grand Caillou are served by a water treatment facility located at the confluence of the Houma Navigation Canal (HNC) and the Gulf Intracoastal Waterway (GIWW), which draws water from the GIWW. The remainder of Terrebonne Parish is served by the water treatment facility in Schriever that draws water from Bayou Lafourche. The Schriever plant is also periodically used to supplement the Houma water supply. Under existing conditions, above normal salinity levels occur each year during late summer and early fall in the GIWW, which impacts the Houma Water Treatment Plant (HWTP), and in the portion of Bayou Lafourche located between the Company Canal in Lafourche Parish and the GIWW, which impacts the Schriever Water Treatment Plant (SWTP). The HNC has been identified as the major conduit for the intrusion of saltwater from the Gulf of Mexico**.** During periods of high salinity levels, the HWTP and the SWTP must obtain water from the Bayou Black Reservoir in order to meet their municipal water supply demands.

Water for Lafourche Parish is provided by five water treatment facilities located along Bayou Lafourche. The Lockport facility, which is located on Bayou Lafourche downstream from the Company Canal, is the only plant in Lafourche Parish to have reported excessive salinity levels. Since the Lockport water treatment facility has no alternative water sources, the plant typically treats the saltwater and then sends out advisories to the residents of the area.

Average Annual Number of Days of High Salinity. Since the HNC was constructed in 1961, chloride concentrations at the Houma Water Treatment Plant (HWTP) have exceeded the State standard of 250 parts per million (ppm) an average of 37 days per year with a standard deviation of 25.6 days. The number of days of high salinity ranged from a high of 109 days in 1999 to a low of zero days in 1961, 1989, and 1993. The @Risk program was used to determine that a Gamma probability distribution would best fit these data for the period between 1961 through 2011. The Gamma probability distribution was then used to predict the number of days of high salinity for each year in the period of analysis (2012 through 2084). The expected value for the number of days of high salinity

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during the period is 31.2 days per year, not accounting for rising sea level. The distribution was truncated so as not to generate values below zero days of high salinity.

Construction of the lock-gate complex on the HNC, which is projected to be completed in the year 2019, will reduce the amount of saltwater intrusion into the evaluation area. The number of days with salinity greater than 250 ppm with the project in place depends on how the HNC and Bayou Grand Caillou navigational and environmental structures are operated. A system-wide salinity model was used by Engineering to assess the salinity at 78 locations throughout the project area. However, for purposes of this analysis, only the three locations closest to the HWTP were used to determine the reductions in salinity. Salinity levels were simulated in the model to compare the reduction in salinity levels under the without-project and with-project conditions for year 2004. Under the without-project condition, there were 53 days of salinity greater than 250 ppm. With the HNC floodgate closed and all other environmental and navigational structures open, the number of days of high salinity was reduced to 40 days, which is a decrease of 23.91 percent. With the HNC lock and the other structures open, the number of days was only reduced to 49 days. With the HNC lock open and all environmental structures closed, the number of days was only reduced to 50 days. For this analysis, the most likely operation is for the HNC gate to be closed and the other structures to be open.

Additional Costs Associated with High Salinity Levels. The expected annual number of days for each of the years in the period of analysis was then multiplied by the increase in chemical costs per million gallons (MG) per day using water from Bayou Black instead of the GIWW. Based on information provided by HWTP, the incremental treatment cost is $84.79 per MG. The incremental cost was then multiplied by the average number of gallons treated per day of 4.056 MG based on data from FY 2008-09 to determine the average daily increase in treatment cost. The average daily treatment cost was calculated to be $343.94. This cost was then multiplied by the average number of days of high salinity for each year to determine the average annual cost under the without-project conditions.   The average daily treatment cost under the with-project conditions was determined by reducing the without-project cost by 23.91 percent beginning in the year 2019.

Additionally, granular activated carbon (GAC) must be added to water obtained from Bayou Black because the high level of total organic carbon (TOC) in the water decreases the life of the GAC in the water supply. The cost of each GAC treatment was estimated by the SWTP to be $275,000. Without the project in place, the GAC would need to be replaced every 3 years and with the project in place, the GAC would need to be replaced every 4 years. The reduction in the number of years in the GAC replacement cycle generates a cost savings during the period of analysis (2019 through 2084).

The project alternative would reduce the costs associated with the operation of the four gates located along Bayou Black (Water Proof Pump Station, Minors, Hanson, and Elliot Jones) to prevent saltwater intrusion. The power usage under the without-project condition costs $330 per year, while the power usage with the project in place costs $251 per year. The reduced power usage leads to an estimated cost savings of $79 per year.

The gates would no longer need to be refurbished every 20 years at a cost of $135,000 and replaced every 40 years at a cost of $1 million. Thus, the project would create a cost savings of $2.27 million during the period of analysis. Finally, the Water Proof Pump Station would no longer need to be refurbished every 20 years at a cost of $135,000 and replaced every 40 years at a cost of $500,000. This would create a cost savings of $1,270,000 during the period of analysis (2019 through 2084).

Annualized Cost Savings. Table 44 shows the projected annual increase in the cost of supplying water that results from increased salinity under the without-project and with-project conditions in October 2011 price levels. The difference between the two total costs is the total cost savings attributable to the project alternative. The total cost savings were annualized over the period of analysis using the current Federal discount rate of 3.75 percent to determine the average annual cost savings or benefits associated with the project alternative.

PART 4: LIFE CYCLE COSTS OF THE PROJECT ALTERNATIVES

CONSTRUCTION OF THE PROJECT ALTERNATIVES

Construction Schedule. Construction of each of the project alternatives is scheduled to begin in the year 2014 and will continue through the year 2070 for the 3% AEP alternative and through the year 2071 for the 1% AEP alternative. The authorized levee alignment for each of the alternatives will be constructed utilizing the existing non-Federal levee systems throughout the area whenever possible and will be constructed in phases due to the relatively poor foundation conditions and the absence of quality burrow material. The 3% AEP alternative requires one or two levee lifts, depending on the levee reach, to achieve the design elevation by the year 2026. Two additional levee lifts are scheduled after the year 2026 to maintain the design elevation. The 1% AEP alternative requires two or three levee lifts, depending on the levee reach, to achieve the design elevation by the year 2035. Three additional levee lifts are scheduled after the year 2035 to maintain the design elevation. The first levee lifts will be overbuilt and allowed to settle for several years before the later levee lifts are added. The later lifts will account for the relative sea-level rise and subsidence that is projected to occur throughout the

period of analysis. The life cycle costs also include the construction of sector gates and a lock structure on the Houma Navigation Canal and the major periodic rehabilitation cost of these navigation structures.

Average Annual Costs. Life cycle cost estimates were provided for both the 3% AEP and the 1% AEP alternatives in October 2011 price levels. The first costs, along with the schedule of expenditures, were used to determine the interest during construction and gross investment cost at the end of the installation period (2035 for the 3% AEP alternative and the 1% AEP alternative). The current Federal discount rate of 3.75 percent was used to discount the costs to the base year and then amortize the costs over the 50-year period of analysis. After the average annual construction costs were calculated, the annual operations and maintenance costs were added.

Tables 45 and 46 provide the life cycle costs for each of the project alternatives, the average annual construction costs, the annual operation and maintenance costs, and the total average annual costs.

PART 5: RESULTS OF THE ECONOMIC ANALYSIS

NET BENEFIT ANALYSIS

Calculation of Net Benefits. The expected annual benefits attributable to each of the project alternatives for each of the benefit categories were converted to an equivalent time frame by using the current Federal discount rate of 3.75 percent. The base year for this conversion is the year 2035 for the 3% AEP alternative and the 1% AEP alternative. The equivalent annual benefits were then compared to the average annual costs to develop a benefit-to-cost ratio for each alternative. The net benefits for each alternative were calculated by subtracting the average annual costs from the equivalent annual benefits.   The net benefits were used to determine the economic justification of each of the project alternatives.

Comparison of Net Benefits for the Project Alternatives. Tables 47 and 48 summarize the equivalent annual damages and benefits, total annual costs, benefit-to-cost ratio, and equivalent annual net benefits for the 3% AEP and the 1% AEP alternatives. Tables 49 and 50 show the net benefits for the project alternatives using only the existing condition (2010) structure inventory for the 3% and 1% AEP alternatives.

Sensitivity Analysis. The purpose of this sensitivity analysis was to investigate the impact that a change in depth-damage relationships from an adjacent area would have on the net benefits and benefit-to-cost ratios of the 3% AEP alternative and the 1% AEP alternative. The saltwater long-duration depth-damage relationships developed by a panel of experts as part of the Donaldsonville to the Gulf evaluation were applied to the residential and non-residential structures, contents, and vehicles in the Morganza evaluation area. The depth-damage relationships developed for the Donaldsonville evaluation are shown in Table 51. The net benefits and benefit-to-cost ratios calculated for the two project alternatives using the Donaldsonville to the Gulf depth-damage relationships are shown in Tables 52 and 53.

In addition to the sensitivity analysis performed by applying the depth-damage relationships for the Donaldsonville to the Gulf of Mexico evaluation, sensitivity analysis was also performed to determine the impacts of changes in sea level rise and subsidence and changes in the project interest rate on the damages, benefits and costs for each of the project alternatives.

The difference in stages across all probability events for all study area reaches under without-project conditions between 2035 and 2085 averaged approximately 1.6 feet. This increase in water surface elevations reflects the sea level rise and subsidence projected to occur in the evaluation area during the period. To estimate the sensitivity of damages and benefits to changes in sea level rise and subsidence for the project alternatives, damages were held constant between the years 2035 and 2085. This resulted in a 21 percent decrease in without-project damages between 2035 and 2085, and a 17 percent decrease in benefits for the 3 % AEP and 1% AEP alternatives over the period of analysis (2024-2085). The average annual cost of the project alternatives declined 0.9 percent because the height of the flood risk management structures was lowered without projected sea level rise and subsidence. The adjustments for the 3% AEP alternative resulted in a 64 percent decline in the net benefits and a decrease in the benefit-to-cost ratio from 1.34 to 1.12. The adjustments resulted in a 72 percent reduction in net benefits and a decrease in the benefit-to-cost ratio from 1.30 to 1.08 for the 1% AEP alternative.

Sensitivity analysis was also performed to determine the sensitivity of damages, benefits and costs to changes in the project interest rate used in the economic analysis. When the interest rate used in the analysis is increased from the current rate of 3.75 percent to 5.0 percent, the benefit-to-cost ratio decreases to approximately 1.0 for both the 1% AEP alternative and the 3% AEP alternative. The project alternatives would not be economically justified at any interest rate above 5 percent.

Update to 2012 Price Level. The damages, benefits, and costs values were updated to a 2012 price level and are shown in Table 54 for the 3% AEP alternative and Table 55 for the 1% AEP alternative. The following indexes were used to update the benefit categories from 2011 to 2012: the Construction Index developed by the Bureau of Labor Statistics was used for residential and non-residential benefit categories, including the industrial benefit category, and the avoided structure-raising costs category; the National

Highway Construction Cost Index was used for the highway and streets benefit categories; the Remediation Services Index developed by the Bureau of Labor Statistics was used for the debris removal and cleanup benefit category and the Diesel Fuel Price Index developed by the Energy Information Administration was used for boat fleets benefit category. Project costs were estimated to reflect 2012 prices.

RISK ANALYSIS AND PROJECT PERFORMANCE

Benefit Exceedance Probability Relationship. The HEC-FDA model used the uncertainty surrounding the economic and engineering inputs to generate results that can be used to assess the performance of the two project alternatives. A spreadsheet was developed using the expected annual damage and benefit results from the HEC-FDA model to calculate the equivalent annual without-project and with-project damages and the damages reduced for each of the project alternatives. Table 56 shows the equivalent annual benefits at the 75, 50, and 25 percentiles. These percentiles reflect the percentage chance that the benefits will be greater than or equal to the indicated values. A trend function was applied to estimate the forecasted damage reduction above the 75 percentile for each of the project alternatives. The benefit exceedance probability relationship for each of the project alternatives can be compared to the point estimate of the average annual costs for each of the project alternatives. The table and graphs for each of the project alternatives shows the percent chance that the benefit-to-cost ratio will be greater than one and the net benefits will be positive.

Residual Risk. Residual risk is the flood risk that remains in the floodplain after a proposed flood risk management alternative is implemented. It includes the consequence of capacity exceedance as well as consideration of project performance. Table 57 shows the number of structures damaged and the structural damages in dollars under the without-project conditions for each of the eight ACE events, the residual damages in dollars under the with-project conditions for the 2% ACE (50-year),1% ACE (100-year), 0.5% ACE (200-year), and 0.2% ACE (500-year) events, and the percentage of the total number of structures including automobiles, residential structures, commercial structures, and mobile homes damaged by each of the four ACE events for the year 2035. All three ACE events exceed the design of the 3% AEP alternative, while only the 0.5 % ACE (200-year) event and 0.2% ACE (500-year) event exceeds the design of the 1% AEP (100-year) alternative. The residual damages in each of these cases are higher than the without-project damages because structures below the 10% ACE (10-year) event are elevated to above the 1% ACE (100-year) event to account for the response of residents to repetitive flood losses beginning in the year 2024. Finally, the table shows the minimum and maximum flood depths under the without-project conditions, which assumes that the non-Federal levees will fail, for each of the four ACE events.

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Table 58 shows the number and the percentage of the total structures in the study area that would be inundated at three-foot increments of flooding at the under the without project conditions in the year 2035. The residual damages with the proposed Federal alternatives would be higher due to structures being elevated under the without-project condition to account for the response of residents to repetitive flooding, but not elevated with the project alternatives in place. For example, 19 percent of the structures would not be inundated, 12 percent of the structures would receive between 0 and 3 feet of flooding, and approximately 36 percent would have a depth of flooding between 3 and 6 feet above the first floor elevation.

AEP by Reach for the Years of Analysis. The results from the HEC-FDA model were also used to calculate the long-term annual exceedance probability (AEP) and the conditional non-exceedance probability, or assurance, for various probability storm events. The model provided a target stage to assess project performance for each study area reach under both existing (2010) and future (2024, 2035, and 2085) without-project and with-project conditions. For study area reaches without Federal or non-Federal levees, the target stage was set by default at the elevation where the model calculated five percent residual damages for the 1% ACE (100-year) event. For levees without geotechnical failure, which includes the Federal levees in the Morganza analysis, the target stage was set equal to the assigned top of the Federal levee elevation. For levees with geotechnical failure, which includes the non-Federal levees in the Morganza analysis, the target stage was computed based on the joint probability of annual exceedance and probability of geotechnical failure.

The model calculated a target stage AEP with a median and expected value that reflected the likelihood that the target stages will be exceeded in a given year. The median value was calculated using point estimates, while the expected value was calculated using Monte Carlo simulation. The results also show the long-term risk or the probability of a target stage being exceeded over 10-year, 30-year, and 50-year periods. Finally, the model results show the conditional non-exceedance probability or the likelihood that a target stage will not be exceeded by the 10% ACE (10 year), the 4% ACE (25-year), the 2% ACE (50-year), the 1% ACE (100-year), the 0.4% ACE (250-year), and the 0.2% ACE (500-year) events.

Table 59 displays the project performance results for four high damage study area reaches, 11BW79, 11BW5, 1-5, and BL89, which correspond to HEC-FDA model station numbers 64, 58, 82, and 298, under existing (2010) and future (2024, 2035, and 2085) without-project and with-project conditions. Study area reaches 11BW79 an 11BW5 are both located in the northern portion of the city of Houma, study area reach 1-5 is located south and east of the city of Houma, and study area reach BL89 is north and east of the city of Houma and south of Bayou Lafourche. The location of these four high damage study area reaches can be found on the 11 x 17 maps containing the study area reaches in the main report. The project performance information for the remaining 260 study area reaches follows the same logic and format, but is not displayed in the table.

As an example, the target stage for study area reach 11BW79 under existing and future without project conditions shown in the table is based on the joint probability of an annual exceedance event and geotechnical failure since there is a non-Federal levee with geotechnical failure entered into the model. The target stages for the 3% AEP and 1% AEP are shown as the assigned top of Federal levee elevation since geotechnical failure was not entered into the model. Using the year 2035 as an example, the median AEP is 0.1196 (without risk), and the expected AEP is 0.1190 (with risk) and the return interval is 8.4 years under without-project conditions. The median and expected AEP for the 3% AEP alternative is 0.0237 and 0.0256 with a return interval of 39.1 years, respectively. The median and expected AEP for the 1% AEP alternative is 0.004 and 0.072, respectively.

The long term risk is the likelihood that the target stage will be exceeded during a multi-year time window (10, 25, or 50 years). The long term risk of the target stage being exceeded is 71.8 percent for a 10-year period, 95.8 percent for a 30-year period, and 99.8 percent for a 50-year period under without project conditions for 2035.   For the 3% AEP alternative, the long term risk of the target stage being exceeded is 22.9 percent for a 10-year period, 47.7 percent for a 30-year period, and 72.7 percent for a 50-year period. For the 1% AEP alternative, the long term risk of the target stage being exceeded is 7.0 percent for a 10-year period, 16.5 percent for a 30-year period, and 30.3 percent for a 50-year period. The output also shows the assurance or conditional non-exceedance for various probability events. This is the likelihood that a target stage will not be exceeded by a specified event. For this reach, there is a 79.6 percent chance that the stage associated with the 10% ACE (10-year) event will not exceed the target stage, 17.4 percent for the 4% ACE (25-year), 5.0 percent for the 2% ACE (50-year), 2.1 percent for the 1% ACE (100-year), 1.1 percent for the 0.4% (250-year), and 0.007 percent for the 0.2% (500-year) under without project conditions. For the 3% AEP alternative, there is a 99.8-percent for the 10% ACE (10-year), 79.6 percent for the 4% ACE (25-year), 41.3 percent for the 2% ACE (50-year), 20.9 percent for the 1% (100-year), 11.1 percent for the 0.4% ACE (250-year), and 6.3 percent for the 0.2% ACE (500-year). For the 1% AEP alternative, there is a 99.9 percent for the 10% ACE (10-year), 99.8 percent for the 4% ACE (25-year), 91.6 percent for the 2% ACE (50-year), 71.5 percent for the 1% ACE (100-year), 49.8 percent for the 0.4% ACE (250-year), and 33.1 percent for the 0.2% ACE (500-year) events.

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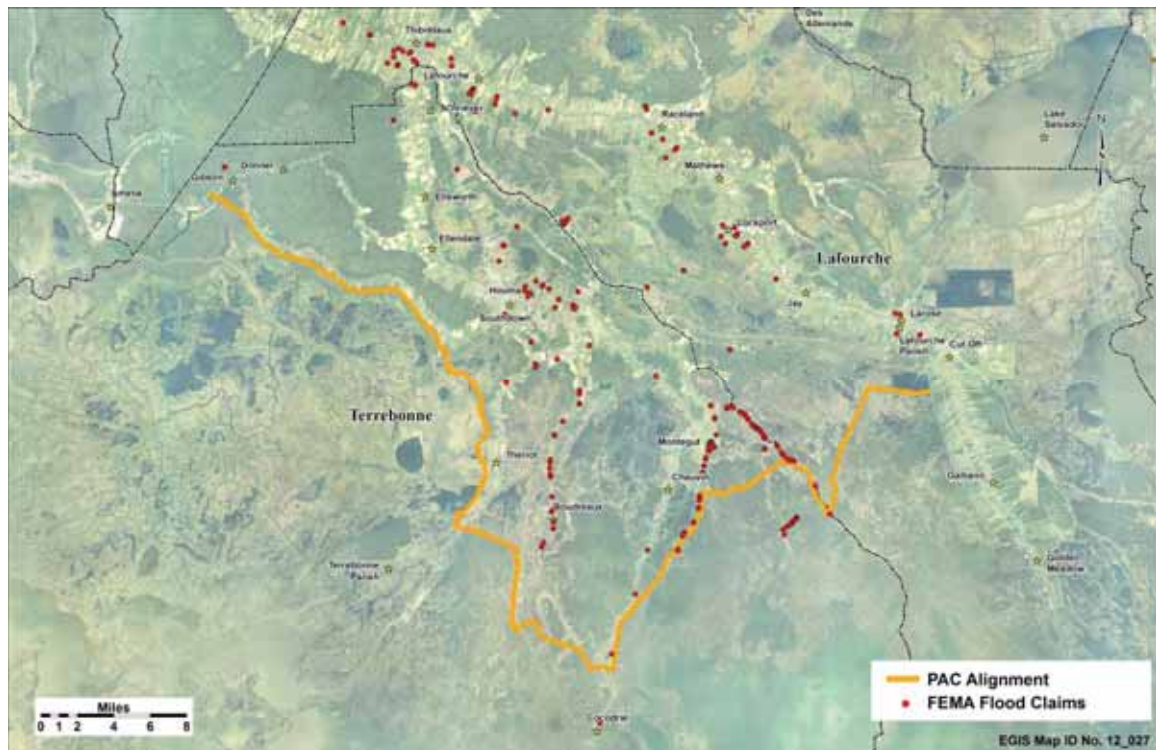
Figure 1

Figure 1. FEMA Repetitive Loss Properties – 1978-2010

Figure 2
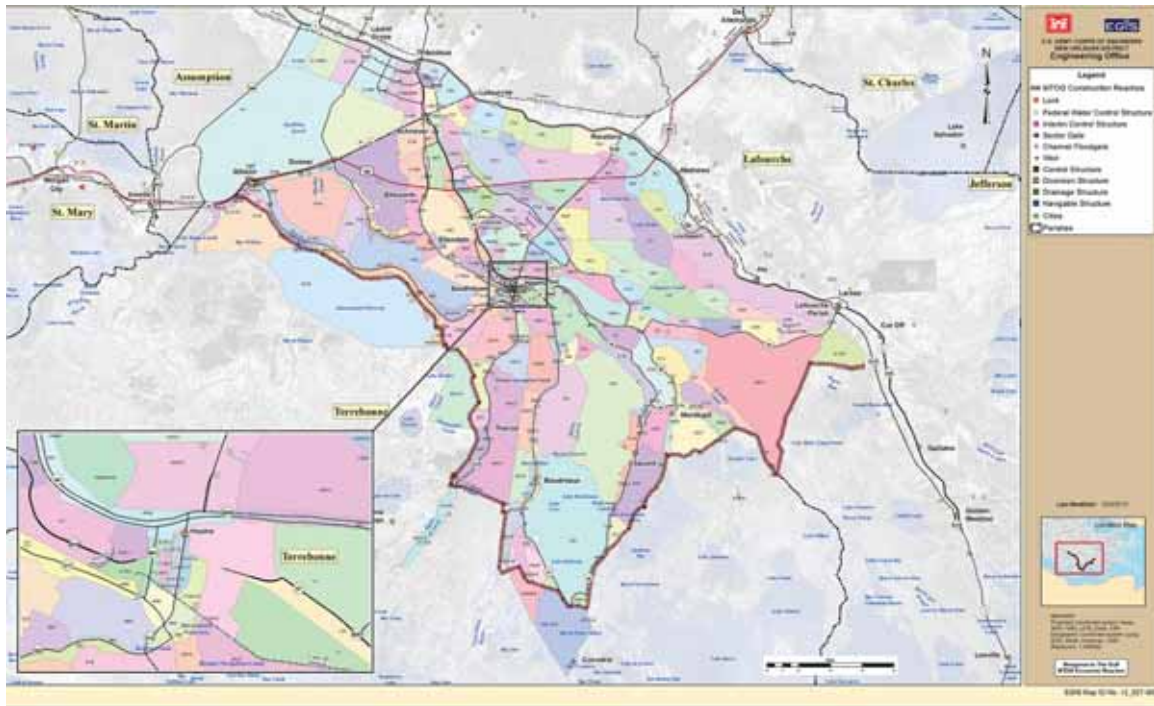
Figure 2. Study Area Reaches and Authorized Alignment

Table 1
Land Use in the Study Area
(2009)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Land Class Name	Acres	Percentage of Total
Developed land	38,798	8.7
Agricultural Land		
Pasture/Hay	46,544	10.5
Sugarcane	20,681	4.6
Fallow/Idle Cropland	8,606	1.9
Soybeans	425	0.1
Rice	1	0.0
Subtotal	76,257	17.1
Undeveloped Land		
Barren/Wetlands	289,737	65.1
Shrubland	1,758	0.4
Grasslands	347	0.1
Forests	41	0.1
Open Space	1,486	0.3
Subtotal	293,369	65.9
Open Water	36,487	8.2
Total	444,911	100.0

Source: National Agricultural Statistical Service

Table 2
Historical and Projected Parish Population
(1,000s)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	69.1	83.5	85.8	90.0	96.3	97.9	104.2
Terrebonne	76.2	95.1	97.0	104.5	112.0	120.9	142.8
Total	145.2	178.6	182.9	194.4	208.3	218.8	247.0

Source: U.S. Census data, Moody's County Forecast Database, and discussions with parish planning officials.

Table 3
Existing Condition and Projected Population
within Inventoried Study Area
(1,000s)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	2010	2035	2085
Lafourche	28.8	29.3	31.2
Terrebonne	104.9	113.2	133.8
Total	133.7	142.5	165.0

Source: Moody's County Forecast Database and discussions with parish planning officials.

Table 4
Number of Households by Parish
(1,000s)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	18.0	25.7	28.8	32.1	33.7	36.3	38.1
Terrebonne	19.6	29.5	31.9	36.0	38.2	43.4	50.4
Total	37.6	55.2	60.7	68.1	71.9	79.7	88.5

Source: U.S. Census data, Moody's County Forecast Database, and discussions with parish planning officials.

Table 5
Per Capita Income
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	1990	2000	2005	2008	2009
Lafourche	$ 13,070	$ 23,039	$ 30,422	$ 42,613	$ 42,205
Terrebonne	$ 13,218	$ 20,991	$ 28,037	$ 39,772	$ 39,049

Source: Bureau of Economic Analysis

Table 6
Total Non-Farm Employment
(1,000s)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	15.1	24.4	22.1	30.4	37.5	40.7	44.2
Terrebonne	24.6	42.4	35.8	47.3	58.9	67.3	81.3
Total	39.7	66.8	57.9	77.7	96.4	108.0	125.5

Source: Based on Moody's County Forecast Database and discussions with parish planning officials.

Table 7
FEMA Flood Claims in Louisiana
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Event	Year	Number of Paid Claims	Total Amount Paid (1,000s)	Average Amount Paid (1,000s)
Tropical Storm Juan	Oct-85	6,187	$ 189,842	$ 30.7
Hurricane Andrew	Aug-92	5,589	$ 270,791	$ 48.5
Tropical Storm Isadore	Sep-02	8,441	$ 141,869	$ 16.8
Hurricane Lili	Oct-02	2,563	$ 46,049	$ 18.0
Hurricane Katrina	Aug-05	167,099	$ 18,556,254	$ 111.0
Hurricane Rita	Sep-05	9,507	$ 539,086	$ 56.7
Hurricane Gustav	Sep-08	4,524	$ 115,250	$ 25.5
Hurricane Ike	Sep-08	46,137	$ 2,712,969	$ 58.8

Source: Federal Emergency Management Agency (FEMA)

Note: Total amount paid and average amount paid have been updated
to the Oct 2011 price level using the CPI for all urban consumers.

Table 8
FEMA Flood Claims by Parish
1978-2011
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Parish	Number of Policies September 2011	Number of Claims
Lafourche	14,222	5,066
Terrebonne	20,044	12,780

Source: FEMA

Table 9
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
1-1AB	1	36	81	120	237	474
1-1AN	4	1,090	415	217	1,925	3,647
11BE1	7	2	199	-	201	402
11BE2	10	159	37	14	217	427
11BE3	13	234	346	35	877	1,492
11BE4	16	163	109	67	272	611
11BE5	19	69	104	44	433	650
11BE6-E	22	-	1	2	1	4
11BE6-W	25	1	125	24	126	276
1-1BU3-U1	28	-	-	-	-	-
1-1BU3-U2	31	-	-	-	-	-
1-1BU3-U3	34	-	-	-	-	-
11BU4	37	-	-	-	-	-
11BW11	40	89	41	38	130	298
11BW2-W1	43	63	19	1	88	171
11BW2-W2	46	368	143	10	772	1,293
11BW4-W3	49	9	12	4	30	55
11BW4-W4	52	658	86	29	1,198	1,971
11BW4-W4A	55	230	3	12	329	574
11BW5	58	1,565	1	54	4,721	6,341
11BW6	61	672	8	81	3,108	3,869
11BW79	64	1,567	35	89	1,996	3,687
11BW79-W7	67	767	67	120	1,916	2,870
1-2MID	70	-	-	62	-	62
1-2N	73	209	34	89	308	640
1-2S	76	-	-	27	-	27
1-3	79	1,003	84	51	1,347	2,485
1-5	82	2,395	315	358	2,710	5,778
1-7_N3-4	85	16	-	2	28	46
1-7_N4-7	88	35	-	3	76	114
1-7_N7-10	91	68	-	3	80	151
1-7-N10-13	94	87	3	7	104	201
1-7N13-16	97	38	4	33	49	124
1-7N16-17	100	-	-	2	-	2

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
1-7N17-24	103	43	1	36	56	136
1-7N24-28	106	217	4	22	296	539
1-8	109	336	44	221	710	1,311
2-1A2	112	-	-	-	-	-
2-1B2-MID	115	6	1	2	7	16
2-1B2N	118	37	2	6	39	84
2-1B2S	121	1,032	19	218	1,211	2,480
3-1B	124	250	31	19	281	581
3-1C	127	72	19	6	91	188
4-1N	130	169	35	12	204	420
4-1S	133	162	88	10	250	510
4-2	136	449	99	10	548	1,106
4-2A	139	323	289	23	612	1,247
4-2B	142	114	112	11	226	463
4-2C	145	98	30	5	128	261
4-7	148	195	29	15	224	463
4MGT	151	192	74	8	315	589
5-1A	154	858	188	40	1,364	2,450
5-1B	157	496	105	37	601	1,239
6-1B1	160	3	-	2	3	8
6-1B1-B	163	2	1	-	3	6
8-1N	166	15	5	3	20	43
8-1N-B	169	39	12	1	51	103
8-1S-B	175	122	42	10	164	338
8-2C	178	-	-	2	-	2
8-2D	181	51	23	3	74	151
9-1AE	184	-	2	-	2	4
9-1AMID	187	-	-	-	-	-
9-1AW	190	-	1	-	1	2
9-1BE	193	4	1	2	5	12
9-1BMIDE	196	1	2	2	3	8
9-1BMIDW	199	-	-	-	-	-
9-1BW	202	3	22	1	25	51
A1	205	29	21	20	50	120
B1	208	12	11	2	23	48
BB1	211	141	1	8	267	417
BB2	214	4	-	10	4	18
BB3	217	16	3	49	39	107

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
BB4	220	6	-	-	6	12
BB5	223	388	-	2	388	778
BB6	226	8	5	3	13	29
BB7	229	120	101	43	221	485
BB8-B	235	-	6	47	21	74
BD1	238	54	18	4	72	148
BDL0	241	14	51	1	65	131
BDL1	244	21	7	5	28	61
BDL2	247	4	-	-	4	8
BDL3	250	82	27	5	109	223
BDL4	253	65	-	3	65	133
BDL4-B	256	53	15	11	68	147
BDL5	259	35	10	19	45	109
BGC0	262	21	76	9	97	203
BGC1	265	23	7	2	30	62
BGC2	268	24	11	3	35	73
BGC3	271	132	49	26	181	388
BGC4	274	49	31	41	80	201
BL1	277	1	10	7	11	29
BL2	280	132	15	35	147	329
BL3	283	66	13	24	79	182
BL4	286	58	33	21	91	203
BL5	289	379	197	125	576	1,277
BL6	292	1,382	397	140	1,839	3,758
BL7	295	1,465	146	225	2,322	4,158
BL89	298	1,897	523	239	3,758	6,417
BPC1	301	339	12	2	351	704
BPC2	304	54	35	7	89	185
BPC3	307	112	58	13	170	353
BPC4	310	55	21	18	76	170
BPC5	313	250	34	9	284	577
BPC5-B	316	198	23	39	221	481
BT1	319	485	45	118	592	1,240
BT10	322	-	-	-	-	-
BT2	325	107	27	3	134	271
BT3	328	17	3	6	20	46
BT4	331	97	68	15	165	345
BT4-SA	334	55	6	3	61	125
BT5	337	10	-	4	10	24
BT5-B	340	10	-	-	10	20
BT6	343	395	25	239	792	1,451

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
BT6A	346	275	62	162	419	918
BT7	349	146	58	69	381	654
BT8	352	16	7	22	23	68
BT9	355	-	-	-	-	-
C1	358	22	9	5	31	67
C1-LF	361	7	1	2	8	18
CC1	364	50	67	7	117	241
D-01	367	21	11	-	32	64
D-06	370	25	9	1	34	69
D10	373	28	12	4	40	84
D-16N	376	37	30	7	67	141
D-16S	379	147	119	8	266	540
D-1732	382	119	86	13	205	423
D1A	385	-	-	-	-	-
D1B	388	-	-	-	-	-
D1b-LF	391	2	1	4	3	10
D1C	394	12	9	10	21	52
D1c-LF1	397	180	108	29	404	721
D1c-LF2	400	150	65	20	215	450
D1c-LF3	403	5	1	4	6	16
D-25	406	116	29	24	154	323
D-25-B	409	-	-	-	-	-
D-26	412	47	2	2	49	100
D-28	415	20	20	4	40	84
D-29	418	1,391	-	50	1,471	2,912
D-30	421	32	2	1	34	69
D-31	424	12	6	3	18	39
D-34N	427	16	-	5	16	37
D-34S	430	4	1	2	5	12
D-35	433	7	-	2	7	16
D-36	436	133	99	6	232	470
D-37	439	62	-	-	62	124
D-38	442	273	-	22	734	1,029
D-39-1	445	300	14	30	314	658
D-39-2	448	66	1	22	274	363
D-39-3	451	184	3	70	329	586
D-42	454	24	29	3	53	109
D-43	457	152	50	12	202	416
D-44	460	3	71	6	94	174
D-45	463	4	-	-	4	8
D-48	466	8	3	-	11	22

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
D-49	469	-	5	-	5	10
D-50	472	30	34	5	142	211
D-51	475	47	1	2	48	98
D-53	478	84	-	5	84	173
D-56	481	65	11	5	76	157
D-60	484	-	370	2	370	742
D-61	487	44	28	1	72	145
D-61-B	490	6	-	-	6	12
D-62-B	496	58	4	2	62	126
D-64	499	93	-	-	93	186
E1	502	2	18	14	20	54
E1-LF	505	-	1	-	1	2
E1-LF-B	508	-	-	8	-	8
E2	511	-	-	1	-	1
E2-B	514	-	-	4	-	4
E2-LF	517	133	72	75	205	485
E2-LF-B	520	-	-	-	-	-
FC	523	-	-	1	-	1
GW10	526	434	4	35	474	947
GW11	529	54	-	14	54	122
GW12	532	977	48	147	2,276	3,448
GW13	535	288	478	64	776	1,606
GW14	538	817	37	114	2,673	3,641
GW14-1	541	32	13	12	45	102
GW15	544	129	145	22	274	570
GW16	547	28	64	7	92	191
GW17	550	-	-	13	-	13
GW18	553	44	-	1	44	89
GW18-B	556	-	1	-	1	2
GW2	559	21	8	1	29	59
GW3	562	21	24	12	45	102
GW4	565	-	4	1	4	9
GW5	568	-	4	-	4	8
GW6	571	-	10	-	10	20
GW7	574	-	4	-	4	8
GW8	577	-	2	-	2	4
GW9	580	24	7	16	31	78
HC1	583	100	120	19	220	459
HC2	586	-	-	2	-	2
HC3	589	28	50	9	78	165
HC4	592	7	-	3	7	17

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
HNC0	595	2	3	75	5	85
HNC1	598	25	10	10	35	80
HNC10	601	14	3	1	17	35
HNC10-B	604	89	26	9	115	239
HNC2	607	129	55	22	184	390
HNC3	610	58	36	13	94	201
HNC4	613	27	8	1	35	71
HNC5	616	60	110	5	170	345
HNC6	619	-	9	51	9	69
HNC7	622	33	9	253	42	337
HNC8	625	60	3	13	63	139
HNC9	628	-	-	-	-	-
HNC9-B	631	142	29	7	171	349
HNC9-E	634	6	9	-	15	30
HNC9-W	637	7	4	6	11	28
LB1	640	-	-	-	-	-
LB2	643	9	15	7	24	55
LB3	646	-	-	3	-	3
LB4	649	31	264	17	295	607
LB5	652	30	19	12	49	110
LBB2	655	3	-	2	3	8
LBB3	658	51	9	7	60	127
LBB4	661	99	3	139	105	346
LBB5	664	610	-	28	610	1,248
LBB6	667	88	-	35	88	211
LBC1	670	-	-	2	-	2
LBC2	673	-	-	3	-	3
LF1	676	24	-	11	24	59
LF2	679	13	1	4	14	32
LF-GB	682	-	5	9	5	19
LL1	685	3	-	-	3	6
LL2	688	-	-	-	-	-
LL3	691	-	1	-	1	2
MC1	694	-	-	-	-	-
OB1	697	-	-	-	-	-
OB2	700	40	74	5	114	233
OB3	703	18	12	10	150	190
OB4	706	55	-	2	55	112
PAC1	709	3	2	7	5	17
SL1	712	54	55	10	109	228
SL2	715	20	-	2	20	42

Table 9 (Cont.)
Number of Structures per Reach in the Existing Condition
(2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	HEC-FDA Station Number	Residential	Mobile Home	Non-Residential	Vehicle	Total
SL3	718	140	54	8	194	396
TS1	721	-	-	-	-	-
TS10	724	-	-	-	-	-
TS11	727	77	-	18	77	172
TS12	730	19	25	23	44	111
TS13	733	13	6	4	19	42
TS14	736	-	3	-	3	6
TS15	739	-	-	-	-	-
TS16	742	123	177	5	300	605
TS17	745	30	8	2	38	78
TS18	748	12	-	-	12	24
TS19	751	401	195	31	750	1,377
TS2	754	-	-	-	-	-
TS20	757	1	-	-	1	2
TS21	760	-	-	-	-	-
TS22	763	228	205	46	433	912
TS3	766	-	-	-	-	-
TS4	769	37	3	14	40	94
TS5	772	82	57	39	139	317
TS6	775	226	48	56	274	604
TS7	778	-	-	-	-	-
TS9	781	80	57	20	137	294
US1	784	-	-	2	-	2
GW11-B	787	-	-	-	-	-
E1-B	790	-	-	-	-	-
BB7-B	793	-	-	-	-	-
BD1-B	796	-	-	-	-	-
BC	799	-	-	-	-	-
Total		36,681	9,858	6,227	64,365	117,131

Note: Industrial Structures were modeled as a separate category and therefore are not included in the above structure inventory.

Table 10
Residential and Non-Residential Structure Inventory
Existing Conditions (2010)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Structure Category	Number	Average Depreciated Replacement Value
Residential		
One-Story Slab	21,693	$ 168,000
One-Story Pier	12,717	$ 92,000
Two-Story Slab	1,656	$ 232,000
Two-Story Pier	615	$ 148,000
Mobile Home	9,858	$ 10,000
Total Residential	**46,539**	
Non-Residential		
Eating and Recreation	297	$ 348,000
Professional	1,167	$ 555,000
Public and Semi-Public	642	$ 813,000
Repair and Home Use	148	$ 175,000
Retail and Personal Services	586	$ 572,000
Warehouse	2,932	$ 181,000
Grocery and Gas Station	146	$ 359,000
Multi-Family Occupancy	309	$ 431,000
Industrial	24	$ 1,854,000
Total Non-Residential	**6,251**	

Source: Based on *Morganza to the Gulf Post Authorization Change Report: Residential and Non-residential Structure Inventory and Nonresidential Surveys Final Report* dated May 2009

Table 11
Number of Projected Residential and Non-Residential Structures
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Future Conditions (2010-2035)	
Structure Category	Number
Residential	
One-Story Slab	3,522
One-Story Pier	1,924
Two-Story Slab	204
Two-Story Pier	91
Mobile Home	1,579
Total Residential	**7,320**
Non-Residential	
Eating and Recreation	137
Professional	286
Public and Semi-Public	92
Repair and Home Use	32
Retail and Personal Services	122
Warehouse	620
Grocery and Gas Station	30
Multi-Family Occupancy	0
Industrial	0
Total Non-Residential	**1,319**
Future Conditions (2035-2085)	
Structure Category	Number
Residential	
One-Story Slab	4,344
One-Story Pier	2,328
Two-Story Slab	263
Two-Story Pier	111
Mobile Home	1,866
Total Residential	**8,912**
Non-Residential	
Eating and Recreation	537
Professional	484
Public and Semi-Public	91
Repair and Home Use	66
Retail and Personal Services	251
Warehouse	1,850
Grocery and Gas Station	63
Multi-Family Occupancy	0
Industrial	0
Total Non-Residential	**3,342**

Source: Based on *Projections of Future Development and Land Usage Morganza to the Gulf Feasibility Evaluation Final Report* dated February 2011

Table 12

Content-to-Structure Value Ratios (CSVRs) and Standard Deviations (SDs)
by Structure Category
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Structure Category		(CSVR, SD)
Residential	One-story	(0.71, 0.23)
	Two-story	(0.50, 0.27)
	Mobile home	(1.48, 0.68)
Non-Residential	Eating and Recreation	(3.05, 4.48)
	Groceries and Gas Stations	(1.28, 0.96)
	Professional Buildings	(0.78, 0.70)
	Public and Semi-Public Buildings	(0.81, 1.03)
	Multi-Family Buildings	(0.23, 0.13)
	Repair and Home Use	(2.51, 2.08)
	Retail and Personal Services	(1.48, 1.13)
	Warehouses and Contractor Services	(3.73, 4.81)

Table 13
Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Category Name	Damage Type	Parameter	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
1STY-PIER	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	1.1	12.2	15.2	49.4	50.1	66.7	70.2	71.2	97.5	97.5	97.5	97.5	97.5	97.5	97.5	97.5	97.5	97.5	97.5
			Lower %	0.0	1.0	11.9	13.7	44.4	45.1	60.0	63.2	64.1	87.7	87.7	87.7	87.7	87.7	87.7	87.7	87.7	87.7	87.7	87.7
			Upper %	0.0	1.7	18.3	22.8	74.0	75.1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	0.0	95.0	95.0	95.0	95.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0
			Lower %	0.0	0.0	0.0	0.0	90.0	90.0	95.0	95.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0
			Upper %	0.0	0.0	0.0	0.0	98.0	98.0	98.0	98.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
1STY-SLAB	Residential	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	1.1	1.0	23.3	23.3	37.2	41.9	45.3	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0
			Lower %	0.0	1.0	1.0	21.0	21.0	35.5	37.7	40.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8	82.8
			Upper %	0.0	1.7	1.7	35.0	35.0	55.9	62.9	68.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	0.0	0.0	95.0	95.0	95.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0
			Lower %	0.0	0.0	0.0	0.0	0.0	90.0	95.0	95.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0	98.0
			Upper %	0.0	0.0	0.0	0.0	0.0	98.0	98.0	98.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Struct	N	N	0.3	N		12.2	N		71.0	23.0			-901.0							
2STY-PIER	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	1.4	2.2	6.4	19.0	19.0	31.9	32.6	33.3	93.4	93.4	93.4	93.4	93.4	93.4	93.6	93.6	93.6	93.6	93.6
			Lower %	0.0	1.2	2.0	5.8	17.1	17.1	28.7	29.3	30.0	84.0	84.0	84.0	84.0	84.0	84.0	84.0	84.0	84.0	84.0	84.0
			Upper %	0.0	2.1	3.3	9.6	28.5	28.5	47.9	48.9	49.9	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	0.0	69.6	74.7	74.7	78.5	79.9	83.2	83.2	83.2	83.2	83.2	83.2	97.5	97.8	98.5	98.5	98.5
			Lower %	0.0	0.0	0.0	0.0	66.2	70.9	70.9	74.6	75.9	79.0	79.0	79.0	79.0	79.0	79.0	92.6	92.9	93.6	93.6	93.6
			Upper %	0.0	0.0	0.0	0.0	73.1	78.4	78.4	82.5	83.9	87.3	87.3	87.3	87.3	87.3	87.3	100.0	100.0	100.0	100.0	100.0
2STY-SLAB	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	1.2	1.2	16.1	16.1	26.1	27.1	28.5	80.0	80.0	80.0	80.0	80.0	80.0	80.3	80.3	80.3	83.2	83.2	83.2
			Lower %	0.0	1.1	1.1	14.5	14.5	23.5	24.4	25.7	72.0	72.0	72.0	72.0	72.0	72.0	72.0	72.0	72.0	72.0	72.0	72.0
			Upper %	0.0	1.8	1.8	24.2	24.2	39.1	40.7	42.8	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	69.9	74.7	74.7	78.5	79.9	83.2	83.2	83.2	83.2	83.2	83.2	97.5	97.8	98.5	98.5	98.5	98.5
			Lower %	0.0	0.0	0.0	66.2	70.9	70.9	74.6	75.9	79.0	79.0	79.0	79.0	79.0	79.0	92.6	92.9	93.6	93.6	93.6	93.6
			Upper %	0.0	0.0	0.0	73.1	78.4	78.4	82.5	83.9	87.3	87.3	87.3	87.3	87.3	87.3	100.0	100.0	100.0	100.0	100.0	100.0
AUTO	AUTO	Structure	Stage	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0	17.0
			Mean %	0.0	0.0	3.7	13.0	46.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	2.3	12.0	44.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Upper %	0.0	0.0	4.7	15.0	45.3	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 13 (Cont)
Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Category Name	Damage Type	Parameter	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
EAT	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	6.6	19.8	19.8	24.5	24.5	29.6	34.7	37.9	37.9	37.9	63.3	63.3	63.3	63.3	63.3	63.3	63.3	63.3
			Lower %	0.0	0.0	6.2	18.4	18.4	22.8	22.8	26.6	31.2	34.1	34.1	34.1	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
			Upper %	0.0	0.0	7.6	22.8	22.8	28.2	28.2	37.0	43.4	47.4	47.4	47.4	79.2	79.2	79.2	79.2	79.2	79.2	79.2	79.2
		Contents	Mean %	0.0	0.0	0.0	41.2	45.6	73.3	74.8	92.4	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	0.0	39.2	43.3	69.6	71.1	87.8	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0
			Upper %	0.0	0.0	0.0	51.5	57.0	91.6	93.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
GROC	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	6.6	19.8	19.8	24.5	24.5	29.6	34.7	37.9	37.9	37.9	63.3	63.3	63.3	63.3	63.3	63.3	63.3	63.3
			Lower %	0.0	0.0	6.2	18.4	18.4	22.8	22.8	26.6	31.2	34.1	34.1	34.1	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
			Upper %	0.0	0.0	7.6	22.8	22.8	28.2	28.2	37.0	43.4	47.4	47.4	47.4	79.2	79.2	79.2	79.2	79.2	79.2	79.2	79.2
		Contents	Mean %	0.0	0.0	0.0	99.1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	0.0	94.1	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0
			Upper %	0.0	0.0	0.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
MOBHOM	MOBHOME	Structure	Stage	-1.1	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	6.4	7.3	9.9	43.4	44.7	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6	97.6
			Lower %	0.0	6.1	6.9	9.4	41.2	42.5	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7	92.7
			Upper %	0.0	8.6	9.8	13.4	58.6	60.3	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	90.0	95.0	96.0	97.0	98.0	99.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	90.0	92.0	94.0	96.0	98.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Upper %	0.0	0.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
MULT	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	6.6	19.8	19.8	24.5	24.5	29.6	34.7	37.9	37.9	37.9	63.3	63.3	63.3	63.3	63.3	63.3	63.3	63.3
			Lower %	0.0	0.0	6.2	18.4	18.4	22.8	22.8	26.6	31.2	34.1	34.1	34.1	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
			Upper %	0.0	0.0	7.6	22.8	22.8	28.2	28.2	37.0	43.4	47.4	47.4	47.4	79.2	79.2	79.2	79.2	79.2	79.2	79.2	79.2
		Contents	Mean %	0.0	0.0	20.1	26.2	33.5	42.4	49.8	50.3	51.7	51.7	51.7	51.7	51.7	51.7	71.8	85.2	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	15.8	22.4	31.2	40.5	46.6	50.3	50.3	50.3	50.3	50.3	50.3	50.3	56.4	79.6	93.5	97.1	97.1	97.1
			Upper %	0.0	0.0	22.2	28.7	35.2	46.2	51.4	53.0	53.1	54.6	54.6	54.6	54.6	54.6	79.3	89.5	100.0	100.0	100.0	100.0
PROF	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	6.6	19.8	19.8	24.5	24.5	29.6	34.7	37.9	37.9	37.9	63.3	63.3	63.3	63.3	63.3	63.3	63.3	63.3
			Lower %	0.0	0.0	6.2	18.4	18.4	22.8	22.8	26.6	31.2	34.1	34.1	34.1	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
			Upper %	0.0	0.0	7.6	22.8	22.8	28.2	28.2	37.0	43.4	47.4	47.4	47.4	79.2	79.2	79.2	79.2	79.2	79.2	79.2	79.2
		Contents	Mean %	0.0	0.0	35.0	43.3	56.7	63.9	85.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	30.0	37.1	48.6	54.8	85.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Upper %	0.0	0.0	50.0	61.8	81.0	91.3	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 13 (Cont)
Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
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				-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
PUBL	COM		Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Structure	Mean %	0.0	0.0	1.1	22.3	23.7	25.8	32.7	34.4	79.1	79.1	79.1	79.1	79.1	79.1	79.1	79.1	80.5	80.5	80.5	80.5
			Lower %	0.0	0.0	1.1	20.8	22.1	24.0	29.5	31.0	71.2	71.2	71.2	71.2	71.2	71.2	71.2	71.2	72.4	72.4	72.4	72.4
			Upper %	0.0	0.0	1.3	25.7	27.3	29.7	39.3	43.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	80.0	85.0	85.7	86.6	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	0.0	60.0	63.8	64.3	65.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0
			Upper %	0.0	0.0	0.0	88.0	93.5	94.2	95.3	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
REPA	COM		Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Structure	Mean %	0.0	0.0	1.1	22.3	23.7	25.8	32.7	34.4	79.1	79.1	79.1	79.1	79.1	79.1	79.1	79.1	80.5	80.5	80.5	80.5
			Lower %	0.0	0.0	1.1	20.8	22.1	24.0	29.5	31.0	71.2	71.2	71.2	71.2	71.2	71.2	71.2	71.2	72.4	72.4	72.4	72.4
			Upper %	0.0	0.0	1.3	25.7	27.3	29.7	39.3	43.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	33.3	34.3	34.3	69.2	70.6	72.1	80.6	83.7	83.7	83.7	83.7	83.7	83.7	83.7	83.7	83.7	83.7
			Lower %	0.0	0.0	0.0	31.7	32.6	32.6	65.7	67.1	68.5	76.6	79.6	79.6	79.6	79.6	79.6	79.6	79.6	79.6	79.6	79.6
			Upper %	0.0	0.0	0.0	41.7	42.9	42.9	86.5	88.3	90.2	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
RETA	COM		Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Structure	Mean %	0.0	0.0	1.1	22.3	23.7	25.8	32.7	34.4	79.1	79.1	79.1	79.1	79.1	79.1	79.1	79.1	80.5	80.5	80.5	80.5
			Lower %	0.0	0.0	1.1	20.8	22.1	24.0	29.5	31.0	71.2	71.2	71.2	71.2	71.2	71.2	71.2	71.2	72.4	72.4	72.4	72.4
			Upper %	0.0	0.0	1.3	25.7	27.3	29.7	39.3	43.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	36.6	60.5	60.5	75.4	85.1	94.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	0.0	34.8	57.5	57.5	71.6	80.8	89.7	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0	95.0
			Upper %	0.0	0.0	0.0	45.7	75.7	75.7	94.2	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
WARE	COM		Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
		Structure	Mean %	0.0	0.0	1.1	22.3	23.7	25.8	32.7	34.4	79.1	79.1	79.1	79.1	79.1	79.1	79.1	79.1	80.5	80.5	80.5	80.5
			Lower %	0.0	0.0	1.1	20.8	22.1	24.0	29.5	31.0	71.2	71.2	71.2	71.2	71.2	71.2	71.2	71.2	72.4	72.4	72.4	72.4
			Upper %	0.0	0.0	1.3	25.7	27.3	29.7	39.3	43.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Mean %	0.0	0.0	0.0	17.6	22.1	22.1	29.2	34.0	42.8	50.8	58.7	66.7	74.6	79.7	79.7	79.7	79.7	79.7	79.7	79.7
			Lower %	0.0	0.0	0.0	16.8	21.0	21.0	27.8	32.3	40.7	48.3	55.8	63.4	70.9	75.7	75.7	75.7	75.7	75.7	75.7	75.7
			Upper %	0.0	0.0	0.0	22.0	27.7	27.7	36.6	42.5	53.6	63.5	73.4	83.4	93.3	99.6	99.6	99.6	99.6	99.6	99.6	99.6

Source: Based on Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios (CSVRs) in Support of the Lower Atchafalaya and Morganza to the Gulf, Louisiana, Feasibility Study Final Report dated May 1997

Table 14
Non-Federal Levee Fragility Curve
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

| Reach Name | Station | Stage (ft.) associated w/Probability of Failure | | | | Top of Levee |
		0%	10%	45%	95%	(ft.)
1-1AB	1	2.0	3.8	4.4	4.7	5.0
1-1AN	4	2.0	3.8	4.4	4.7	5.0
11BE4	16	2.0	4.5	5.3	5.6	6.0
11BE5	19	2.0	3.0	3.5	3.7	4.0
11BE6-W	25	2.0	4.5	5.3	5.6	6.0
11BW11	40	2.0	2.3	2.6	2.8	3.0
11BW5	58	2.0	4.1	4.8	5.1	5.5
11BW6	61	2.0	4.1	4.8	5.1	5.5
11BW79	64	2.0	4.5	5.3	5.6	6.0
11BW79-W7	67	2.0	4.1	4.8	5.1	5.5
1-2S	76	2.0	3.0	3.5	3.7	4.0
1-3	79	2.0	4.9	5.7	6.0	6.5
1-5	82	2.0	2.3	2.6	2.8	3.0
1-7_N3-4	85	2.0	4.1	4.8	5.1	5.5
1-7_N4-7	88	2.0	4.1	4.8	5.1	5.5
1-7_N7-10	91	2.0	4.1	4.8	5.1	5.5
1-7-N10-13	94	2.0	4.1	4.8	5.1	5.5
1-7N13-16	97	2.0	4.1	4.8	5.1	5.5
1-7N16-17	100	2.0	4.1	4.8	5.1	5.5
1-7N17-24	103	2.0	4.1	4.8	5.1	5.5
1-7N24-28	106	2.0	4.1	4.8	5.1	5.5
3-1B	124	2.0	7.1	8.4	8.8	9.5
3-1C	127	2.0	4.5	5.3	5.6	6.0
4-1N	130	2.0	3.0	3.5	3.7	4.0
4-1S	133	2.0	5.3	6.2	6.5	7.0
4-2	136	2.0	3.0	3.5	3.7	4.0
4-2A	139	2.0	4.5	5.3	5.6	6.0
4-2B	142	2.0	4.5	5.3	5.6	6.0
4-2C	145	2.0	4.5	5.3	5.6	6.0
4-7	148	2.0	4.5	5.3	5.6	6.0
4MGT	151	2.0	4.5	5.3	5.6	6.0
5-1A	154	2.0	4.5	5.3	5.6	6.0
5-1B	157	2.0	4.5	5.3	5.6	6.0
6-1B1	160	2.0	4.5	5.3	5.6	6.0
6-1B1-B	163	2.0	4.5	5.3	5.6	6.0
8-1N	166	2.0	3.0	3.5	3.7	4.0

Table 14 (Cont.)
Non-Federal Levee Fragility Curve
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Reach Name	Station	Stage (ft.) associated w/Probability of Failure				Top of Levee (ft.)
		0%	10%	45%	95%	
8-1N-B	169.0	2.0	3.0	3.5	3.7	4.0
8-1S-B	175.0	2.0	3.0	3.5	3.7	4.0
8-2C	178.0	2.0	4.5	5.3	5.6	6.0
8-2D	181.0	2.0	4.5	5.3	5.6	6.0
9-1AE	184.0	2.0	6.0	7.0	7.4	8.0
9-1AMID	187.0	2.0	6.0	7.0	7.4	8.0
9-1AW	190.0	2.0	6.0	7.0	7.4	8.0
9-1BMIDE	196.0	2.0	6.0	7.0	7.4	8.0
9-1BMIDW	199.0	2.0	6.0	7.0	7.4	8.0
9-1BW	202.0	2.0	6.0	7.0	7.4	8.0
BL2	280.0	2.0	4.5	5.3	5.6	6.0
BL3	283.0	2.0	4.5	5.3	5.6	6.0
BL4	286.0	2.0	3.8	4.4	4.7	5.0
BL5	289.0	2.0	3.8	4.4	4.7	5.0
BL6	292.0	2.0	3.8	4.4	4.7	5.0
BL7	295.0	2.0	4.5	5.3	5.6	6.0
BL89	298.0	2.0	3.8	4.4	4.7	5.0
BPC3	307.0	2.0	4.5	5.3	5.6	6.0
BPC4	310.0	2.0	4.5	5.3	5.6	6.0
BT4	331.0	2.0	4.5	5.3	5.6	6.0
BT4-SA	334.0	2.0	5.3	6.2	6.5	7.0
D-01	367.0	2.0	7.5	8.8	9.3	10.0
D10	373.0	2.0	4.5	5.3	5.6	6.0
D-16S	379.0	2.0	3.0	3.5	3.7	4.0
D-25	406.0	2.0	5.3	6.2	6.5	7.0
D-29	418.0	2.0	4.9	5.7	6.0	6.5
D-30	421.0	2.0	3.0	3.5	3.7	4.0
D-36	436.0	2.0	7.1	8.4	8.8	9.5
D-48	466.0	2.0	3.0	3.5	3.7	4.0
D-53	478.0	2.0	3.8	4.4	4.7	5.0
D-56	481.0	2.0	4.5	5.3	5.6	6.0
D-60	484.0	2.0	4.5	5.3	5.6	6.0
D-61	487.0	2.0	4.5	5.3	5.6	6.0
D-61-B	490.0	2.0	4.5	5.3	5.6	6.0
D-62-B	496.0	2.0	4.5	5.3	5.6	6.0
D-64	499.0	2.0	3.8	4.4	4.7	5.0
E2-LF	517.0	2.0	4.0	4.7	5.0	5.4
E2-LF-B	520.0	2.0	4.0	4.7	5.0	5.4
LBC1	670.0	2.0	4.5	5.3	5.6	6.0
LBC2	673.0	2.0	4.5	5.3	5.6	6.0
PAC1	709.0	2.0	7.5	8.8	9.3	10.0
SL3	718.0	2.0	7.5	8.8	9.3	10.0

Table 15
Still Water Stage Associated with Federal Levee Failure by Levee Reach
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Federal Levee Reach	Levee Failure Still Water Stage (ft.)					
	2024		2035		2085	
	3% AEP	1% AEP	3% AEP	1% AEP	3% AEP	1% AEP
A	9.4	13.6	10.5	15.9	11.2	18.3
B	11.1	15.5	10.8	16.5	11.5	17.7
E	13.2	13.3	14.2	19.8	13.5	20.8
F	13.0	13.2	13.3	20.6	13.5	20.8
G	12.9	14.5	13.1	19.5	13.6	19.6
H	14.8	17.2	16.2	20.5	15.8	21.8
I	14.9	18.2	15.1	20.5	15.8	21.8
J	15.3	18.5	15.5	20.9	15.8	21.8
K	14.0	17.8	15.1	21.0	14.4	21.8
L	14.7	17.3	15.1	20.3	14.4	21.8

Note: The Federal levee heights associated with failure of the 3% AEP do not uniformally rise across the selected years due to the estimated settlement that occurs relative to the levee lift schedule.

Table 16
Expected Annual Damages (1,000's)
Structures, Contents, and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Analysis Year	Unadjusted Without-Project Damages	Percent Increase from 2010
2010	$ 515,000	
2024	$ 591,000	15
2035	$ 726,000	41
2085	$ 1,462,000	184

Note: Without-project damages before adjusting the structure inventories for repetitive flood losses after the year 2010.

Table 17
Number of Structures Receiving Damages By Probability Event in 2035
Residential, Commercial, and Mobile Homes
Unadjusted Without-Project Condition
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Annual Chance Exceedance Event (ACE)	Residential	Non-Residential	Mobile Home	Total
0.99 (1 yr)	1,114	371	211	1,696
0.20 (5 yr)	1,905	586	400	2,891
0.10 (10 yr)	5,240	1,117	1,178	7,535
0.04 (25 yr)	26,442	3,848	6,603	36,893
0.02 (50 yr)	35,072	6,054	9,185	50,311
0.01 (100 yr)	41,801	7,562	11,252	60,615
0.005 (200 yr)	42,147	7,591	11,428	61,166
0.002 (500 yr)	42,356	7,594	11,437	61,387

Note: The table reflects the number of structures damaged by ACE event before adjustments were made to the structure inventory for repetitive flooding. In contrast, Table 55 shows the number of structures damaged by ACE event in Table 55 after the adjustments have been made for repetitive flooding. It should be noted that this table uses damages below their first floor elevation as a criteria for being damaged by an ACE event.

Table 18

Number of Structures Receiving 50% or Greater Damages By
Probability Event in 2035
Residential and Mobile Homes
Without-Project Condition
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Annual Chance Exceedance (ACE) Event	Residential Structures Receiving Greater Than 50% Damage
0.99 (1 yr)	95
0.20 (5 yr)	341
0.10 (10 yr)	1,702
0.04 (25 yr)	17,316
0.02 (50 yr)	30,830
0.01 (100 yr)	34,045
0.005 (200 yr)	40,692
0.002 (500 yr)	41,460

Notes: Calculations do not include performance of non-Federal levees.

Calculations include mobile homes.

Calculations are based on 50% damage to structure value not including damage to contents.

Records containing multiple structures were only counted once.

Table 19
Expected Annual Damages and Benefits (1000's)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

2010						
	Expected Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
Plan Name	Total Without-Project	Total With-Project	Damages Reduced	0.75	0.50	0.25
Without	$ 485,766	$ -	$ -	$ -	$ -	$ -
2024						
	Expected Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
Plan Name	Total Without-Project	Total With-Project	Damages Reduced	0.75	0.50	0.25
Without	$ 538,070	$ 538,070	$ -	$ -	$ -	$ -
Alt 3%	$ 538,070	$ 299,936	$ 238,134	$ 229,960	$ 304,649	$ 386,288
Alt 1%	$ 538,070	$ 158,762	$ 379,308	$ 339,305	$ 531,108	$ 709,097
2035						
	Expected Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
Plan Name	Total Without-Project	Total With-Project	Damages Reduced	0.75	0.50	0.25
Without	$ 642,259	$ 642,259	$ -	$ -	$ -	$ -
Alt 3%	$ 642,259	$ 334,582	$ 307,677	$ 229,960	$ 304,649	$ 386,288
Alt 1%	$ 642,259	$ 111,291	$ 530,968	$ 339,305	$ 531,108	$ 709,097
2085						
	Expected Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
Plan Name	Total Without-Project	Total With-Project	Damages Reduced	0.75	0.50	0.25
Without	$ 1,156,553	$ 1,156,553	$ -	$ -	$ -	$ -
Alt 3%	$ 1,156,553	$ 553,116	$ 603,437	$ 464,706	$ 606,124	$ 743,005
Alt 1%	$ 1,156,553	$ 146,304	$ 1,010,249	$ 687,360	$ 1,003,295	$ 1,317,193

Note : Damage values based on HEC-FDA model executions for structures, their contents, and vehicles only.

Table 20
3% AEP Calculation of Equivalent Annual Damages and Benefits (1000's)
Base Year 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Year	Years from Base Year	PV Factor	Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Present Value (PV) Expected Annual Damages/Benefits		
					Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Expected Annual Benefits
2010-12	-22	2.370	$ 485,766	-	$ -	$ -	
2013	-21	2.279	$ -	-	$ -	$ -	
2014	-20	2.191	-	-	-	-	
2015	-19	2.107	-	-	-	-	
2016	-18	2.026	-	-	-	-	
2017	-17	1.948	-	-	-	-	
2018	-16	1.873	-	-	-	-	
2019	-15	1.801	-	-	-	-	
2020	-14	1.732	-	-	-	-	
2021	-13	1.665	-	-	-	-	
2022	-12	1.601	-	-	-	-	
2023	-11	1.539	-	-	-	-	
2024	-10	1.480	$ 538,070	299,936	$ 777,535	$ 433,420	$ 344,115
2025	-9	1.423	$ 547,542	309,407	$ 762,624	$ 430,947	$ 331,677
2026	-8	1.369	$ 557,014	249,336	$ 747,774	$ 334,727	$ 413,048
2027	-7	1.316	$ 566,485	258,808	$ 733,002	$ 334,884	$ 398,118
2028	-6	1.265	$ 575,957	268,280	$ 718,321	$ 334,593	$ 383,729
2029	-5	1.217	$ 585,429	277,751	$ 703,744	$ 333,885	$ 369,859
2030	-4	1.170	$ 594,900	287,223	$ 689,282	$ 332,791	$ 356,490
2031	-3	1.125	$ 604,372	296,695	$ 674,945	$ 331,340	$ 343,605
2032	-2	1.082	$ 613,844	306,166	$ 660,745	$ 329,559	$ 331,186
2033	-1	1.040	$ 623,315	315,638	$ 646,690	$ 327,475	$ 319,215
2034	0	1.000	$ 632,787	325,110	$ 632,787	$ 325,110	$ 307,677
2035	1	0.962	$ 642,259	334,582	$ 619,045	$ 322,488	$ 296,556
2036	2	0.925	$ 652,545	338,952	$ 606,225	$ 314,892	$ 291,333
2037	3	0.889	$ 662,831	343,323	$ 593,524	$ 307,424	$ 286,099
2038	4	0.855	$ 673,116	347,694	$ 580,949	$ 300,085	$ 280,864
2039	5	0.822	$ 683,402	352,064	$ 568,507	$ 292,874	$ 275,633
2040	6	0.790	$ 693,688	356,435	$ 556,206	$ 285,793	$ 270,413
2041	7	0.760	$ 703,974	360,806	$ 544,051	$ 278,841	$ 265,211
2042	8	0.731	$ 714,260	365,176	$ 532,049	$ 272,018	$ 260,031
2043	9	0.703	$ 724,546	369,547	$ 520,203	$ 265,324	$ 254,879
2044	10	0.676	$ 734,832	373,918	$ 508,519	$ 258,759	$ 249,760
2045	11	0.650	$ 745,118	378,288	$ 496,999	$ 252,321	$ 244,678
2046	12	0.625	$ 755,404	382,659	$ 485,648	$ 246,011	$ 239,637
2047	13	0.601	$ 765,690	387,030	$ 474,468	$ 239,827	$ 234,641
2048	14	0.577	$ 775,975	391,400	$ 463,462	$ 233,769	$ 229,693
2049	15	0.555	$ 786,261	395,771	$ 452,632	$ 227,836	$ 224,796
2050	16	0.534	$ 796,547	400,142	$ 441,979	$ 222,026	$ 219,953
2051	17	0.513	$ 806,833	404,512	$ 431,505	$ 216,339	$ 215,166
2052	18	0.494	$ 817,119	408,883	$ 421,211	$ 210,772	$ 210,439

Table 20 (Cont.)
3% AEP Calculation of Equivalent Annual Damages and Benefits (1000's)
Base Year 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Year	Years from Base Year	PV Factor	Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Present Value (PV) Expected Annual Damages/Benefits		
					Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Expected Annual Benefits
2053	19	0.475	$ 827,405	$ 413,254	$ 411,097	$ 205,325	$ 205,771
2054	20	0.456	$ 837,691	$ 417,625	$ 401,164	$ 199,997	$ 201,167
2055	21	0.439	$ 847,977	$ 421,995	$ 391,412	$ 194,786	$ 196,626
2056	22	0.422	$ 858,263	$ 426,366	$ 381,840	$ 189,690	$ 192,150
2057	23	0.406	$ 868,548	$ 430,737	$ 372,450	$ 184,708	$ 187,742
2058	24	0.390	$ 878,834	$ 435,107	$ 363,239	$ 179,838	$ 183,401
2059	25	0.375	$ 889,120	$ 439,478	$ 354,208	$ 175,079	$ 179,128
2060	26	0.361	$ 899,406	$ 443,849	$ 345,354	$ 170,429	$ 174,925
2061	27	0.347	$ 909,692	$ 448,219	$ 336,679	$ 165,887	$ 170,792
2062	28	0.333	$ 919,978	$ 452,590	$ 328,179	$ 161,450	$ 166,729
2063	29	0.321	$ 930,264	$ 456,961	$ 319,853	$ 157,117	$ 162,736
2064	30	0.308	$ 940,550	$ 461,331	$ 311,701	$ 152,887	$ 158,815
2065	31	0.296	$ 950,836	$ 465,702	$ 303,721	$ 148,757	$ 154,964
2066	32	0.285	$ 961,121	$ 470,073	$ 295,909	$ 144,726	$ 151,184
2067	33	0.274	$ 971,407	$ 474,443	$ 288,266	$ 140,792	$ 147,475
2068	34	0.264	$ 981,693	$ 478,814	$ 280,789	$ 136,953	$ 143,836
2069	35	0.253	$ 991,979	$ 483,185	$ 273,476	$ 133,208	$ 140,268
2070	36	0.244	$ 1,002,265	$ 487,555	$ 266,324	$ 129,554	$ 136,770
2071	37	0.234	$ 1,012,551	$ 491,926	$ 259,333	$ 125,991	$ 133,341
2072	38	0.225	$ 1,022,837	$ 496,297	$ 252,498	$ 122,516	$ 129,982
2073	39	0.217	$ 1,033,123	$ 500,668	$ 245,819	$ 119,128	$ 126,691
2074	40	0.208	$ 1,043,409	$ 505,038	$ 239,293	$ 115,824	$ 123,469
2075	41	0.200	$ 1,053,694	$ 509,409	$ 232,918	$ 112,604	$ 120,314
2076	42	0.193	$ 1,063,980	$ 513,780	$ 226,690	$ 109,465	$ 117,225
2077	43	0.185	$ 1,074,266	$ 518,150	$ 220,609	$ 106,406	$ 114,203
2078	44	0.178	$ 1,084,552	$ 522,521	$ 214,671	$ 103,425	$ 111,246
2079	45	0.171	$ 1,094,838	$ 526,892	$ 208,874	$ 100,521	$ 108,353
2080	46	0.165	$ 1,105,124	$ 531,262	$ 203,216	$ 97,691	$ 105,525
2081	47	0.158	$ 1,115,410	$ 535,633	$ 197,694	$ 94,935	$ 102,759
2082	48	0.152	$ 1,125,696	$ 540,004	$ 192,306	$ 92,250	$ 100,055
2083	49	0.146	$ 1,135,982	$ 544,374	$ 187,048	$ 89,636	$ 97,413
2084	50	0.141	$ 1,146,267	$ 548,745	$ 181,920	$ 87,089	$ 94,831

	W/O	With	Benefit
Amortization Factor	0.04457	0.04457	0.04457
Equivalent Annual (2024-2084)	1,164,866	581,462	583,405
Equivalent Annual (2035-2084)	819,530	409,908	409,622
Equivalent Annual (2024-2034)	345,336	171,554	173,782

Note: Present value and amortization factors are based on the fiscal year 2012 Federal discount rate of 3.75 percent

	3 % AEP
Partial Performance begins in:	2024
Full Performance begins in:	2035
Base Year	2035

Table 21
1% AEP Calculation of Equivalent Annual Damages and Benefits (1000's)
Base Year 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Year	Years from Base Year	PV Factor	Expected Annual Without- Project Damages	Expected Annual With-Project Damages	Present Value (PV) Expected Annual Damages/Benefits		
					Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Expected Annual Benefits
2010-12	-22	2.370	$ 485,766	-	-	-	-
2013	-21	2.279	-	-	-	-	-
2014	-20	2.191	-	-	-	-	-
2015	-19	2.107	-	-	-	-	-
2016	-18	2.026	-	-	-	-	-
2017	-17	1.948	-	-	-	-	-
2018	-16	1.873	-	-	-	-	-
2019	-15	1.801	-	-	-	-	-
2020	-14	1.732	-	-	-	-	-
2021	-13	1.665	-	-	-	-	-
2022	-12	1.601	-	-	-	-	-
2023	-11	1.539	-	-	-	-	-
2024	-10	1.480	$ 538,070	$ 145,647	$ 777,535	$ 229,418	$ 548,117
2025	-9	1.423	$ 547,542	$ 154,162	$ 762,624	$ 234,318	$ 528,306
2026	-8	1.369	$ 557,014	$ 162,676	$ 747,774	$ 238,564	$ 509,211
2027	-7	1.316	$ 566,485	$ 171,191	$ 733,002	$ 242,197	$ 490,805
2028	-6	1.265	$ 575,957	$ 179,706	$ 718,321	$ 245,256	$ 473,065
2029	-5	1.217	$ 585,429	$ 188,221	$ 703,744	$ 247,777	$ 455,967
2030	-4	1.170	$ 594,900	$ 196,735	$ 689,282	$ 249,796	$ 439,486
2031	-3	1.125	$ 604,372	$ 205,250	$ 674,945	$ 251,345	$ 423,601
2032	-2	1.082	$ 613,844	$ 213,765	$ 660,745	$ 252,455	$ 408,290
2033	-1	1.040	$ 623,315	$ 222,279	$ 646,690	$ 253,157	$ 393,533
2034	0	1.000	$ 632,787	$ 230,794	$ 632,787	$ 253,479	$ 379,308
2035	1	0.962	$ 642,259	$ 102,896	$ 619,045	$ 107,269	$ 511,776
2036	2	0.925	$ 652,545	$ 103,534	$ 606,225	$ 104,042	$ 502,183
2037	3	0.889	$ 662,831	$ 104,172	$ 593,524	$ 100,908	$ 492,615
2038	4	0.855	$ 673,116	$ 104,810	$ 580,949	$ 97,866	$ 483,083
2039	5	0.822	$ 683,402	$ 105,447	$ 568,507	$ 94,911	$ 473,596
2040	6	0.790	$ 693,688	$ 106,085	$ 556,206	$ 92,042	$ 464,164
2041	7	0.760	$ 703,974	$ 106,723	$ 544,051	$ 89,256	$ 454,795
2042	8	0.731	$ 714,260	$ 107,361	$ 532,049	$ 86,552	$ 445,497
2043	9	0.703	$ 724,546	$ 107,998	$ 520,203	$ 83,926	$ 436,277
2044	10	0.676	$ 734,832	$ 108,636	$ 508,519	$ 81,377	$ 427,142
2045	11	0.650	$ 745,118	$ 109,274	$ 496,999	$ 78,903	$ 418,096
2046	12	0.625	$ 755,404	$ 109,912	$ 485,648	$ 76,501	$ 409,147
2047	13	0.601	$ 765,690	$ 110,550	$ 474,468	$ 74,170	$ 400,298
2048	14	0.577	$ 775,975	$ 111,187	$ 463,462	$ 71,907	$ 391,555
2049	15	0.555	$ 786,261	$ 111,825	$ 452,632	$ 69,711	$ 382,921
2050	16	0.534	$ 796,547	$ 112,463	$ 441,979	$ 67,580	$ 374,399
2051	17	0.513	$ 806,833	$ 113,101	$ 431,505	$ 65,512	$ 365,993
2052	18	0.494	$ 817,119	$ 113,738	$ 421,211	$ 63,505	$ 357,705
2053	19	0.475	$ 827,405	$ 114,376	$ 411,097	$ 61,558	$ 349,539
2054	20	0.456	$ 837,691	$ 115,014	$ 401,164	$ 59,668	$ 341,496
2055	21	0.439	$ 847,977	$ 115,652	$ 391,412	$ 57,835	$ 333,577
2056	22	0.422	$ 858,263	$ 116,290	$ 381,840	$ 56,056	$ 325,785
2057	23	0.406	$ 868,548	$ 116,927	$ 372,450	$ 54,330	$ 318,120
2058	24	0.390	$ 878,834	$ 117,565	$ 363,239	$ 52,656	$ 310,583
2059	25	0.375	$ 889,120	$ 118,203	$ 354,208	$ 51,031	$ 303,176
2060	26	0.361	$ 899,406	$ 118,841	$ 345,354	$ 49,456	$ 295,899
2061	27	0.347	$ 909,692	$ 119,478	$ 336,679	$ 47,927	$ 288,751
2062	28	0.333	$ 919,978	$ 120,116	$ 328,179	$ 46,445	$ 281,734
2063	29	0.321	$ 930,264	$ 120,754	$ 319,853	$ 45,007	$ 274,846

Table 21 (Cont.)
1% AEP Calculation of Equivalent Annual Damages and Benefits (1000's)
Base Year 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Year	Years from Base Year	PV Factor	Expected Annual Without- Project Damages	Expected Annual With-Project Damages	Present Value (PV) Expected Annual Damages/Benefits		
					Expected Annual Without-Project Damages	Expected Annual With-Project Damages	Expected Annual Benefits
2064	30	0.308	$ 940,550	$ 131,599	$ 311,701	$ 43,612	$ 268,089
2065	31	0.296	$ 950,836	$ 132,299	$ 303,721	$ 42,260	$ 261,461
2066	32	0.285	$ 961,121	$ 132,999	$ 295,909	$ 40,948	$ 254,962
2067	33	0.274	$ 971,407	$ 133,700	$ 288,266	$ 39,676	$ 248,591
2068	34	0.264	$ 981,693	$ 134,400	$ 280,789	$ 38,442	$ 242,347
2069	35	0.253	$ 991,979	$ 135,100	$ 273,476	$ 37,245	$ 236,230
2070	36	0.244	$ 1,002,265	$ 135,800	$ 266,324	$ 36,085	$ 230,239
2071	37	0.234	$ 1,012,551	$ 136,501	$ 259,333	$ 34,960	$ 224,372
2072	38	0.225	$ 1,022,837	$ 137,201	$ 252,498	$ 33,870	$ 218,629
2073	39	0.217	$ 1,033,123	$ 137,901	$ 245,819	$ 32,812	$ 213,007
2074	40	0.208	$ 1,043,409	$ 138,601	$ 239,293	$ 31,787	$ 207,507
2075	41	0.200	$ 1,053,694	$ 139,302	$ 232,918	$ 30,792	$ 202,125
2076	42	0.193	$ 1,063,980	$ 140,002	$ 226,690	$ 29,829	$ 196,862
2077	43	0.185	$ 1,074,266	$ 140,702	$ 220,609	$ 28,894	$ 191,715
2078	44	0.178	$ 1,084,552	$ 141,403	$ 214,671	$ 27,989	$ 186,683
2079	45	0.171	$ 1,094,838	$ 142,103	$ 208,874	$ 27,111	$ 181,764
2080	46	0.165	$ 1,105,124	$ 142,803	$ 203,216	$ 26,259	$ 176,957
2081	47	0.158	$ 1,115,410	$ 143,503	$ 197,694	$ 25,434	$ 172,260
2082	48	0.152	$ 1,125,696	$ 144,204	$ 192,306	$ 24,635	$ 167,671
2083	49	0.146	$ 1,135,982	$ 144,904	$ 187,048	$ 23,860	$ 163,189
2084	50	0.141	$ 1,146,267	$ 145,604	$ 181,920	$ 23,108	$ 158,812
					W/O	With	Benefit
				Amortization Factor	0.04457	0.04457	0.04457
				Equivalent Annual (2024-2084)	1,064,961	223,725	841,236
				Equivalent Annual (2035-2084)	747,898	113,888	634,010
				Equivalent Annual (2024-2034)	317,063	109,837	207,226

Note: Present value and amortization factors are based on the fiscal year 2012 Federal discount rate of 3.75 percen

	1% AEP
Partial Performance begins in:	2024
Full Performance begins in:	2035
Base Year	2035

Table 22
Equivalent Annual Damages and Benefits to Residential and Non-Residential Categories (1000's)
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

| Plan Name | Equivalent Annual Damage | | | Probability Damage Reduced Exceeds Indicated Values | | |
	Without-Project	With-Project	Damages Reduced	0.75	0.50	0.25
Alt 3%	$ 1,064,961	$ 540,054	$ 524,907	$ 183,350	$ 503,018	$ 653,719
Alt 1%	$ 1,064,961	$ 223,725	$ 841,236	$ 563,796	$ 886,158	$ 1,171,757

Note: Expected annual damages for structures, their contents, and vehicles were calculated for the years 2024, 2035, and 2085 and converted to equivalent annual values.

Table 23
Equivalent Annual Damages and Benefits for Industrial Properties Category (1000's)
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

| Plan Name | Equivalent Annual Damage | | | Probability Damage Reduced Exceeds Indicated Values | | |
	Without-Project	With-Project	Damages Reduced	0.75	0.50	0.25
Alt 3%	$ 24,252	$ 9,566	$ 14,686	$ 6,315	$ 15,670	$ 20,406
Alt 1%	$ 24,252	$ 3,695	$ 20,557	$ 14,322	$ 21,939	$ 28,564

Note: Expected annual damages for industrial properties for the years 2024, 2035, and 2085, were converted to equivalent annual values.

Table 24
Number of Structures Elevated by Analysis Year
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Analysis Year	Residential	Non-Residential
2024	707	66
2035	417	47
2085	1,789	66
Total	2,913	179

Table 25
Structure-Raising Costs
(Dollars per Square Foot in 2011 price level)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Ft. Raised	1-Sty Slab	2-Sty Slab	1-Sty Pier	2-Sty Pier	Mobile Home
1.0	74.52	82.62	65.88	72.90	36.72
2.0	74.52	82.62	65.88	72.90	36.72
3.0	76.14	84.24	68.58	75.60	36.72
4.0	78.84	89.64	68.58	75.60	36.72
5.0	78.84	89.64	68.58	75.60	44.82
6.0	80.46	91.26	70.20	77.22	44.82
7.0	80.46	91.26	70.20	77.22	44.82
8.0	83.16	93.96	71.82	78.84	44.82
9.0	83.16	93.96	71.82	78.84	44.82
10.0	83.16	93.96	71.82	78.84	44.82
11.0	83.16	93.96	71.82	78.84	44.82
12.0	83.16	93.96	71.82	78.84	44.82
13.0	85.86	99.36	73.44	80.46	44.82

Source: Based on interviews with three major shoring companies in the Metropolitan New Orleans area
Note: Temporary Relocation costs equal to $3,750 were also applied to the elevated structures.

Table 26
Structure-Raising Costs Avoided
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Time Period	Total No. of Structures	Average No. of Feet Raised	Total Cost (in Millions)	Average Cost (in Thousands)
2010 to 2024	773	11	$ 108	$ 140
2025 to 2035	464	12	$ 95	$ 205
2036 to 2085	1855	11	$ 238	$ 128

Table 27
Depth-Damage Relationships for Debris Removal and Cleanup Cost
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Parameter	Stage in Feet				
1STY-PIER	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 4,957	$ 5,354	$ 5,828
	Standard Deviation	$ -	$ -	$ 817	$ 831	$ 854
1STY-SLAB	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 4,956	$ 5,353	$ 5,748
	Standard Deviation	$ -	$ -	$ 816	$ 830	$ 840
2STY-PIER	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 4,957	$ 5,353	$ 5,828
	Standard Deviation	$ -	$ -	$ 817	$ 830	$ 851
2STY-SLAB	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 6,262	$ 6,870	$ 7,610
	Standard Deviation	$ -	$ -	$ 855	$ 881	$ 916
EAT	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 33,060	$ 33,645	$ 34,451
	Standard Deviation	$ -	$ -	$ 7,740	$ 7,744	$ 7,748
GROC	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 34,736	$ 35,483	$ 36,481
	Standard Deviation	$ -	$ -	$ 7,757	$ 7,756	$ 7,766
MOBHOM	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 4,822	$ 5,252	$ 5,860
	Standard Deviation	$ -	$ -	$ 814	$ 823	$ 860
MULT	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 7,955	$ 8,581	$ 10,277
	Standard Deviation	$ -	$ -	$ 685	$ 738	$ 997
PROF	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 33,799	$ 34,294	$ 35,643
	Standard Deviation	$ -	$ -	$ 7,742	$ 7,745	$ 7,762
PUBL	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 33,799	$ 34,294	$ 35,643
	Standard Deviation	$ -	$ -	$ 7,742	$ 7,746	$ 7,763
REPA	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 35,141	$ 35,889	$ 36,886
	Standard Deviation	$ -	$ -	$ 7,757	$ 7,758	$ 7,768
RETA	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 33,585	$ 34,080	$ 35,429
	Standard Deviation	$ -	$ -	$ 7,741	$ 7,745	$ 7,762
WARE	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 48,057	$ 54,939	$ 63,208
	Standard Deviation	$ -	$ -	$ 8,283	$ 8,541	$ 8,929

Source: Based on *Development of Depth-Emergency Costs and Infrastructure Damage Relationships for Selected South Louisiana Parishes Final Report dated March 2012*

Table 28
Equivalent Annual Damages and Benefits for Debris Category (1000's)
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

| Plan Name | Equivalent Annual Damage | | | Probability Damage Reduced Exceeds Indicated Values | | |
	Without-Project	With-Project	Damages Reduced	0.75	0.50	0.25
Alt 3%	$ 36,905	$ 17,908	$ 18,997	$ 14,108	$ 18,460	$ 23,377
Alt 1%	$ 36,905	$ 7,878	$ 29,027	$ 19,960	$ 30,217	$ 39,392

Note: Expected annual damages for the years 2024, 2035, and 2085 were converted to equivalent annual values.

Table 29
Depth-Damage Relationships for Major & Secondary Highways and Streets
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Parameter	Stage in Feet					
	Stage	0.0	1.9	2.0	5.0	12.0
STREETS	Percentage of Structure Damage	$ -	$ -	$ 88,262	$ 162,965	$ 246,059
	Standard Deviation	$ -	$ -	$ 22,441	$ 27,017	$ 37,795
HIGHWAY	Stage	0.0	1.9	2.0	5.0	12.0
	Percentage of Structure Damage	$ -	$ -	$ 158,070	$ 483,837	$ 669,393

Source: Based on Development of Depth-Emergency Costs and Infrastructure Damage Relationships for Selected South
Louisiana Parishes Final Report dated March 2012

Table 30
Equivalent Annual Damages and Benefits for Major & Secondary Highways and Streets (1000's)
(2024-2085)

Morganza to the Gulf of Mexico, LA

Post-Authorization Change Report

Plan Name	Equivalent Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
	Without-Project	With-Project	Damages Reduced	0.75	0.50	0.25
Alt 3%	$ 31,476	$ 14,326	$ 17,151	$ 10,291	$ 15,779	$ 19,895
Alt 1%	$ 31,476	$ 8,088	$ 23,389	$ 14,570	$ 21,645	$ 27,838

Note: Expected annual damages for the years 2024, 2035, and 2085 were converted to equivalent annual values.

Table 31
Average Annual Agricultural Acres Impacted
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Study Area Reaches	2010	2035					2085				
		Without-Project	Without-Project	With 35 Yr	With 100 Yr	Benefits 35 Yr	Benefits 100 Yr	Without-Project	With 35 Yr	With 100 Yr	Benefits 35 Yr	Benefits 100 Yr
		Acres	Acres	Acres	Acres	Acres	Acres	Acres	Acres	Acres	Acres	Acres
1	1-5	77	138	5	0	133	138	317	10	2	307	314
2	2-1A2	19	23	10	1	13	22	23	10	1	13	22
3	2-1B2N	53	61	30	4	31	57	68	31	4	38	65
4	4-7	11	17	1	0	16	17	47	1	0	46	47
5	9-1AE	10	10	10	10	0	0	20	20	20	0	0
6	9-1AMID	18	18	18	18	0	0	36	36	36	0	0
7	9-1AW	9	9	9	9	0	0	18	18	18	0	0
8	9-1BE	2	2	0	0	2	2	3	0	0	3	3
9	9-1BMIDE	0	0	0	0	0	0	1	1	1	0	0
10	9-1BMIDW	3	3	3	3	0	0	5	5	5	0	0
11	9-1BW	10	11	11	11	0	0	21	21	21	0	0
12	BL5	13	15	8	1	8	14	35	8	1	27	34
13	BL6	1	2	0	0	2	2	6	1	0	5	6
14	BL7	7	9	2	0	7	9	46	3	0	43	46
15	BL89	14	22	4	1	18	21	75	8	1	67	74
16	C1-LF	27	27	12	1	15	26	31	12	1	20	30
17	D1b-LF	16	16	7	1	9	15	16	7	1	9	15
18	D1c-LF2	19	25	10	1	15	24	86	10	1	76	85
19	D1c-LF3	38	57	17	2	40	55	297	17	2	280	295
20	D-28	18	20	9	1	11	19	21	10	1	11	20
21	D-31	12	12	5	1	7	11	12	5	1	7	11
22	E2-LF	82	138	14	2	124	136	323	14	2	309	322
23	GW14	8	8	2	0	6	8	19	4	0	15	18
24	GW16	8	10	2	0	8	10	20	2	0	18	20
25	GW2	36	68	2	1	66	67	139	5	1	134	139
26	HNC6	16	24	1	0	23	24	43	2	0	41	43
27	SL2	28	40	5	1	35	39	172	10	1	162	171
28	TS1	1	1	1	0	0	1	1	1	0	0	1
29	TS10	8	11	8	1	3	10	18	8	1	10	17
30	TS11	0	0	0	0	0	0	0	0	0	0	0
31	TS2	3	3	3	1	0	2	6	6	1	0	4
32	TS3	1	3	1	0	2	3	3	1	0	2	3
33	TS5	9	10	6	1	4	9	12	6	1	6	12
34	TS6	9	10	6	1	4	9	15	6	1	9	15
35	TS7	5	7	5	1	2	6	10	5	1	5	9
36	TS9	16	19	10	1	9	18	23	11	1	13	22
Total		607	849	236	75	613	774	1989	312	128	1677	1861

Note: Agricultural acres in the eastern Federal levee tie-in areas north of Bayou Lafourche are not included in the table.

Table 32
Percent of Total Catch Caught by Vessel Size Category
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Length	2009	2024	2035	2085
Total <26'	7.2%	6.5%	5.9%	5.7%
26' to < 40'	23.2%	20.9%	19.3%	18.5%
40' to < 65'	55.2%	56.7%	57.8%	58.3%
65' and over	14.4%	15.9%	17.0%	17.5%

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report
dated April 2012

Table 33
Median Catch by Vessel Size
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Length	Median Catch (lbs)
≤20'	3,000
21' to ≤ 40'	20,300
41' to ≤ 60'	47,653
60' and over	69,050

Source: Economic Benefits of Protecting the Large
Recreational and Commercial Boat Fleets Final Report
dated April 2012

Table 34
Commercial Fishing Vessel Forecast
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Low			Median			High		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	222	204	196	339	313	300	457	421	404
40' to < 65'	256	261	263	391	399	402	527	537	542
65' and over	50	53	55	76	81	83	102	109	112
Total >=26	527	518	514	807	793	786	1,086	1,067	1,059

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 35
Recreational Fleet Historical Growth Rates
(1999-2009)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Vessel Size	Annual Growth	
	1999-2009	2002-2009
26' to < 40'	1.8%	2.5%
40' to < 65'	3.2%	5.2%
65' and over	5.0%	7.1%
Total ≥26	2.0%	2.9%

Source: Economic Benefits of Protecting the Large
Recreational and Commercial Boat Fleets Final Report
dated April 2012

Table 36
Recreational Fleet Growth Rates
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Vessel Size	Median		High	
	To 2040	To 2085	To 2040	To 2085
26' to < 40'	1.0%	0.5%	2.0%	0.7%
40' to < 65'	2.5%	1.0%	4.0%	1.3%
65' and over	3.0%	1.5%	6.0%	2.0%

Source: Economic Benefits of Protecting the Large Recreational and Commercial
Boat Fleets Final Report dated April 2012

Table 37
Number of Recreational Vessel Forecast
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Low			Median			High		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	283	283	283	329	367	482	381	474	716
40' to < 65'	67	67	67	97	127	225	121	186	404
65' and over	11	11	11	17	24	54	26	50	163
Total >=26	361	361	361	443	518	761	528	709	1,283

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 38
Commercial Passenger Vessel Forecast
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Low			Median			High		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	18	16	11	21	21	21	24	26	33
40' to < 65'	8	7	5	9	9	9	10	12	17
65' and over	3	2	2	3	3	3	3	4	5
Total >=26	28	25	18	33	33	33	37	42	55

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 39
Distribution of Vessel Fleets
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Waterway	Percentage of Vessel Fleet	Below Project Alignment	Above Project Alignment
Houma Navigation Channel	5%	2%	3%
Bayou Petit Caillou	30%	5%	25%
Bayou Grand Caillou	30%	0%	30%
Bayou Dularge	15%	12%	3%
Bayou Terrebonne	15%	1%	14%
Bayou Pointe aux Chene	5%	2%	3%
Total	100%	22%	78%

Source: Economic Benefits of Protecting the Large Recreational and Commercial
Boat Fleets Final Report dated April 2012

Table 40
Distance to Refuge Without- & With-Project
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Without-Project				
	Percent Below	Nautical Miles	Percent Above	Distance	Weighted Average
Houma Navigation Channel	2%	15.0	3%	0.0	0.30
Bayou Petit Caillou	5%	20.0	25%	15.0	4.75
Bayou Grand Caillou	0%	0.0	30%	10.0	3.00
Bayou Dularge	12%	8.0	3%	2.0	1.02
Bayou Terrebonne	1%	15.0	14%	12.0	1.83
Bayou Pointe aux Chene	2%	1.0	3%	0.5	0.04
Total	**22%**		**78%**		**10.94**
	With-Project				
	Percent Below	Nautical Miles	Percent Above	Distance	Weighted Average
Houma Navigation Channel	2%	10.0	3%	0.0	0.20
Bayou Petit Caillou	5%	8.0	25%	0.0	0.40
Bayou Grand Caillou	0%	0.0	30%	0.0	0.00
Bayou Dularge	12%	4.0	3%	0.0	0.48
Bayou Terrebonne	1%	5.0	14%	0.0	0.05
Bayou Pointe aux Chene	2%	0.5	3%	0.0	0.01
Total	**22%**		**78%**		**1.14**

Source: *Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report* dated April 2012

Table 41
Travel Cost by Vessel Type and Size
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Recreation & Commercial Passenger

Vessel Size	Speed (knots)	Hourly Fuel Consumption	Cost of Diesel (gal)	Hourly Fuel Cost	Hourly Cost of Crew	Hourly Operating Cost	Cost per Nautical Mile
26' to < 40'	23	25	$ 3.13	$ 78.25	$ 20.00	$ 98.25	$ 4.27
40' to < 65'	18	25	$ 3.13	$ 79.29	$ 35.00	$ 114.29	$ 6.35
65' and over	11	16	$ 3.13	$ 50.08	$ 50.00	$ 100.08	$ 9.10

Commercial Fishing

Vessel Size	Speed (knots)	Hourly Fuel Consumption	Cost of Diesel (gal)	Hourly Fuel Cost	Hourly Cost of Crew	Hourly Operating Cost	Cost per Nautical Mile
26' to < 40'	8	6	$ 3.13	$ 18.78	$ 35.00	$ 53.78	$ 6.72
40' to < 65'	10	8	$ 3.13	$ 25.04	$ 50.00	$ 75.04	$ 7.50
65' and over	10	16	$ 3.13	$ 50.08	$ 65.00	$ 115.08	$ 11.51

Other Vessels

Vessel Size	Speed (knots)	Hourly Fuel Consumption	Cost of Diesel (gal)	Hourly Fuel Cost	Hourly Cost of Crew	Hourly Operating Cost	Cost per Nautical Mile
26' to < 40'	8	10	$ 3.13	$ 31.30	$ 50.00	$ 81.30	$ 10.16
40' to < 65'	10	16	$ 3.13	$ 50.08	$ 65.00	$ 115.08	$ 11.51
65' and over	12	20	$ 3.13	$ 62.60	$ 80.00	$ 142.60	$ 11.88

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 42
Total Travel Cost by Vessel Size
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Without-Project

Vessel Size	Low Forecast			Median Forecast			High Forecast		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	30,804	29,426	28,609	41,715	41,524	46,032	52,942	54,718	65,151
40' to < 65'	29,704	30,054	30,122	42,998	45,724	52,818	55,835	61,333	77,224
65' and over	15,259	15,663	15,803	19,217	20,532	23,818	23,491	26,775	38,564
Total	75,767	75,143	74,534	103,930	107,780	122,668	132,268	142,826	180,940

With-Project

Vessel Size	Low Forecast			Median Forecast			High Forecast		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	3,211	3,068	2,983	4,349	4,329	4,799	5,519	5,705	6,792
40' to < 65'	3,097	3,133	3,140	4,483	4,767	5,506	5,821	6,394	8,051
65' and over	1,591	1,633	1,647	2,003	2,140	2,483	2,449	2,791	4,020
Total	7,899	7,834	7,770	10,835	11,236	12,788	13,789	14,890	18,863

Benefits

Vessel Size	Low Forecast			Median Forecast			High Forecast		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
26' to < 40'	27,593	26,359	25,626	37,366	37,195	41,233	47,423	49,014	58,359
40' to < 65'	26,607	26,921	26,982	38,515	40,957	47,312	50,014	54,939	69,174
65' and over	13,668	14,030	14,155	17,213	18,391	21,335	21,042	23,983	34,544
Total	67,868	67,309	66,763	93,095	96,544	109,880	118,478	127,936	162,076

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 43
Expected Annual Travel Cost and Benefits
(2024-2085)

Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

	Low Forecast			Median Forecast			High Forecast		
	2024	2035	2085	2024	2035	2085	2024	2035	2085
Without-Project Costs	11,214	11,121	11,031	15,382	15,951	18,155	19,576	21,138	26,779
With-Project Costs	1,169	1,159	1,150	1,604	1,663	1,893	2,041	2,204	2,792
Benefits	10,045	9,962	9,881	13,778	14,288	16,262	17,535	18,934	23,987

Source: Economic Benefits of Protecting the Large Recreational and Commercial Boat Fleets Final Report dated April 2012

Table 44
Additional Costs Associated with High Salinity Levels
Morganza to the Gulf
Post-Authorization Change Report

| | Without-Project | | | | | | | With-Project 1% and 3% AEP Alternatives | | | | | |
Year	Chemicals Costs	Treatment Costs	Operation Costs	Refurbish Costs	Replacement Costs	TOTAL	Year	Chemicals Costs	Treatment Costs	Operation Costs	Refurbish Costs	Replacement Costs	TOTAL
2012	$10,744	$275,000	$330			$286,074	2012	$10,744	$275,000	$330			$286,074
2013	$10,744		$330			$11,074	2013	$10,744		$330			$11,074
2014	$10,744		$330			$11,074	2014	$10,744		$330			$11,074
2015	$10,744	$275,000	$330			$286,074	2015	$10,744	$275,000	$330			$286,074
2016	$10,744		$330			$11,074	2016	$10,744		$330			$11,074
2017	$10,744		$330			$11,074	2017	$10,744		$330			$11,074
2018	$10,744	$275,000	$330			$286,074	2018	$10,744	$275,000	$330			$286,074
2019	$10,744		$330			$11,074	2019	$8,175		$251			$8,426
2020	$10,744		$330	$1,000,000	$500,000	$1,511,074	2020	$8,175		$251			$8,426
2021	$10,744	$275,000	$330			$286,074	2021	$8,175		$251			$8,426
2022	$10,744		$330			$11,074	2022	$8,175	$275,000	$251			$283,426
2023	$10,744		$330			$11,074	2023	$8,175		$251			$8,426
2024	$10,744	$275,000	$330			$286,074	2024	$8,175		$251			$8,426
2025	$10,744		$330			$11,074	2025	$8,175		$251			$8,426
2026	$10,744		$330			$11,074	2026	$8,175	$275,000	$251			$283,426
2027	$10,744	$275,000	$330			$286,074	2027	$8,175		$251			$8,426
2028	$10,744		$330			$11,074	2028	$8,175		$251			$8,426
2029	$10,744		$330			$11,074	2029	$8,175		$251			$8,426
2030	$10,744	$275,000	$330			$286,074	2030	$8,175	$275,000	$251			$283,426
2031	$10,744		$330			$11,074	2031	$8,175		$251			$8,426
2032	$10,744		$330			$11,074	2032	$8,175		$251			$8,426
2033	$10,744	$275,000	$330			$286,074	2033	$8,175		$251			$8,426
2034	$10,744		$330			$11,074	2034	$8,175	$275,000	$251			$283,426
2035	$10,744		$330			$11,074	2035	$8,175		$251			$8,426
2036	$10,744	$275,000	$330			$286,074	2036	$8,175		$251			$8,426
2037	$10,744		$330			$11,074	2037	$8,175		$251			$8,426
2038	$10,744		$330			$11,074	2038	$8,175	$275,000	$251			$283,426
2039	$10,744	$275,000	$330			$286,074	2039	$8,175		$251			$8,426
2040	$10,744		$330	$135,000	$135,000	$281,074	2040	$8,175		$251			$8,426
2041	$10,744		$330			$11,074	2041	$8,175		$251			$8,426
2042	$10,744	$275,000	$330			$286,074	2042	$8,175	$275,000	$251			$283,426
2043	$10,744		$330			$11,074	2043	$8,175		$251			$8,426
2044	$10,744		$330			$11,074	2044	$8,175		$251			$8,426
2045	$10,744	$275,000	$330			$286,074	2045	$8,175		$251			$8,426
2046	$10,744		$330			$11,074	2046	$8,175	$275,000	$251			$283,426
2047	$10,744		$330			$11,074	2047	$8,175		$251			$8,426
2048	$10,744	$275,000	$330			$286,074	2048	$8,175		$251			$8,426
2049	$10,744		$330			$11,074	2049	$8,175		$251			$8,426
2050	$10,744		$330			$11,074	2050	$8,175	$275,000	$251			$283,426
2051	$10,744	$275,000	$330			$286,074	2051	$8,175		$251			$8,426
2052	$10,744		$330			$11,074	2052	$8,175		$251			$8,426
2053	$10,744		$330			$11,074	2053	$8,175		$251			$8,426
2054	$10,744	$275,000	$330			$286,074	2054	$8,175	$275,000	$251			$283,426
2055	$10,744		$330			$11,074	2055	$8,175		$251			$8,426
2056	$10,744		$330			$11,074	2056	$8,175		$251			$8,426
2057	$10,744	$275,000	$330			$286,074	2057	$8,175		$251			$8,426
2058	$10,744		$330			$11,074	2058	$8,175	$275,000	$251			$283,426
2059	$10,744		$330			$11,074	2059	$8,175		$251			$8,426
2060	$10,744	$275,000	$330	$1,000,000	$500,000	$1,786,074	2060	$8,175		$251			$8,426
2061	$10,744		$330			$11,074	2061	$8,175		$251			$8,426
2062	$10,744		$330			$11,074	2062	$8,175	$275,000	$251			$283,426
2063	$10,744	$275,000	$330			$286,074	2063	$8,175		$251			$8,426
2064	$10,744		$330			$11,074	2064	$8,175		$251			$8,426
2065	$10,744		$330			$11,074	2065	$8,175		$251			$8,426
2066	$10,744	$275,000	$330			$286,074	2066	$8,175	$275,000	$251			$283,426
2067	$10,744		$330			$11,074	2067	$8,175		$251			$8,426
2068	$10,744		$330			$11,074	2068	$8,175		$251			$8,426
2069	$10,744	$275,000	$330			$286,074	2069	$8,175		$251			$8,426
2070	$10,744		$330			$11,074	2070	$8,175	$275,000	$251			$283,426
2071	$10,744		$330			$11,074	2071	$8,175		$251			$8,426
2072	$10,744	$275,000	$330			$286,074	2072	$8,175		$251			$8,426
2073	$10,744		$330			$11,074	2073	$8,175		$251			$8,426
2074	$10,744		$330			$11,074	2074	$8,175	$275,000	$251			$283,426
2075	$10,744	$275,000	$330			$286,074	2075	$8,175		$251			$8,426
2076	$10,744		$330			$11,074	2076	$8,175		$251			$8,426
2077	$10,744		$330			$11,074	2077	$8,175		$251			$8,426
2078	$10,744	$275,000	$330			$286,074	2078	$8,175	$275,000	$251			$283,426
2079	$10,744		$330			$11,074	2079	$8,175		$251			$8,426
2080	$10,744		$330	$135,000	$135,000	$281,074	2080	$8,175		$251			$8,426
2081	$10,744	$275,000	$330			$286,074	2081	$8,175		$251			$8,426
2082	$10,744		$330			$11,074	2082	$8,175	$275,000	$251			$283,426
2083	$10,744		$330			$11,074	2083	$8,175		$251			$8,426
2084	$10,744	$275,000	$330			$286,074	2084	$8,175		$251			$8,426

Source: Based on schedule of water supply costs provided by Houma Water Treatment Plant

Note: The expected value for the annual number of days of high salinity during the period, 2012 to 2084, is 31.2 days.

Table 45
3% AEP Total Annual Costs
(2011 Price Level; 3.75% Discount Rate)
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Year	Years from Base Year	Expenditures	Present Value Factor	Present Value of Expenditures
2010	-24		2.419	0
2011	-23		2.332	0
2012	-22	$0	2.248	0
2013	-21	$0	2.166	0
2014	-20	$14	2.088	30
2015	-19	$695	2.013	1,398
2016	-18	$625	1.940	1,213
2017	-17	$716	1.870	1,339
2018	-16	$708	1.802	1,276
2019	-15	$398	1.737	691
2020	-14	$355	1.674	595
2021	-13	$587	1.614	947
2022	-12	$557	1.555	866
2023	-11	$328	1.499	491
2024	-10	$102	1.445	147
2025	-9	$35	1.393	48
2026	-8	$20	1.342	27
2027	-7	$0	1.294	0
2028	-6	$0	1.247	0
2029	-5	$0	1.202	0
2030	-4	$10	1.159	12
2031	-3	$8	1.117	9
2032	-2	$8	1.076	8
2033	-1	$106	1.038	110
2034	0	$106	1.000	106
2035	1	$213	0.964	205
2036	2	$34	0.929	32
2037	3	$13	0.895	12
2038	4	$13	0.863	11
2039	5	$0	0.832	0
2040	6	$32	0.802	25
2041	7	$21	0.773	16
2042	8	$14	0.745	10
2043	9	$14	0.718	10
2044	10	$0	0.692	0
2045	11	$20	0.667	14
2046	12	$10	0.643	7

Table 45 (Cont.)
3% AEP Total Annual Costs
(2011 Price Level; 3.75% Discount Rate)
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Year	Years from Base Year	Expenditures	Present Value Factor	Present Value of Expenditures
2047	13	$0	0.620	0
2048	14	$0	0.597	0
2049	15	$0	0.576	0
2050	16	$25	0.555	14
2051	17	$8	0.535	4
2052	18	$0	0.515	0
2053	19	$17	0.497	8
2054	20	$17	0.479	8
2055	21	$0	0.462	0
2056	22	$0	0.445	0
2057	23	$0	0.429	0
2058	24	$0	0.413	0
2059	25	$0	0.398	0
2060	26	$0	0.384	0
2061	27	$0	0.370	0
2062	28	$0	0.357	0
2063	29	$0	0.344	0
2064	30	$0	0.331	0
2065	31	$11	0.319	4
2066	32	$0	0.308	0
2067	33	$0	0.297	0
2068	34	$0	0.286	0
2069	35	$0	0.276	0
2070	36	$35	0.266	9
2071	37	$0	0.256	0
2072	38	$0	0.247	0
2073	39	$15	0.238	3
2074	40	$15	0.229	3
2075	41		0.221	0

Discount Rate (%)	3.75			
Amortization Factor	0.04457			
Annual Implementation Costs			$	432.8
Operations and Maintenance Cost			$	5.5
Total Annual Costs ($Millions)			$	438.3

*Project costs include acquistion costs of structures in 12 study area reaches receiving induced damages south of the proposed alternatives.

Table 46
1% AEP Total Annual Costs
(2011 Price Level; 3.75% Discount Rate)
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Year	Period of Analysis	Construction Cost	PV Factor	PV Construction Cost
2010	-24		2.419	0
2011	-23		2.332	0
2012	-22	$0	2.248	0
2013	-21	$0	2.166	0
2014	-20	$22	2.088	46
2015	-19	$822	2.013	1,653
2016	-18	$728	1.940	1,412
2017	-17	$893	1.870	1,670
2018	-16	$958	1.802	1,727
2019	-15	$663	1.737	1,151
2020	-14	$445	1.674	744
2021	-13	$773	1.614	1,248
2022	-12	$836	1.555	1,301
2023	-11	$723	1.499	1,084
2024	-10	$605	1.445	874
2025	-9	$405	1.393	565
2026	-8	$231	1.342	310
2027	-7	$185	1.294	240
2028	-6	$162	1.247	202
2029	-5	$188	1.202	226
2030	-4	$185	1.159	214
2031	-3	$115	1.117	129
2032	-2	$21	1.076	22
2033	-1	$142	1.038	147
2034	0	$192	1.000	192
2035	1	$261	0.964	252
2036	2	$63	0.929	59
2037	3	$22	0.895	20
2038	4	$22	0.863	19
2039	5	$22	0.832	18
2040	6	$22	0.802	18
2041	7	$0	0.773	0
2042	8	$0	0.745	0
2043	9	$0	0.718	0
2044	10	$0	0.692	0
2045	11	$99	0.667	66
2046	12	$52	0.643	34

Table 46 (Cont.)
1% AEP Total Annual Costs
(2011 Price Level; 3.75% Discount Rate)
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Year	Period of Analysis	Construction Cost	PV Factor	PV Construction Cost
2047	13	$9	0.620	6
2048	14	$9	0.597	6
2049	15	$0	0.576	0
2050	16	$13	0.555	7
2051	17	$29	0.535	16
2052	18	$16	0.515	8
2053	19	$0	0.497	0
2054	20	$0	0.479	0
2055	21	$47	0.462	22
2056	22	$47	0.445	21
2057	23	$0	0.429	0
2058	24	$0	0.413	0
2059	25	$0	0.398	0
2060	26	$21	0.384	8
2061	27	$5	0.370	2
2062	28	$19	0.357	7
2063	29	$19	0.344	7
2064	30	$0	0.331	0
2065	31	$0	0.319	0
2066	32	$0	0.308	0
2067	33	$0	0.297	0
2068	34	$0	0.286	0
2069	35	$0	0.276	0
2070	36	$27	0.266	7
2071	37	$27	0.256	7
2072	38	$0	0.247	0
2073	39	$0	0.238	0
2074	40	$14	0.229	3
2075	41	$14	0.221	3
		$ 10,177.2		$ 15,772.4

Discount Rate (%)	3.75			
Amortization Factor	0.04457			
Annual Implementation Costs			$	703.0
Operations and Maintenance Cost			$	7.3
Total Annual Costs ($Millions)			$	710.3

*Project costs include acquistion costs of structures in 12 study area reaches receiving induced damages south of the proposed alternatives.

Table 47
3% Annual Exceedance Probability Alternative
(2011 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits
Damage Category					
Residential & Commercial - Structure/Content/Vehicles	$ 819.5	$ 409.9	$ 409.6	$ 173.8	$ 583.4
Industrial - Structure/Contents	$ 16.5	$ 6.4	$ 10.1	$ 4.4	$ 14.5
Highways	$ 6.6	$ 4.2	$ 2.3	$ 1.1	$ 3.4
Streets	$ 15.5	$ 5.6	$ 9.9	$ 3.7	$ 13.6
Debris Removal & Cleanup	$ 25.7	$ 12.6	$ 13.1	$ 5.9	$ 19.0
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3
Total	$ 888.9	$ 438.8	$ 450.1	$ 194.2	$ 644.4
First Costs					$ 5,902
Interest During Construction					$ 3,937
Annual Operation & Maintenance Costs					$ 6
Total Annual Costs					$ 438
B/C Ratio					1.47
Equivalent Annual Net Benefits - 2035 Base Year					$ 206

*Total equivalent annual benefits, net benefits and b/c ratio after changes to CSVRs following model certification.

Table 48
1% Annual Exceedance Probability Alternative
(2011 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits
Damage Category					
Residential & Commercial - Structure/Content/Vehicles	$ 819.5	$ 123.4	$ 696.2	$ 225.1	$ 921.3
Industrial - Structure/Contents	$ 16.5	$ 1.3	$ 15.2	$ 5.1	$ 20.3
Highways	$ 6.6	$ 2.3	$ 4.3	$ 1.5	$ 5.7
Streets	$ 15.5	$ 2.3	$ 13.3	$ 4.2	$ 17.4
Debris Removal & Cleanup	$ 25.7	$ 4.0	$ 21.7	$ 7.3	$ 29.0
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3
Total	$ 888.9	$ 133.2	$ 755.7	$ 248.6	$ 1,004.3
First Costs					$ 10,177
Interest During Construction					$ 5,864
Annual Operation & Maintenance Costs					$ 7
Total Annual Costs					$ 710
B/C Ratio					1.41
Equivalent Annual Net Benefits Base Year 2035					$ 294

*Total equivalent annual benefits, net benefits and b/c ratio after changes to CSVRs following model certification.

Table 49

3% Annual Exceedance Probability Alternative - Without Future Development

(2011 Price Level; 3.75% Discount Rate)

Total Equivalent Annual Net Benefits

Morganza to the Gulf

Post-Authorization Change Report

($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits	Adjusted Total Equiv Annual Benefits
Damage Category						
Residential & Commercial - Structure/Content/Vehicles	$ 727.2	$ 362.0	$ 365.3	$ 155.4	$ 520.7	
Industrial - Structure/Contents	$ 16.5	$ 6.4	$ 10.1	$ 4.4	$ 14.5	
Highways	$ 6.6	$ 4.2	$ 2.3	$ 1.1	$ 3.4	
Streets	$ 15.5	$ 5.6	$ 9.9	$ 3.7	$ 13.6	
Debris Removal & Cleanup	$ 24.2	$ 11.2	$ 13.1	$ 5.8	$ 18.9	
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2	
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0	
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3	
Total	$ 795.1	$ 389.4	$ 405.7	$ 175.8	$ 581.5	$ 639.5
First Costs						$ 5,902
Interest During Construction						$ 3,937
Annual Operation & Maintenance Costs						$ 6
Total Annual Costs						$ 438
B/C Ratio						1.46
Equivalent Annual Net Benefits - 2035 Base Year						$ 201

*Total equivalent annual benefits, net benefits and b/c ratio were adjusted to reflect changes in CSVRs following model certification.

Table 50
1% Annual Exceedance Probability Alternative - Without Future Development
(2011 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits	Adjusted Total Equiv Annual Benefits
Damage Category						
Residential & Commercial - Structure/Content/Vehicles	$ 747.9	$ 113.9	$ 634.0	$ 207.2	$ 841.2	
Industrial - Structure/Contents	$ 16.5	$ 1.3	$ 15.2	$ 5.1	$ 20.3	
Highways	$ 6.6	$ 2.3	$ 4.3	$ 1.5	$ 5.7	
Streets	$ 15.5	$ 2.3	$ 13.3	$ 4.2	$ 17.4	
Debris Removal & Cleanup	$ 25.7	$ 4.0	$ 21.7	$ 7.3	$ 29.0	
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2	
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0	
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3	
Total	$ 795.1	$ 112.0	$ 683.1	$ 230.8	$ 913.9	$ 993.0
First Costs						$ 10,177
Interest During Construction						$ 5,864
Annual Operation & Maintenance Costs						$ 7
Total Annual Costs						$ 710
B/C Ratio						1.40
Equivalent Annual Net Benefits Base Year 2035						$ 283

*Total equivalent annual benefits, net benefits and b/c ratio were adjusted to reflect changes in CSVRs following model certification.

Table 51
Donaldsonville to the Gulf, LA Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Category Name	Damage Type	Parameter	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
1STY-PIER	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	1.7	2.7	27.6	47.1	54.1	56.4	58.7	63.8	72.2	77.4	79.9	80.3	84.4	87.9	88.6	88.9	89.0	89.3	89.4
			Lower %	0.0	0.0	0.9	3.5	16.0	45.1	47.8	48.4	53.3	61.8	64.3	66.7	69.2	72.2	76.2	76.2	76.2	76.2	76.2	76.2
			Upper %	0.0	3.6	4.0	41.9	50.4	64.5	65.2	69.3	73.2	78.0	92.5	98.2	98.2	102.3	107.9	107.9	108.2	108.2	109.2	109.2
		Contents	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	0.0	0.0	35.0	46.8	48.4	50.3	56.7	67.5	76.3	80.9	88.1	88.4	89.1	89.1	89.1	89.1	89.1	89.1	89.1
			Lower %	0.0	0.0	0.0	27.5	39.9	45.2	48.0	53.0	65.6	74.1	78.5	85.5	85.7	86.4	86.5	86.5	86.5	86.5	86.5	86.5
			Upper %	0.0	0.0	0.0	38.6	50.9	51.2	54.9	59.1	69.5	80.6	85.4	92.9	93.2	93.9	94.0	94.0	94.0	94.0	94.0	94.0
1STY-SLAB	Residential	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	1.0	9.8	31.1	37.7	41.4	44.6	50.0	62.3	67.0	69.4	69.8	74.9	80.7	81.4	81.5	81.5	82.0	82.0	82.0	82.0
			Lower %	0.0	2.8	11.1	28.5	30.9	32.3	41.7	54.5	55.1	56.0	59.0	66.3	70.6	70.6	70.6	70.6	70.6	70.6	70.6	70.6
			Upper %	6.0	41.4	46.6	50.9	50.9	55.8	58.1	77.1	77.1	81.5	81.5	90.3	93.9	97.4	97.4	97.4	97.4	97.4	97.4	97.4
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	35.0	46.8	48.4	50.3	56.7	67.5	76.3	80.9	88.1	88.4	89.1	89.1	89.1	89.1	89.1	89.1	89.1	89.1
			Lower %	0.0	0.0	27.5	39.9	45.2	48.0	53.0	65.6	74.1	78.5	85.5	85.7	86.4	86.5	86.5	86.5	86.5	86.5	86.5	86.5
			Upper %	0.0	0.0	38.6	50.9	51.2	54.9	59.1	69.5	80.6	85.4	92.9	93.2	93.9	94.0	94.0	94.0	94.0	94.0	94.0	94.0
2STY-PIER	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	1.5	2.4	8.3	23.1	28.7	31.5	34.2	35.8	43.3	46.5	47.4	47.7	50.1	63.6	65.6	67.0	69.3	72.1	73.3
			Lower %	0.0	0.0	0.4	0.4	10.8	20.5	27.7	28.0	28.2	32.0	37.8	40.1	40.1	40.1	52.5	56.7	59.5	60.4	63.3	65.0
			Upper %	0.0	3.4	5.6	31.6	36.3	37.6	37.8	47.0	47.3	58.7	58.7	59.3	59.3	61.2	75.7	76.9	80.7	80.8	85.6	91.9
		Contents	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	0.7	21.3	24.8	30.7	34.6	37.7	45.6	50.5	56.7	60.6	61.6	62.3	68.1	68.1	72.0	74.0	75.8	77.0	77.2
			Lower %	0.0	0.0	19.6	22.9	29.7	33.5	36.5	44.2	49.0	54.0	58.7	59.7	60.4	66.0	66.0	69.7	71.7	73.4	74.6	74.8
			Upper %	0.0	1.5	22.1	25.8	31.6	35.6	38.8	46.9	53.9	59.3	64.7	65.7	66.4	72.7	72.7	76.7	78.9	80.8	82.1	82.3
2STY-SLAB	Residential	Structure	Stage	-1.1	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
			Mean %	0.0	0.0	0.0	5.9	19.1	25.3	26.0	30.1	32.1	43.0	46.9	47.5	47.6	52.0	61.9	64.3	65.7	67.6	69.6	69.6
			Lower %	0.0	0.0	0.0	0.0	5.8	17.2	18.1	22.6	25.4	26.6	37.0	37.0	37.0	40.4	55.0	57.6	57.6	60.0	60.4	60.4
			Upper %	0.0	0.0	0.0	34.0	38.3	40.5	40.8	48.3	48.3	65.2	65.2	67.3	67.3	73.0	74.1	75.2	75.2	75.2	75.2	75.2
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	
			Mean %	0.0	0.7	21.3	24.8	30.7	34.6	37.7	45.6	50.5	56.7	60.6	61.6	62.3	68.1	68.1	72.0	74.0	75.8	77.0	
			Lower %	0.0	0.0	19.6	22.9	29.7	33.5	36.5	44.2	49.0	54.0	58.7	59.7	60.4	66.0	66.0	69.7	71.7	73.4	74.6	
			Upper %	0.0	1.5	22.1	25.8	31.6	35.6	38.8	46.9	53.9	59.3	64.7	65.7	66.4	72.7	72.7	76.7	78.9	80.8	82.1	
AUTO	AUTO	Structure	Stage	0.5	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0	17.0		
			Mean %	0.0	8.0	19.0	27.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0		
			Lower %	0.0	4.0	13.0	18.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0		
			Upper %	0.0	10.0	20.0	30.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0		

Table 51 (cont)
Donaldsonville to the Gulf, LA Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Occupancy Type	Category Name	Damage Type	Parameter	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
EAT	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.6	0.6	14.3	18.6	21.4	23.5	26.2	30.9	31.3	33.0	34.6	43.6	51.0	54.1	56.8	60.1	60.4	61.6	61.8
			Lower %	0.0	0.0	0.0	4.0	13.3	15.7	16.1	20.2	23.6	24.0	24.0	24.0	32.6	39.1	46.3	46.9	49.8	50.1	51.4	51.4
			Upper %	0.0	1.5	1.6	21.8	23.0	31.7	33.4	35.2	39.5	39.5	49.3	54.8	61.5	68.0	68.0	68.7	69.9	69.9	70.9	70.9
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	31.2	34.3	60.3	65.0	84.0	91.0	93.4	93.4	96.1	96.1	96.1	96.1	96.1	96.1	96.1	96.1	96.1
			Lower %	0.0	0.0	0.0	28.1	30.9	54.2	58.5	75.6	81.9	84.1	84.1	86.5	86.5	86.5	86.5	86.5	86.5	86.5	86.5	86.5
			Upper %	0.0	0.0	0.0	39.0	42.9	75.4	81.2	89.9	95.8	97.8	97.8	98.3	98.3	98.3	98.3	98.3	98.3	98.3	98.3	98.3
GROC	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.6	0.6	14.3	18.6	21.4	23.5	26.2	30.9	31.3	33.0	34.6	43.6	51.0	54.1	56.8	60.1	60.4	61.6	61.8
			Lower %	0.0	0.0	0.0	4.0	13.3	15.7	16.1	20.2	23.6	24.0	24.0	24.0	32.6	39.1	46.3	46.9	49.8	50.1	51.4	51.4
			Upper %	0.0	1.5	1.6	21.8	23.0	31.7	33.4	35.2	39.5	39.5	49.3	54.8	61.5	68.0	68.0	68.7	69.9	69.9	70.9	70.9
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	85.7	87.3	87.5	88.1	89.6	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	71.2	78.6	78.7	79.3	80.7	88.9	89.9	89.9	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0
			Upper %	0.0	0.0	92.7	94.9	95.7	96.1	97.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
MOBHOM	MOBHOME	Structure	Stage	-1.1	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	10.3	12.3	34.5	62.3	64.1	68.9	71.6	74.7	79.8	80.4	81.7	94.0	94.3	95.4	95.4	95.4	95.4	95.4	95.4
			Lower %	0.0	0.0	0.0	9.6	41.1	47.0	53.3	56.9	58.5	71.0	71.0	71.0	80.9	84.6	84.6	84.6	84.6	84.6	84.6	84.6
			Upper %	0.0	15.4	19.1	55.2	68.2	69.9	74.7	77.8	83.9	84.6	84.6	88.4	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.1	15.0	30.1	45.6	58.8	69.2	78.3	82.4	84.3	84.4	84.4	84.4	84.4	84.4	84.4	84.4	84.4	84.4
			Lower %	0.0	0.0	0.0	14.5	29.1	44.2	57.0	67.1	75.9	79.8	81.7	81.7	81.7	81.7	81.7	81.7	81.7	81.7	81.7	81.7
			Upper %	0.0	0.0	0.1	15.4	30.9	46.9	62.8	73.9	83.6	87.9	89.9	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0	90.0
MULT	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	28.1	32.8	35.2	37.9	39.7	47.2	49.7	52.0	54.6	61.7	64.9	67.8	68.0	70.3	70.4	70.6	70.9
			Lower %	0.0	0.0	0.0	9.2	22.6	28.2	29.1	30.4	43.4	43.7	47.2	47.9	53.3	53.3	60.2	60.3	64.5	64.5	64.5	64.5
			Upper %	0.0	0.0	0.0	38.5	45.7	49.9	51.9	52.6	54.0	58.9	59.1	71.6	78.1	78.3	78.6	78.8	79.0	79.2	79.5	79.7
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	15.2	19.9	22.4	27.7	44.5	48.9	48.9	48.9	48.9	48.9	48.9	64.1	71.3	93.4	97.8	97.8	97.8
			Lower %	0.0	0.0	0.0	12.0	17.0	21.0	28.4	41.6	47.5	47.5	47.5	47.5	47.5	47.5	50.5	66.6	87.4	95.1	95.1	95.1
			Upper %	0.0	0.0	0.0	16.9	21.8	23.6	30.2	46.0	50.1	50.2	51.6	51.6	51.6	51.6	70.9	75.0	96.5	100.0	100.0	100.0
PROF	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	4.1	16.6	18.9	21.2	24.5	27.1	35.4	36.3	36.9	38.0	41.9	51.4	52.9	54.2	54.7	54.8	55.4	55.5
			Lower %	0.0	0.0	0.2	5.3	11.1	15.3	18.7	22.7	27.4	28.0	28.0	28.0	40.1	40.3	46.3	46.3	46.3	46.3	46.3	46.3
			Upper %	0.0	0.0	23.1	25.2	26.2	26.2	36.2	36.6	53.9	53.9	54.4	56.5	65.9	65.9	65.9	65.9	66.3	66.3	69.4	69.4
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	26.0	31.2	34.8	38.2	67.6	86.9	86.9	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0
			Lower %	0.0	0.0	23.4	28.0	31.3	34.4	60.9	78.3	78.3	89.2	89.2	89.2	89.2	89.2	89.2	89.2	89.2	89.2	89.2	89.2
			Upper %	0.0	0.0	32.5	38.9	43.4	47.7	84.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 51 (cont)
Donaldsonville to the Gulf, LA Depth-Damage Relationships for Structures, Contents and Vehicles
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

				-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
PUBL	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.6	0.6	0.6	14.3	18.6	21.4	23.5	26.2	30.9	31.3	33.0	34.6	43.6	51.0	54.1	56.8	60.1	60.4	61.6	61.8
			Lower %	0.0	0.0	0.0	4.0	13.3	15.7	16.1	20.2	23.6	24.0	24.0	24.0	32.6	39.1	46.3	46.9	49.8	50.1	51.4	51.4
			Upper %	0.0	1.5	1.6	21.8	23.0	31.7	33.4	35.2	39.5	39.5	49.3	54.8	61.5	68.0	68.0	68.7	69.9	69.9	70.9	70.9
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	8.6	11.7	13.6	16.6	90.2	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
			Lower %	0.0	0.0	0.0	6.5	8.8	10.2	12.4	67.6	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0
			Upper %	0.0	0.0	0.0	9.5	12.8	15.0	18.2	92.8	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
REPA	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	28.1	32.8	35.2	37.9	39.7	47.2	49.7	52.0	54.6	61.7	64.9	67.8	68.0	70.3	70.4	70.6	70.9
			Lower %	0.0	0.0	0.0	9.2	22.6	28.2	29.1	30.4	43.4	43.7	47.2	47.9	53.3	53.3	60.2	60.3	64.5	64.5	64.5	64.5
			Upper %	0.0	0.0	0.0	38.5	45.7	49.9	51.9	52.6	54.0	58.9	59.1	71.6	78.1	78.3	78.6	78.8	79.0	79.2	79.5	79.7
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	32.9	33.7	33.7	63.9	66.0	68.0	73.0	76.4	76.4	76.4	76.4	76.4	76.4	76.4	76.4	76.4	76.4
			Lower %	0.0	0.0	0.0	29.7	30.3	30.3	57.5	59.4	61.2	65.8	68.7	68.7	68.7	68.7	68.7	68.7	68.7	68.7	68.7	68.7
			Upper %	0.0	0.0	0.0	41.2	42.1	42.1	79.8	82.5	85.0	91.3	95.5	95.5	95.5	95.5	95.5	95.5	95.5	95.5	95.5	95.5
RETA	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.6	0.6	0.6	14.3	18.6	21.4	23.5	26.2	30.9	31.3	33.0	34.6	43.6	51.0	54.1	56.8	60.1	60.4	61.6	61.8
			Lower %	0.0	0.0	0.0	4.0	13.3	15.7	16.1	20.2	23.6	24.0	24.0	24.0	32.6	39.1	46.3	46.9	49.8	50.1	51.4	51.4
			Upper %	0.0	1.5	1.6	21.8	23.0	31.7	33.4	35.2	39.5	39.5	49.3	54.8	61.5	68.0	68.0	68.7	69.9	69.9	70.9	70.9
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	55.0	66.0	77.0	88.0	89.9	91.9	93.8	99.6	99.6	99.6	99.6	99.6	99.6	99.6	99.6	99.6	99.6
			Lower %	0.0	0.0	0.0	49.4	59.4	69.1	79.2	81.0	82.6	84.4	89.7	89.7	89.7	89.7	89.8	89.8	89.8	89.8	89.8	89.8
			Upper %	0.0	0.0	0.0	63.1	75.8	88.6	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
WARE	COM	Structure	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	28.1	32.8	35.2	37.9	39.7	47.2	49.7	52.0	54.6	61.7	64.9	67.8	68.0	70.3	70.4	70.6	70.9
			Lower %	0.0	0.0	0.0	9.2	22.6	28.2	29.1	30.4	43.4	43.7	47.2	47.9	53.3	53.3	60.2	60.3	64.5	64.5	64.5	64.5
			Upper %	0.0	0.0	0.0	38.5	45.7	49.9	51.9	52.6	54.0	58.9	59.1	71.6	78.1	78.3	78.6	78.8	79.0	79.2	79.5	79.7
		Contents	Stage	-1.0	-0.5	0.0	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			Mean %	0.0	0.0	0.0	15.6	19.4	19.4	26.8	34.1	41.6	49.0	56.5	63.9	71.4	75.2	75.2	75.2	75.2	75.2	75.2	75.2
			Lower %	0.0	0.0	0.0	14.1	17.5	17.5	24.1	30.6	37.4	44.1	50.9	57.6	64.2	67.6	67.6	67.6	67.6	67.6	67.6	67.6
			Upper %	0.0	0.0	0.0	19.6	24.4	24.4	33.5	42.7	51.9	61.2	70.6	79.9	89.1	93.9	93.9	93.9	93.9	93.9	93.9	93.9

Source: Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios (CSVRs) in Support of the Donaldsonville to the Gulf, Louisiana, Feasibility Study Final Report dated March 2006
Note: Long duration, saltwater depth-damage relationships were used for the sensitivity analysis

Table 52
3% Annual Exceedance Probability Alternative Using Donaldsonville to the Gulf Depth-Damage Relationships
(2011 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Sensitivity Analysis
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits	Adjusted Total Equiv Annual Benefits
Damage Category						
Residential & Commercial - Structure/Content/Vehicles	$ 615.2	$ 319.0	$ 296.3	$ 126.4	$ 422.6	
Industrial - Structure/Contents	$ 16.5	$ 6.4	$ 10.1	$ 4.4	$ 14.5	
Highways	$ 6.6	$ 4.2	$ 2.3	$ 1.1	$ 3.4	
Streets	$ 15.5	$ 5.6	$ 9.9	$ 3.7	$ 13.6	
Debris Removal & Cleanup	$ 25.7	$ 12.6	$ 13.1	$ 5.9	$ 19.0	
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2	
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0	
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3	
Total	$ 684.6	$ 347.9	$ 336.7	$ 146.8	$ 483.6	$ 532
First Costs						$ 5,902
Interest During Construction						$ 3,937
Annual Operation & Maintenance Costs						$ 6
Total Annual Costs						$ 438
B/C Ratio						1.21
Equivalent Annual Net Benefits - 2035 Base Year						$ 94

*Total equivalent annual benefits, net benefits and b/c ratio were adjusted to reflect changes in M2G CSVRs following model certification.

Table 53
1% Annual Exceedance Probability Alternative Using Donaldsonville to the Gulf Depth-Damage Relationships
(2011 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Sensitivity Analysis
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits	
Damage Category						
Residential & Commercial - Structure/Content/Vehicles	$ 615.2	$ 98.5	$ 516.7	$ 166.1	$ 682.8	
Industrial - Structure/Contents	$ 16.5	$ 1.3	$ 15.2	$ 5.1	$ 20.3	
Highways	$ 6.6	$ 2.3	$ 4.3	$ 1.5	$ 5.7	
Streets	$ 15.5	$ 2.3	$ 13.3	$ 4.2	$ 17.4	
Debris Removal & Cleanup	$ 25.7	$ 4.0	$ 21.7	$ 7.3	$ 29.0	
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2	
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0	
Avoided Structure-Raising Costs	$ 4.9	$ -	$ 4.9	$ 5.3	$ 10.3	
Total	$ 684.6	$ 108.4	$ 576.2	$ 189.6	$ 765.8	$ 832
First Costs						$ 10,177
Interest During Construction						$ 5,864
Annual Operation & Maintenance Costs						$ 7
Total Annual Costs						$ 710
B/C Ratio						1.17
Equivalent Annual Net Benefits Base Year 2035						
*Total equivalent annual benefits, net benefits and b/c ratio were adjusted to reflect changes in M2G CSVRs following model certification.						$ 122

Table 54
3% Annual Exceedance Probability Alternative
(2012 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits
Damage Category					
Residential & Commercial - Structure/Content/Vehicles	$ 834.3	$ 417.3	$ 417.0	$ 176.9	$ 593.9
Industrial - Structure/Contents	$ 16.8	$ 6.5	$ 10.3	$ 4.5	$ 14.8
Highways	$ 6.9	$ 4.5	$ 2.5	$ 1.1	$ 3.6
Streets	$ 16.5	$ 6.0	$ 10.5	$ 3.9	$ 14.4
Debris Removal & Cleanup	$ 26.2	$ 12.8	$ 13.3	$ 6.0	$ 19.3
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0
Avoided Structure-Raising Costs	$ 5.0	$ -	$ 5.0	$ 5.4	$ 10.4
Total	$ 905.8	$ 447.1	$ 458.7	$ 197.9	$ 656.6
First Costs					$ 5,950
Interest During Construction					$ 3,981
Annual Operation & Maintenance Costs					$ 6
Total Annual Costs					$ 442
B/C Ratio					1.48
Equivalent Annual Net Benefits - 2035 Base Year					214

Note: The following indexes were used to update the benefit categories and cost values from 2011 to 2012: the Construction Index developed by the Bureau of Labor Statistics was used for residential and non-residential benefit categories, including the industrial benefit category, and the avoided structure-raising costs category; the National Highway Construction Cost Index was used for the highway and streets benefit categories; the Remediation Services Index developed by the Bureau of Labor Statistics was used for the debris removal and cleanup benefit category and the Diesel Fuel Price Index developed by the Energy Information Administration was used for boat fleets benefit category.
Costs were calculated to reflect 2012 prices.

Table 55
1% Annual Exceedance Probability Alternative
(2012 Price Level; 3.75% Discount Rate)
Total Equivalent Annual Net Benefits
Morganza to the Gulf
Post-Authorization Change Report
($ Millions)

Item	Equiv Annual W/O Project Damages (2035-2084)	Equiv Annual With-Project Damages (2035-2084)	Equiv Annual Benefits (2035-2084)	Equiv Annual Benefits During Construction (2024-2034)	Total Equiv Annual Benefits
Damage Category					
Residential & Commercial - Structure/Content/Vehicles	$ 834.3	$ 125.6	$ 708.7	$ 229.1	$ 937.8
Industrial - Structure/Contents	$ 16.8	$ 1.3	$ 15.5	$ 5.2	$ 20.7
Highways	$ 6.9	$ 2.4	$ 4.5	$ 1.6	$ 6.1
Streets	$ 16.5	$ 2.4	$ 14.1	$ 4.4	$ 18.5
Debris Removal & Cleanup	$ 26.2	$ 4.1	$ 22.1	$ 7.4	$ 29.5
Water Supply	$ 0.1	$ 0.1	$ 0.1	$ 0.1	$ 0.2
Boat Fleets	$ 0.0	$ 0.0	$ 0.0	$ 0.0	$ 0.0
Avoided Structure-Raising Costs	$ 5.0	$ -	$ 5.0	$ 5.4	$ 10.4
Total	$ 905.8	$ 135.8	$ 770.0	$ 253.3	$ 1,023.3
First Costs					$ 10,265
Interest During Construction					$ 5,914
Annual Operation & Maintenance Costs					$ 7
Total Annual Costs					$ 716
B/C Ratio					1.43
Equivalent Annual Net Benefits Base Year 2035					$ 307

Note: The following indexes were used to update the benefit categories and cost values from 2011 to 2012: the Construction Index developed by the Bureau of Labor Statistics was used for residential and non-residential benefit categories, including the industrial benefit category, and the avoided structure-raising costs category; the National Highway Construction Cost Index was used for the highway and streets benefit categories; the Remediation Services Index developed by the Bureau of Labor Statistics was used for the debris removal and cleanup benefit category and the Diesel Fuel Price Index developed by the Energy Information Administration was used for boat fleets benefit category.
Costs were calculated to reflect 2012 prices.

Table 56
Equivalent Annual Damages and Benefits (1000's)
(2024-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

| Plan Name | Equivalent Annual Damage | | | Probability Damage Reduced Exceeds Indicated Values (Forecasted) | | | | | |
	Total Without Project	Total With Project	Damages Reduced	1.00	0.75	0.71	0.69	0.50	0.25
Alt 3%	$ 1,267,403	581,641	585,856	$ 248,478	$ 413,212	$ 438,252	$ 452,748	$ 577,946	$ 727,617
Alt 1%	$ 1,267,403	$ 243,226	$ 924,272	$ 275,863	$ 623,134	$ 678,697	$ 710,342	$ 970,405	$ 1,288,282

Note: Based on CSVR values after model certification and 2011 prices.

Notes 1: A trend function was applied to estimate the forecasted damaged reduced values above 0.75.
      2: Highlighted values represent the equivalent annual cost (1,000s) of each alternative.
      3: The 3% AEP has a 71 percent chance of having postivite net benefits.
      4: The 1% AEP has a 69 percent chance of having postivite net benefits.
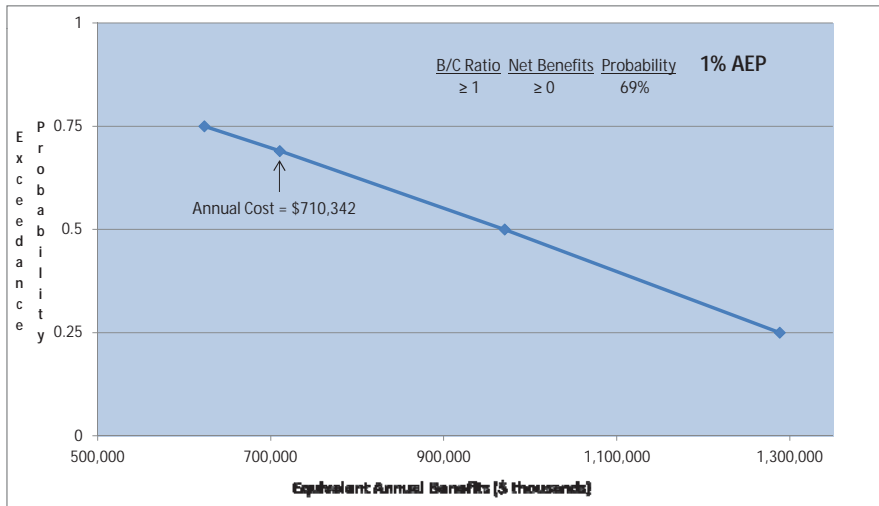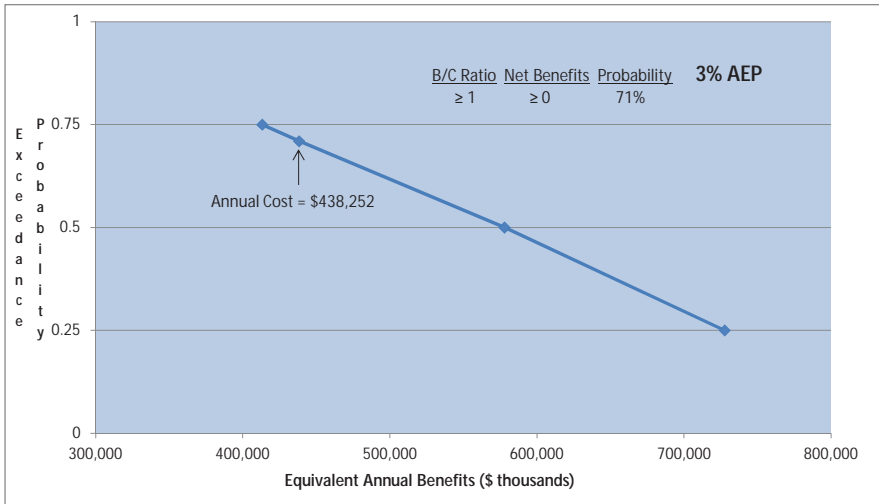
Table 57
Residual Risk for Total Study Area in 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

ACE	Without Project		3% Alt	1% Alt
	Number of Structures Damaged assuming Non-Federal Levees Fail (No Autos)	Damages assuming Non-Federal Levees Fail ($1,000s)	Residual Damages assuming Federal Levees do not Fail ($1,000s)	Residual Damages assuming Federal Levees do not Fail ($1,000s)
0.99	260	$ 17,341	$ -	$ -
0.20	1,120	$ 34,788	$ -	$ -
0.10	5,585	$ 270,082	$ -	$ -
0.04	34,943	$ 4,308,716	$ -	$ -
0.02	48,362	$ 9,893,168	$ 12,011,222	$ -
0.01	58,836	$ 12,266,509	$ 14,835,662	$ -
0.005	61,166	$ 14,947,946	$ 15,900,216	$ 15,900,216
0.002	61,387	$ 16,095,073	$ 16,708,703	$ 16,708,703

Note: It should be noted that the residual risk will be higher with the project alternatives in place relative to the without-project conditions since structures below the 10% ACE (10-year) event are elevated above the 1% ACE (100-year) event to account for the response of residents to repetitive flood loss. Also, the residual damages under with-project conditions are higher since the structure detail output file produced by the HEC-FDA model uses the exterior water surface profiles to the Federal levee to show damages under with-project conditions and interior water surface profiles to the non-Federal levee to show damages under without project conditions.

Source: HEC-FDA model Structure Detail output file for the year 2035.

Table 58
Total Study Area Depth of Flooding for the 1% ACE (100-year) Event in 2035
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Depth of Flooding in Feet	Number of Structures*	Percentage by Bin
0	11,679	19.00%
3	7,384	12.02%
6	21,980	35.77%
9	18,396	29.94%
12	1,827	2.97%
>12	187	0.30%

Total Number of Structures		61,453
Total Number of Structures that have a depth of flooding >0		49,773

* No Automobiles

Number of Structures*

■ Number of Structures*
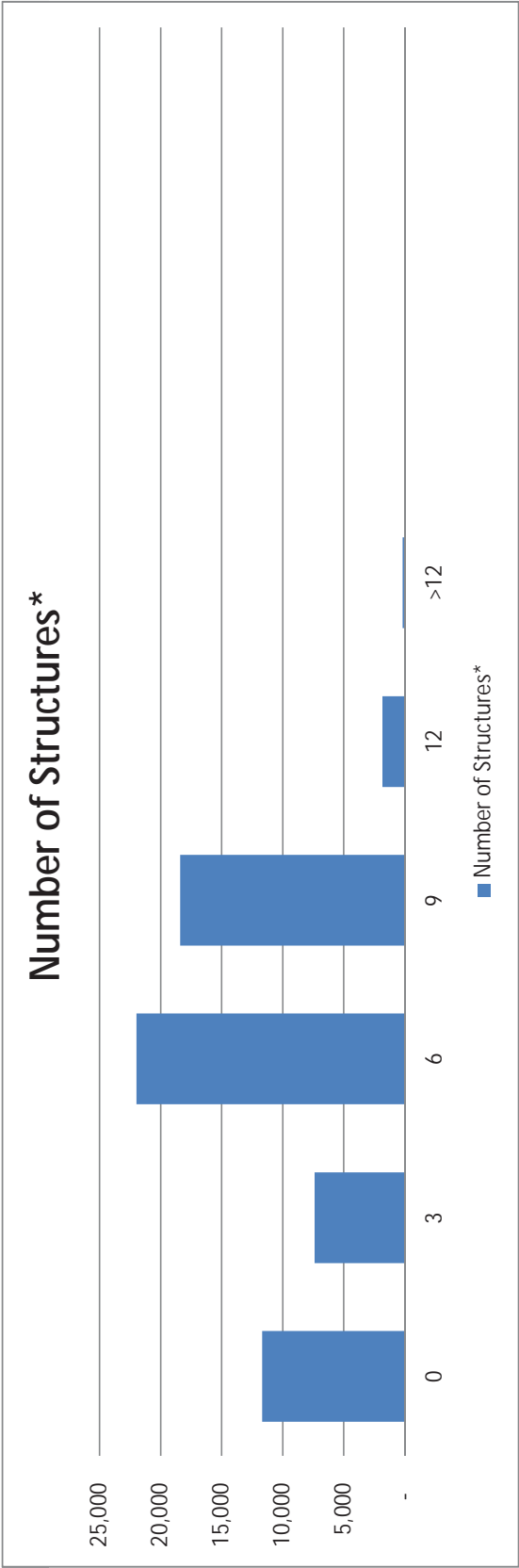
Table 59
Levee Performance Annual Exceedance Probability
(2010-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Study Area Reach 11BW79 Station 64

2010

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.1068	0.1054	0.6719	0.9647	0.9962	0.8864	0.2528	0.0864	0.0441	0.0233	0.0153

2024

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.1147	0.1136	0.7005	0.9731	0.9976	0.8423	0.2053	0.0637	0.0292	0.0150	0.0098
3%	9.4	0.0254	0.0271	0.2399	0.5609	0.7463	0.9998	0.7857	0.3715	0.1786	0.0953	0.0509
1%	13.6	0.0072	0.0104	0.0997	0.2702	0.4084	0.9998	0.9915	0.8250	0.5631	0.3645	0.2204

2035

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.1196	0.1190	0.7184	0.9579	0.9982	0.7955	0.1743	0.0492	0.0206	0.0106	0.0066
3%	10.5	0.0237	0.0256	0.2287	0.4776	0.7269	0.9997	0.7959	0.4132	0.2097	0.1107	0.0628
1%	15.9	0.0040	0.0072	0.0697	0.1654	0.3034	0.9997	0.9976	0.9156	0.7153	0.4977	0.3305

2085

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.2231	0.2193	0.9159	0.9994	1.0000	0.3740	0.0564	0.0195	0.0104	0.0054	0.0042
3%	11.2	0.0307	0.0330	0.2849	0.6343	0.8130	0.9994	0.6656	0.2913	0.1279	0.0546	0.0310
1%	18.3	0.0033	0.0052	0.0505	0.1441	0.2284	0.9998	0.9998	0.9764	0.8291	0.5569	0.3822

Note: Non-Federal Levee has 7 foot top of levee elevation under without-project conditions.

Table 59 (Cont.)
Levee Performance Annual Exceedance Probability
(2010-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Study Area Reach 11BW5 Station 58

2010

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0663	0.0683	0.5072	0.8803	0.9709	0.8226	0.2587	0.0509	0.0245	0.0129	0.0084

2024

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0842	0.0827	0.5782	0.9249	0.9866	0.7681	0.1777	0.0423	0.0191	0.0096	0.0060
3%	9.4	0.0253	0.0269	0.2389	0.5591	0.7446	0.9997	0.7993	0.3681	0.1727	0.0890	0.0455
1%	13.6	0.0072	0.0103	0.0987	0.2679	0.4053	0.9997	0.9930	0.8309	0.5654	0.3608	0.2132

2035

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.1191	0.1200	0.7214	0.9590	0.9983	0.7131	0.1369	0.0351	0.0137	0.0067	0.0039
3%	10.5	0.0237	0.0256	0.2286	0.4774	0.7269	0.9997	0.7959	0.4132	0.2097	0.1107	0.0628
1%	15.9	0.0040	0.0072	0.0696	0.1650	0.3029	0.9997	0.9976	0.9156	0.7153	0.4977	0.3305

2085

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.3564	0.3460	0.9857	1.0000	1.0000	0.2489	0.0384	0.0128	0.0062	0.0029	0.0031
3%	11.2	0.0307	0.0327	0.2832	0.6317	0.8108	0.9993	0.6693	0.2954	0.1304	0.0566	0.0327
1%	18.3	0.0033	0.0052	0.0505	0.1441	0.2284	0.9998	0.9764	0.8291	0.5569	0.3822	0.3822

Note: Non-Federal Levee has 5.5 foot top of levee elevation under without-project conditions.

Table 59 (Cont.)
Levee Performance Annual Exceedance Probability
(2010-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Study Area Reach 1-5 Station 82

2010

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.1863	0.1786	0.8602	0.9973	0.9999	0.3185	0.0524	0.0145	0.0089	0.0051	0.0036

2024

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.4063	0.3854	0.9923	1.0000	1.0000	0.1626	0.0259	0.0106	0.0054	0.0028	0.0020
3%	14.8	0.0159	0.0184	0.1693	0.2890	0.4336	0.9997	0.9851	0.8042	0.5544	0.3133	0.1978
1%	17.2	0.0026	0.0048	0.0467	0.1336	0.2126	0.9998	0.9997	0.9712	0.8476	0.6097	0.4355

2035

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.7554	0.7538	1.0000	1.0000	1.0000	0.0025	0.0052	0.0007	0.0027	0.0014	0.0001
3%	16.2	0.0074	0.0101	0.9690	0.2250	0.3993	0.9997	0.9919	0.8486	0.5902	0.3356	0.2126
1%	20.5	0.0015	0.0031	0.0302	0.0737	0.1419	0.9998	0.9998	0.9941	0.9310	0.7404	0.5635

2085

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0904	0.0939	0.6270	0.9481	0.9928	0.6308	0.1048	0.0340	0.0166	0.0092	0.0059
3%	15.8	0.0159	0.0184	0.1693	0.4267	0.6044	0.9996	0.9277	0.6095	0.3093	0.1442	0.0843
1%	21.8	0.0023	0.0040	0.0394	0.1136	0.1820	0.9998	0.9997	0.9909	0.8875	0.6450	0.4635

Note: Non-Federal Levee has 3.0 foot top of levee elevation under without-project conditions.

Table 59 (Cont.)
Levee Performance Annual Exceedance Probability
(2010-2085)
Morganza to the Gulf of Mexico, LA
Post-Authorization Change Report

Study Area Reach BL89 Station 298

2010

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0529	0.0545	0.4289	0.8137	0.9392	0.9978	0.1924	0.0384	0.0107	0.0000	0.0000

2024

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0691	0.0717	0.5247	0.8926	0.9757	0.8180	0.1383	0.0415	0.0177	0.0060	0.0027
3%	14.7	0.0111	0.0135	0.1270	0.3347	0.4929	0.9997	0.9778	0.7508	0.4534	0.2328	0.1417
1%	17.3	0.0050	0.0074	0.0714	0.1992	0.3095	0.9997	0.9989	0.9330	0.7091	0.4313	0.2776

2035

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.0899	0.0866	0.5956	0.8960	0.9892	0.5336	0.2005	0.0509	0.0111	0.0017	0.0000
3%	15.1	0.0130	0.0154	0.1436	0.3213	0.5394	0.9996	0.9623	0.6913	0.3948	0.2002	0.1234
1%	20.3	0.0027	0.0046	0.0453	0.1094	0.2068	0.9997	0.9997	0.9821	0.8542	0.6016	0.4254

2085

Plan Name	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
		Median	Expected	10	30	50	0.1000	0.0400	0.0200	0.0100	0.0040	0.0020
Without	Levee	0.2475	0.2358	0.9324	0.9997	1.0000	0.1590	0.0275	0.0027	0.0000	0.0000	0.0000
3%	14.4	0.0250	0.0272	0.2411	0.5630	0.7483	0.9996	0.7727	0.3886	0.1808	0.0857	0.0529
1%	21.8	0.0031	0.0052	0.0509	0.1450	0.2298	0.9998	0.9997	0.9738	0.8181	0.5638	0.4026

Note: Non-Federal Levee has 5.0 foot top of levee elevation under without-project conditions.

Regional Economic Development Appendix
Morganza to the Gulf of Mexico, La

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Definitions

Gross Regional Product: total economic activity in the study area during the model year as measured by either production value of final goods and services (final demand) or income generation to factors of production (value added).

Employment: average annual jobs, both full and part time, not full time equivalents.

Labor Income: all forms of employment income, including employee compensation (wages and benefits) and proprietor income.

Output: represents the value of industry production, includes both value added and intermediate goods purchased in the economy.

Direct Effects: the response (change in employment, income, output, or gross regional product) for a given industry to a change in its final demand.

Indirect Effects: the impacts caused by industries purchasing from other industries in response to final demand changes, a multiplier effect.

Induced Effects: the impacts on all local industries caused by the expenditures of new household income generated by the direct and indirect effects of final demand changes, a multiplier effect.

<u>Background:</u>

The Morganza to the Gulf of Mexico hurricane risk reduction system is located in the parishes of Terrebonne and Lafourche.   The project is being constructed in response to reoccurring hurricane storm damage and is designed to prevent the loss of life, to reduce flood damages, to reduce negative impacts on navigation, and to prevent the destruction of wetlands. This flood risk reduction system would encompass an estimated 120,000 people.

<u>Construction Alternatives:</u>

For this analysis two construction alternatives are being examined: One alternative has a levee height that reduces flood risk up to the 1% annual exceedance probability (AEP) flood event; the other alternative has a levee height that reduces flood risk up to the 3% AEP flood event. This 3% AEP alternative was originally the 1% AEP alternative according to the pre-Katrina requirements for levee construction.

<u>Study Area</u>

The study area of Morganza to the Gulf consists of 20 parishes that were selected by the RECONS project team based on the labor market, commuter-shed, and population centers serving the project location (see table 1). According to RECONS' 2009 data, the population of the study area is 2,199,734. The number of households is 816,005. Total personal income is $90,517,000,000. The employment rate in the study area is 91%. The other region identified is the rest of Louisiana and consists of every other parish except for the ones in the study area.[1]

<u>Methodology:</u>

This Regional Economic Development (RED) analysis employs input-output economic analysis, which measures the interdependence among industries and workers in an economy. This analysis uses a matrix representation of a region's economy to predict the effect of changes in one industry on others. The greater the interdependence among industry sectors, the larger the multiplier effect on the economy. Changes to government spending drive the input-output model to project new levels of sales (output), value added (GRP), employment, and income for each industry.

The specific input-output model used in this analysis is RECONS (Regional Economic System). This model was developed by the Institute for Water Resources (IWR), Michigan State University, and the Louis Berger Group. RECONS uses industry multipliers derived from the commercial input-output model IMPLAN to estimate the effects that spending on USACE projects has on a regional economy. The model is linear and static, showing relationships and impacts at a certain fixed point in time. Spending impacts are composed of three different effects: direct, indirect, and induced.

Direct effects represent the impacts the new federal expenditures have on industries which directly support the new project. Labor and construction materials can be considered direct components

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[1] These metrics are current as of 2009, the year of the data used in the model.

to the project. Indirect effects represent changes to secondary industries that support the direct industries. Induced effects are changes in consumer spending patterns caused by the change in employment and income within the industries affected by the direct and induced effects. The additional income workers receive via a project may be spent on clothing, groceries, dining out, and other items in the regional area.

The inputs for the RECONS model are expenditures that are entered by work activity or industry sector, each with its own unique production function.   For construction, the following work activities were identified: construction and major repairs of floodwalls, construction and major repair of earthen levees, construction activities for ecosystem and habitat restoration, lock or dam gate fabrication and installation, and lock construction of on-site features. For preconstruction, engineering, and design (PE&D), sector 369 engineering services was selected. For Supervision and administration (S&A), sector 386, business support services was selected. For pipeline relocation, sector 39 repair and maintenance construction activities was selected. And for environmental mitigation, the work activity remediation activities and services was selected.[2] The baseline data used by RECONS to represent the regional economy of Louisiana are annual averages from the Bureau of the Census, the Bureau of Labor Statistics, and the Bureau of Economic Analysis for the year 2009. The model results are expressed in 2011 dollars.

Assumptions

Input-output analysis rests on the following assumptions. The production functions of industries have constant returns to scale, so if output is to increase, inputs will increase in the same proportion. Industries face no supply constraints; they have access to all the materials they can use. Industries have a fixed commodity input structure; they will not substitute any commodities or services used in the production of output in response to price changes. Industries produce their commodities in fixed proportions, so an industry will not increase production of a commodity without increasing production in every other commodity it produces. Furthermore, it is assumed that industries use the same technology to produce all of its commodities. Finally, since the model is static, it is assumed that the economic conditions of 2009, the year of the socio-economic data in the RECONS model database, will prevail during the years of the construction process.

Column Descriptions for Tables 1 and 2

"Total Construction Stimulus" is the sum of all inputs including construction, preconstruction, engineering, and design (PED), supervision and administration (S&A), utility relocation, and environmental mitigation. "Output" is the sum total of transactions that take place as a result of the construction project, including both value added and intermediate goods purchased in the economy. "Labor Income" includes all forms of employment income, including employee compensation (wages and benefits) and proprietor income. "Gross Regional Product (GRP)" is the value-added output of the study regions. This metric captures all final goods and services produced in the study areas because of

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[2] Real Estate transactions are considered a transfer of an asset resulting in no multiplier effects, and, therefore, are not included in this RED analysis.

the project's existence. It is different from output in the sense that one dollar of a final good or service may have multiple transactions associated with it. "Employment" is the estimated worker-years of labor required to build the project.

Results

For the 3% AEP alternative, the construction stimulus of $5,897,000,000 would generate 85,000 worker-years of labor, $4,239,000,000 in labor income, $8,839,000,000 in output, and $5,802,000,000 in Gross Regional Product (see table 2). For the 1% AEP alternative, the construction stimulus of $9,819,915,000 would generate 155,000 worker-years of labor, $7,654,000,000 in labor income, $15,254,000,000 in output, and $10,243,000,000 in Gross Regional Product (see table 2).

In the remaining parishes for the 3% AEP alternative, the construction stimulus of $5,897,000,000 would generate 1,600 worker-years of labor, $40,000,000 in labor income, $151,000,000 in output, and $61,000,000 in Gross Regional Product. For the 1% AEP alternative, the construction stimulus of $9,819,915,000 would generate 3,300 worker-years of labor, $83,000,000 in labor income, $303,000,000 in output, and $124,000,000 in Gross Regional Product (see table 2). The annual regional impacts of constructing the hurricane risk reduction system will accrue to the impact areas in amounts proportional to the level of spending for each year of construction.

For both alternatives, the secondary effects, the combined indirect and induced multiplier effects, account for 45% of the total output, about 35% of employment, about 33% of labor income, and 41% of gross regional product in the project area. The study area captures about 85% of the direct spending on the project. The remaining 15% of spending leaks out of the study area (see table 3).

Summary

The construction of the Morganza to the Gulf of Mexico levee system would yield significant increases in employment and gross regional product not only to the parishes of Terrebonne and Lafourche, but to Metro New Orleans and beyond. The 3% annual exceedance probability alternative would generate an estimated $5.8 billion in gross regional product and 85,000 worker-years of labor during the construction of the levee system. The 1% annual exceedance probability alternative would generate an estimated $10.2 billion in gross regional product and 155,000 worker-years of labor.

It should be noted that the costs used to generate the regional economic impacts of construction are in 2011 prices, and, so, the monetary impacts presented are also in 2011 prices. The model was not rerun with the costs in 2012 prices. However, the change in the regional benefits resulting from the price-level change from 2011 prices to 2012 prices would be proportional to the change in the benefits of the NED analysis.

Table 1
Study Area Summary
Morganza to the Gulf of Mexico, La

Parish	Area (sq. mi)	Population	Households	Total Personal Income (in millions)
Ascension	303	104,702	37,280	$3,916
Assumption	365	23,632	8,552	$799
East Baton Rouge	469	429,211	166,068	$18,149
East Feliciana	456	21,057	6,827	$695
Iberville	653	32,987	10,770	$1,035
Jefferson	496	439,261	169,681	$19,446
Lafourche	1,177	93,768	33,790	$3,954
Livingston	703	122,404	43,929	$3,848
Orleans	349	326,968	124,294	$15,261
Plaquemines	1,041	27,039	9,364	$895
Pointe Coupee	591	23,137	8,750	$784
St Bernard	488	29,365	11,218	$1,224
St Charles	410	53,810	18,475	$1,969
St Helena	410	10,582	4,004	$336
St James	258	22,227	7,460	$689
St John The Baptist	348	48,996	16,546	$1,618
St Tammany	1,110	240,775	87,796	$10,406
Terrebonne	1,480	111,202	38,980	$4,268
West Baton Rouge	205	23,108	8,375	$805
West Feliciana	426	15,503	3,846	$421
Total	**11,737**	**2,199,734**	**816,005**	**$90,517**

Source: RECONS Database (2009)

Table 2
Summary of Regional Economic Impacts
Morganza to the Gulf of Mexico, La
(Dollars are in Thousands)

Project Area

Alternative	Construction Stimulus	Employment	Labor Income	Output	Gross Regional Product
3% AEP	$5,897,800	85,000	$4,239,000	$8,839,000	$5,802,000
1% AEP	$9,819,915	155,000	7,654,000	15,254,000	10,243,000

The Remaining Parishes in LA

Alternative	Construction Stimulus	Employment	Labor Income	Output	Gross Regional Product
3% AEP	$5,897,800	1,600	$40,000	$151,000	$61,000
1% AEP	$9,819,915	3,300	83,000	303,000	124,000

Notes: 1. October 2011 Price level
2. Construction Stimulus reflects costs estimates as of October 25, 2012.

Table 3
Direct and Secondary Effects of Expenditures
Morganza to the Gulf PAC
(Dollars are in 000s)

Project Area
3% AEP Alternative

Effects	Gross Regional Product	Output	Labor Income	Employment
Direct	3,411,000	4,892,000	2,828,000	55,000
Secondary	2,391,000	3,947,000	1,411,000	30,000
Total	5,802,000	8,839,000	4,239,000	85,000

1% AEP Alternative

Effects	Gross Regional Product	Output	Labor Income	Employment
Direct	6,090,000	8,394,000	5,212,000	103,000
Secondary	4,153,000	6,859,000	2,442,000	52,000
Total	10,243,000	15,253,000	7,654,000	155,000

The Remaining Parishes
3% AEP Alternative

Effects	Gross Regional Product	Output	Labor Income	Employment
Direct	3,429,000	6,913,000	2,599,000	50
Secondary	58,000	146,000	38,000	1,600
Total	3,487,000	7,059,000	2,637,000	1,650

1% AEP Alternative

Effects	Gross Regional Product	Output	Labor Income	Employment
Direct	5,000	10,000	4,000	70
Secondary	119,000	293,000	79,000	3,000
Total	124,000	303,000	83,000	3,070

Note: The secondary effects include the indirect and induced multiplier effects.

MORGANZA, LOUISIANA, TO THE GULF OF MEXICO POST AUTHORIZATION CHANGE REPORT (PAC)

OTHER SOCIAL EFFECTS APPENDIX

I. INTRODUCTION

This appendix presents a socioeconomic evaluation of the alternatives being considered for storm surge risk reduction for the Morganza to the Gulf evaluation area, which includes portions of two parishes in the state of Louisiana. It was prepared in accordance with Engineering Regulation (ER) 1105-2-100, Planning Guidance Notebook, ER 1105-2-101, Planning Guidance, Risk Analysis for Flood Damage Reduction Studies, and Engineering Circular (EC) 1105-2-409.

Purpose

The purpose of this appendix is to describe the Other Social Effects (OSE) account of the Morganza to the Gulf of Mexico Post-Authorization Change (PAC) Hurricane Protection Project. The OSE account considers the potential social ramifications of Corps actions so that decision makers and stakeholders are able to evaluate the social implications of each alternative and choose an alternative that will be judged as complete, effective, and fair.

Study Area

The Morganza to the Gulf PAC study area is located in coastal Louisiana approximately 60 miles southwest of the city of New Orleans and includes all of Terrebonne Parish and the portion of Lafourche Parish to the south and west of Bayou Lafourche. Communities located within the study area include the city of Houma, the towns of Chauvin, Dulac, and Montegut in southern Terrebonne Parish, the towns of Donner and Gibson in western Terrebonne Parish, and the towns of Gray and Schriever in northern Terrebonne Parish. Also included are the towns of Raceland, Lockport, and Pointe aux Chenes in Lafourche Parish and the portion of the city of Thibodaux south of Bayou Lafourche. Both parishes have historically suffered extensive hurricane and tropical storm damage due to insufficient flood control features. The impact of preparing for, mitigating, and recovering from these damages has placed a significant physical and emotional burden on individuals and has been devastating for communities. The goals of the proposed project are to provide protection to residents within the study area from the damaging effects of storm surges while also protecting and preserving the fragile and rapidly deteriorating coastal wetlands.

Overview of Other Social Effects

While federal water resources planning guidance has long called for an examination of the social effects associated with USACE water resources planning projects, the tendency has historically been to discount the social impacts of Corps projects during the planning process and focus instead on the economic analysis (USACE, 2008). EC 1105-2-409, however, states that "all Corps planning studies will evaluate, display and compare the full range of alternative plans' effects across all four Principles and Guidelines' accounts (National Economic Development

(NED), Regional Economic Development (RED), Environmental Quality (EQ), and Other Social Effects (OSE)" (USACE, 2008 pg. 4).

The OSE account ensures that adequate attention is paid to the beneficial and adverse social effects of Corps' projects during the planning process. This appendix follows the guidance set forth by the USACE Institute for Water Resources (IWR) in the *Handbook on Applying "Other Social Effects" Factors in Corps of Engineers Water Resources Planning* (USACE, 2008). The handbook describes the procedures for analyzing and using OSE criteria in the planning process and identifies social factors that affect individual and group definitions of satisfaction and well-being.

Organization of Appendix

The OSE appendix is organized as follows:

- Section 1 provides an introduction to OSE.
- Section 2 provides a description of the existing and future without-project socioeconomic characteristics and other social factors of the study area.
- Section 3 provides an OSE analysis of the project alternatives.

II. OTHER SOCIAL EFFECTS STUDY AREA CHARACTERISTICS

This section provides a description of the existing and future without-project socioeconomic characteristics and other social factors of the study area.

Socioeconomic Characteristics of the Study Area

In this section, socioeconomic data for Lafourche and Terrebonne Parishes are presented in order to provide a context from which to evaluate the potential social impacts of the proposed project.

Population and Households.

Population characteristics such as size and change constitute important areas of consideration in that they determine consumption patterns, land use activities, and future development patterns. Table 1 displays the population in each of the parishes for the years 1970, 1980, 1990, 2000, and 2010 (study year), as well as projections for the year 2035 and the year 2085, the two years that were modeled by Hydraulics and Hydrology Branch (H&H) and used to calculate damages and benefits. Population projections are based on Moody's County Forecast Database which has population projections to the year 2038. Moody's projections were extended by the New Orleans District from the year 2038 to the year 2085 based on the growth rate forecasted by Moody's for the years 2018 through 2038. The slow, steady growth rate projected by Moody's during this 20-year period was consistent with the growth predicted by parish planning officials.

As shown in Table 1, both Lafourche and Terrebonne Parishes have experienced a steady increase in population between 1970 and 2010. According to U.S. Census data, the population of Lafourche Parish was 89,974 in 2000 and 96,318 in 2010, an increase of 6,344 residents over the ten-year period. During the same period, the population of Terrebonne Parish increased from 104,503 to 111,860, an increase of 7,357 residents. The population in both parishes is projected to maintain this steady increase in population growth, with Lafourche Parish expected to have roughly 97,900 residents in 2035 and approximately 104,200 residents in the year 2085. Terrebonne Parish is expected to experience even more growth with an estimated population of roughly 120,900 in 2035 and 142,800 in 2085. Approximately 218,800 residents are projected to reside in the two-parish area in 2035, while approximately 247,000 residents are projected for the year 2085.

Table 2 shows the number of households in each parish in 1970, 1980, 1990, 2000, and 2010 and projections for the years 2035 and 2085. The projected number of households was based on Moody's County Forecast Database and extended from the year 2038 to 2085 by the New Orleans District based on the growth rate forecasted by Moody's for the years 2018 through 2038.

The total number of households in Lafourche and Terrebonne Parishes experienced a steady increase between 1970 and 2010, which paralleled the growth in population. This increase, which was commensurate with the population growth experienced by the entire Gulf Coast region during the same period, can be attributed to increases in oil and gas exploration in the Gulf of Mexico and technological advancements in the industry. Similar to the projected population growth in the two-parish area, the number of households is expected to continue increasing through the year 2085. Lafourche Parish is projected to have approximately 36,300 households in the year 2035, while Terrebonne Parish is projected to have about 43,400 households. By the year 2085, the number of households in Lafourche Parish is expected to reach approximately 38,100, while the number in Terrebonne Parish is expected to reach to approximately 50,400. In total, the two parishes are projected to have approximately 88,600 households in the year 2085.

Employment.

Table 3 shows the total nonfarm employment by parish for the years 1970, 1980, 1990, 2000, 2010, and projections for the years 2035 and 2085. The employment projections were based on the Moody's County Forecast Database and extended from the year 2038 to the year 2085 by New Orleans District based on the growth rate forecasted by Moody's for the years 2018 through 2038.

Employment trends in the area have historically moved with the demand for oil and gas resources. The unemployment rate in Terrebonne and Lafourche parishes averaged approximately three percent prior to the end of 2008. The Houma-Thibodaux Metropolitan

Statistical Area (MSA) continues to lead the state in jobs created and has one of the lowest unemployment rates in the state.

While the oil and gas industry pays the highest wages of all of the sectors of the economy, the services industry employs the largest number of residents. The retail sector is the second largest employer followed by government and other public agencies. The oil and gas sector in Terrebonne Parish employs slightly over 5,000 residents.

In addition to the oil and gas industry, there are three other sectors of the economy that are important to the region: energy, fisheries, and agriculture. The GIWW, the Houma Navigation Canal, and Bayou Lafourche provide key navigational channels for the energy sector. The coastal region provides a fertile spawning ground for fisheries including shrimp, crabs, oysters, and finfish. Finally, the area grows and processes sugarcane that is used both domestically and abroad.

Social Profile of the Study Area

This section provides a baseline profile of the social characteristics of the study area. Data for the social profile were obtained from a variety of sources including 2010 U.S. Census records, the 2006-2010 U.S. Census Bureau's American Community Survey (ACS) estimates[1], ESRI data, and aerial photography. The baseline characteristics are considered the existing and future-without project conditions.

<u>Health and Safety</u>

Severe flood events threaten the health and safety of residents living within the study area. Loss of life, injury, and post flood health hazards may occur in the event of catastrophic flooding. For example, while the study area was not directly impacted by Hurricane Katrina, the Louisiana Recovery Authority estimated (as of November 2006) that 1,464 fatalities occurred associated with Hurricane Katrina with 135 more residents declared missing. Hurricanes Gustov and Ike were less costly in terms of lives lost, but still claimed 98 deaths. When facilities that provide critical care or emergency services are impacted by flood events, residents are at an even greater risk for experiencing negative health outcomes. Both Hurricanes Katrina and Rita reduced the previous availability of health facilities and services and required additional fire and police protection. During Gustov and Ike, some police stations were required to relocate because of flooding. In addition to the damages of Katrina and Rita to hospitals, police stations, and fire stations, many employees providing related services lost their homes reducing the staff needed to operate health and safety services. As many as 30 hospitals were initially closed following the hurricanes with as many as 141 damaged at various levels of impact.

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[1] The U.S. Census Bureau is now only providing population and housing characteristics in the decennial censuses. Other social characteristics (e.g., low-income) will now be provided in the U.S. Census Bureau's American Community Survey (ACS). The ACS provides estimates of social characteristics based on data collected over five years. The 2006-2010 estimates represent the average characteristics over the 5-year period of time.

The number of medical facilities, police stations, and fire stations located within the study area were obtained using 2010 ESRI data (latest year available).

Medical Care Facilities

There are two hospitals, two nursing homes, and three health care service facilities within the portion of Lafourche Parish included in the study area, and 15 medical care facilities (e.g., hospitals, medical centers, home health care services, and nursing homes) in Terrebonne Parish.

Police Stations

Lafourche Parish has seven police stations/sheriff's offices and a juvenile justice facility located within the study area and Terrebonne Parish has four police stations/sheriff's offices, according to ESRI data.

Fire Stations

There are 23 fire stations located within the study area—five in Lafourche Parish and 18 in Terrebonne Parish.

Social Connectedness

The degree to which communities are able to instill a shared sense of belonging and purpose among residents is in large part determined by the communities' civic infrastructure. The presence of social institutions such as libraries, places of worship, and schools provide residents an opportunity for civic participation and engagement which allows residents to come together and work toward a common goal. The number of libraries, places of worship, and schools located within the study area were obtained using 2010 ESRI data (latest year available).

Civic Infrastructure

According to ESRI data, the portion of Lafourche Parish included in the study area has one library, 7 places of worship, and 16 schools. ESRI data also show that there are 6 libraries, 34 places of worship, and 45 schools located within the study area in Terrebonne Parish.

Leisure and Recreation

Having personal leisure time available and having access to recreational areas contributes to residents' quality of life and is therefore an important aspect of well-being. The number of recreational areas within the study area was obtained using 2010 ESRI data (latest year available).

Recreational Areas

Lafourche Parish has four recreational areas located within the study area—the Sugarland Country Club, Acadia Park, Bayou Country Club, and Peltier Municipal Park. Terrebonne Parish

has four also: Southern Oaks Golf Club, Ellendale Country Club, Gray Park, and Colonial Acres Golf Course.

Additionally, recreational fishing and hunting are very important to the area. The high quality of the recreational fishery, especially an abundance of red fish and trout, has made this an important leisure time activity for residents. Inland saltwater fish species, crabs, and shrimp are also available in the more brackish water. Game species hunted in the area include waterfowl, deer, rabbit, squirrels, rail, gallinule, and snipe.

Social Vulnerability/Resiliency

The devastation left behind after Hurricane Katrina brought attention to the salience of the related concepts of social vulnerability and resiliency when evaluating water resources projects (USACE, 2008). Social vulnerability is a characteristic of groups or communities that limits or prevents their ability to withstand adverse impacts from hazards to which they are exposed. Resiliency, in turn, refers to the ability of groups or communities to cope with and recover from adverse events. The factors that contribute to vulnerability often reduce the ability of groups or communities to recover from a disaster; therefore, more socially vulnerable groups or communities are typically less resilient.

Several factors have been shown to contribute to an area's vulnerability/resiliency, including poverty, racial/ethnic composition, educational attainment, and proportion of the population over the age of 65.

Poverty Rate

High poverty rates negatively impact the social welfare of residents and undermine the community's ability to assist residents in times of need. The 2006-2010[2] U.S. Census data indicate that 15.6 percent of the population of Lafourche and 17.4 percent of the population in Terrebonne Parish fell below the poverty line. In contrast, 18.1 percent of the population in the state of Louisiana and 13.8 percent in the nation overall fell below the poverty line during the same period.

Racial / Ethnic Composition

Race/ethnicity continues to play an important role in the everyday lives of Americans. Unequal access to social resources and language barriers may affect preparing for and recovering from flood events for certain groups. Table 4 shows the racial and ethnic characteristics of Lafourche and Terrebonne Parishes, according to the 2010 U.S. Census. In both parishes, the majority of the population is non-Hispanic white (78.0% in Lafourche Parish and 68.6% in Terrebonne Parish), followed by non-Hispanic black (13.2% in Lafourche Parish and 18.8% in Terrebonne Parish). The Hispanic population in both parishes is roughly 4.0 percent.

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[2] As stated previously, the 2006-2010 estimates represent the average characteristics over the 5-year period of time.

Additionally, approximately 230 members of the Biloxi-Chitimacha tribe are located in Isle de Jean Charles which is in the southern portion of Terrebonne Parish.

Educational Attainment

Educational attainment also has important implications for the social vulnerability/resiliency of communities. More educated individuals have less difficulty accessing information and navigating the sometimes complex process of recovery after flood events (e.g., obtaining government assistance, insurance claims, etc.) According to 2006-2010 ACS data, the percentage of the population age 25 and older in Lafourche Parish with a high school diploma is 72.1 percent and 14.3 percent has a bachelor's degree or higher. Similarly, 73.0 percent of the population 25 and older in Terrebonne Parish has a high school diploma and 13.0 percent has a bachelor's degree or higher. These figures are lower than the state of Louisiana (81.0% has a high school diploma and 20.9% has a bachelor's degree or higher) and the nation overall (85.0% and 27.9%, respectively).

Age

Age is another important factor to consider when examining the social vulnerability/resiliency of a community. For example, elderly residents may have special needs or mobility issues and require more social resources before, during, and after flood events. According to 2010 U.S. Census data, the proportion aged 65 and older in Lafourche Parish is 12.5 percent and 11.2 percent in Terrebonne Parish. The state of Louisiana and the nation overall have roughly the same proportion of the population over the age of 65 (12.3% and 13.0%, respectively).

Social Vulnerability Index

The Hazards and Vulnerability Research Institute at the University of South Carolina created an index that compares the social vulnerability of U.S. counties/parishes to environmental hazards. The variables included in the index are based on previous research which has found that certain characteristics (e.g., poverty, racial/ethnic composition, educational attainment, and proportion over the age of 65) contribute to a community's vulnerability when exposed to hazards. According to the IWR OSE handbook (USACE, 2008), the Social Vulnerability Index (SoVI®)[3] is a valuable tool that can be used in the planning process to identify areas that are socially vulnerable and whose residents may be less able to withstand adverse impacts from hazards.

The SoVI® was computed as a comparative measure of social vulnerability for all counties/parishes in the U.S., with higher scores indicating more social vulnerability than lower scores. Lafourche Parish has a SoVI® 2005-09 score of -1.20 (0.29 national percentile) and Terrebonne Parish has a SoVI® 2005-09 score of -1.08 (0.31 national percentile). Stated another way, Lafourche and Terrebonne Parishes are less socially vulnerable than roughly 70 percent of counties/parishes in the U.S. In comparison, Orleans Parish—notorious for its enduring levels of

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[3] More information on the methodology and data used to calculate the SoVI® can be found here: http://webra.cas.sc.edu/hvri/products/sovi.aspx

high poverty—has a SoVI® 2005-09 score of 2.06 with only 18 percent of counties/parishes in the nation ranked more socially vulnerable.

The study area's social vulnerability, however, is expected to increase over time if subsidence and sea level rise continue to occur, and the population in the study area increases as it is projected to do. The absolute number of socially vulnerable people (e.g., low-income, minority, less-educated, and over the age of 65) at risk for flood events will increase. This, in turn, may lead to an increased burden placed on local, state, and federal agencies to ensure that these socially vulnerable populations have access to resources before, during, and after flood events.

III. OTHER SOCIAL EFFECTS EVALUATION OF ALTERNATIVES

Social Implications of the Alternatives

This section provides an OSE analysis of the project alternatives. The evaluation is based on the differential impact that each alternative is expected to have on the socioeconomic characteristics and other social factors of the study area presented in the previous section.

The analysis was conducted based on without-project overflow and depth-of-flooding data provided by Engineering Division. The data were provided for the years 2035 and 2085 for 2% annual chance exceedance (ACE) events (50-year), 1% ACE events (100-year), and for 0.2% ACE events (500-year). Figures 1-3 show the estimated depth of inundation during 2%, 1%, and 0.2% ACE events for the year 2085.

The data do not take into account the performance of local levees. As a result, impacts to population, housing, medical facilities, etc. are overstated. Local levee systems provide flood risk reduction under existing conditions (2010) for over 25,000 residential and non-residential structures. Local levees are expected to provide flood risk reduction between a 10% ACE event (10-year) and 7% ACE event (15-year), on average.

Performance of the federal levee for the 1% AEP Alternative would reduce risk for elevations up to approximately the stages associated with the 1% ACE event (100-year) or slightly above. This is again assuming that the levee doesn't fail at an elevation below the design elevation of the Federal levee.

Performance of the federal levee for the 3% Annual Exceedance Probability (AEP) Alternative would reduce risk for elevations up to approximately the stages associated with the 3% ACE event (35-year)—assuming that the levee doesn't fail at an elevation below the design elevation of the Federal levee. Therefore, for the purposes of this analysis, it is assumed that the 3% AEP Alternative would fail when exposed to 2% ACE events (50-year) and for less frequent events.

Tables 5 and 6 present the results of the impacts to population and housing, medical/emergency facilities, civic infrastructure, and recreational areas under the No Action Alternative, the 1% AEP Alternative, and the 3% AEP Alternative based on these without-project overflow and depth-of-flooding data. The impacted population and housing figures are based only on without-

project overflow data and not depth-of-flooding data. Therefore, impacts due to flooding could range from minimal to extensive. Medical/emergency facilities, civic infrastructure, and recreational areas are considered impacted if depth-of-flooding data show two feet or more of flooding in the facility location.

Again, it is important to note that the reduced risk associated with the 1% AEP Alternative and the 3% AEP Alternative are based on without-project depth-of-flooding data and the assumption that the 1% AEP Alternative will provide flood risk reduction for 1% (and more frequent) ACE events and the 3% AEP Alternative will provide flood risk reduction for 3% (and more frequent) ACE events.

Population and Housing

No Action Alternative

As shown in Table 5, if the population and housing units increase at the rate projected by Moody's, a 0.2% ACE event in 2035 would impact 206,700 residents/71,300 housing units; a 1% ACE event would impact 181,500 individuals/62,600 housing units; and a 2% ACE event would impact 180,200 residents/62,100 housing units.

Table 6 shows that in 2085, a 0.2% ACE event would impact 242,400 residents/83,600 housing units; a 1% ACE event would impact 217,200 individuals/74,900 housing units; and a 2% ACE event would impact 215,900 residents/74,500 housing units.

The No Action Alternative would not provide risk reduction to the residents living within the study area which would increase over time due to sea level rise. A catastrophic flood would result in severe negative impacts to residents and cause significant damage to residential structures. Additionally, residents in these communities would not be able to benefit from discounted flood insurance premiums offered by the National Flood Insurance Program (NFIP) should the flood rate insurance maps be updated to reflect increases in flood risk over time due to sea level rise.

1% AEP Storm Surge Risk Reduction System Alternative

As shown in Tables 5 and 6, if no action is taken by USACE, approximately 181,500 individuals/62,600 housing units would be impacted by a 1% ACE event in 2035 and 217,200 residents/74,900 housing units in 2085.

Under the 1% AEP Alternative, these residents and housing units would be at a reduced risk for adverse impacts as a result of 1% (and more frequent) ACE events. Additionally, many residents in these communities would be able to benefit from discounted flood insurance premiums offered by the NFIP (should the flood rate insurance maps be updated to reflect increases in flood risk over time due to sea level rise).

It's also important to note that approximately 840 residential structures (roughly 2,500 people) are located outside of the project alignment and would not benefit from this alternative. This

includes approximately 230 members of the Biloxi-Chitimacha tribe who are located in Isle de Jean Charles which is outside of the southern boundary of the project alignment in Terrebonne Parish.

<u>3% AEP Storm Surge Risk Reduction System Alternative</u>

Under the 3% AEP alternative, residents and housing units would be at a reduced risk for adverse impacts as a result of 3% (and more frequent) ACE events. However, if a 2% or less frequent ACE event occurs, these residents and housing units would not experience any reduction in risk. Additionally, residents in these communities would not be able to benefit from discounted flood insurance premiums offered by the NFIP (again, if the flood rate insurance maps are updated to reflect increases in flood risk over time due to sea level rise).

As with the 1% AEP Alternative, approximately 840 residential structures (roughly 2,500 people) are located outside of the project alignment and would not benefit from this alternative. This includes approximately 230 members of the Biloxi-Chitimacha tribe who are located in Isle de Jean Charles which is outside of the southern boundary of the project alignment in Terrebonne Parish.

<u>Health and Safety</u>

<u>No Action Alternative</u>

As stated previously, the study area includes 22 medical care facilities (e.g., hospitals, medical centers, home health care services, and nursing homes). As shown in Table 5, under the No Action Alternative, a 0.2% ACE event in 2035 would impact 20 medical facilities, 12 police stations/sheriff's offices/juvenile justice facility, and 22 fire stations; a 1% ACE event would impact 16 medical facilities, 6 police stations/sheriff's offices/juvenile justice facility, and 18 fire stations; and a 2% ACE event would impact 13 medical facilities, 4 police stations/sheriff's offices/juvenile justice facility, and 17 fire stations.

Table 6 shows that in 2085, a 0.2% ACE event would impact 20 medical facilities, 12 police stations/sheriff's offices/juvenile justice facility, and 22 fire stations; a 1% ACE event would impact 18 medical facilities, 8 police stations/sheriff's offices/juvenile justice facility, and 19 fire stations; and a 2% ACE event would impact 16 medical facilities, 5 police stations/sheriff's offices/juvenile justice facility, and 17 fire stations.

While evacuation for severe weather events in the Morganza study area is typically high due to mandatory evacuation orders by state authorities, flood events threaten the health and safety of those residents who remain in the area. The potential for loss of life and injuries during flood events for those remain, and the risks of post flood health hazards attributable to such widespread flooding, are greater under the No Action Alternative as compared to the project alternatives. Residents are at an even greater risk for experiencing negative health outcomes when facilities that provide critical care or emergency services are impacted by flood events. The No Action Alternative has a higher potential for reducing the availability of health facilities and services and requiring additional fire and police protection than the project alternatives.

1% AEP Storm Surge Risk Reduction System Alternative

As shown in Table 5, the 1% AEP Alternative would provide reduced risk during 1% ACE events (100-year) to 16 medical care facilities, 6 police stations/sheriff's offices, and 18 fire stations that without-project depth-of-flooding data show would experience two feet or more of flooding in 2035. Table 6 shows that by the year 2085, the number of medical facilities experiencing reduced risk under this alternative would increase to 18, police stations/sheriff's offices would increase to 8, and fire stations would increase to 19.

The 1% AEP would result in the greatest potential for reduced risk to the health and safety of residents living within the Morganza study area.

3% AEP Storm Surge Risk Reduction System Alternative

The 3% AEP Alternative would provide reduced risk during 3% (and more frequent) ACE events to medical care facilities, police stations/sheriff's offices, and fire stations located in the study area. However, if a 2% (or less frequent) ACE event occurs, these facilities would not experience any reduction in risk.

The 3% AEP would result in reduced risk to the health and safety of residents living within the study area during 3% (and more frequent) ACE events. However, residents would remain at risk for experiencing negative health outcomes during less frequent events.

Social Connectedness

No Action Alternative

As stated previously, the study area includes 7 libraries, 41 places of worship, and 61 schools. Table 5 shows that under the No Action Alternative, a 0.2% ACE event (500-year) would impact all 7 libraries, 40 places of worship, and 56 schools in 2035. Table 5 also shows that under the No Action Alternative, a 1% ACE event (100-year) would impact 6 libraries, 36 places of worship, and 43 schools, and a 2% ACE event (50-year) would impact 5 libraries, 34 places of worship, and 38 schools.

Table 6 shows that in 2085, 0.2% ACE event would remain similar to that of 2035, while a 1% ACE event would impact 7 libraries, 39 places of worship, and 56 schools, and a 2% ACE event would impact 6 libraries, 35 places of worship, and 43 schools.

1 % AEP Storm Surge Risk Reduction System Alternative

As shown in Table 5, the 1% AEP Alternative would provide reduced risk during 1% ACE events (100-year) to 6 libraries, 36 places of worship, and 43 schools that without-project depth-of-flooding data show would experience two feet or more of flooding in 2035. Table 6 shows that by the year 2085, the number of libraries experiencing reduced risk under this alternative would increase to 7, places of worship would increase to 39, and schools would increase to 56.

3 % AEP Storm Surge Risk Reduction System Alternative

The 3% AEP Alternative would provide reduced risk during 3% (and more frequent) ACE events to libraries, places of worship, and schools located in the study area. However, if a 2% (or less frequent) ACE event occurs, these facilities would not experience any reduction in risk.

Leisure and Recreation

No Action Alternative

Tables 5 and 6 show that under the No Action Alternative, a 0.2% ACE event (500-year) would impact all 8 recreational areas located within the study area under without-project conditions in the years 2035 and 2085, a 1% ACE event would impact 5 in the years 2035 and 2085, and a 3% ACE event would impact 4 in the years 2035 and 2085.

1 % AEP Storm Surge Risk Reduction System Alternative

As shown in Tables 5 and 6, the 1% AEP Alternative would provide reduced risk during 1% ACE events to 5 recreational areas in the years 2035 and 2085.

3 % AEP Storm Surge Risk Reduction System Alternative

The 3% AEP Alternative would provide reduced risk during 3% (and more frequent) ACE events to recreational areas located in the study area. However, if a 2% (or less frequent) ACE event occurs, these areas would not experience any reduction in risk.

Social Vulnerability and Resiliency

No Action Alternative

As stated previously, social vulnerability in the area is expected to increase over time as the absolute number of socially vulnerable people (e.g., low-income, minority, less-educated, and over the age of 65) at risk for flood events increases should subsidence, sea level rise, and population growth occur to levels expected. Under the No Action Alternative, the area would remain vulnerable to flooding, and long term resiliency would be hampered by the continued local efforts necessary to prepare for, and react to, flood events.

1% AEP Storm Surge Risk Reduction System Alternative

Under the 1% AEP Alternative, the study area would experience flood risk reduction for 1% (and more frequent) ACE events. The level of social vulnerability expected in the study area in the year 2085 would be reduced under this alternative as compared to the No Action Alternative, and thus, the study area's potential for long-term growth and sustainability would be enhanced.

3% AEP Storm Surge Risk Reduction System Alternative

Under the 3% AEP Alternative, the study area would experience flood risk reduction for 3% (and more frequent) ACE events. The social vulnerability of the study area would be reduced under this alternative as compared to the No Action Alternative, and thus, the study area's potential for

long-term growth and sustainability would be enhanced. However, the study area would remain vulnerable to less frequent/more damaging events.

Summary of Alternative Analysis

The Morganza to the Gulf PAC study examined three alternatives—the No Action Alternative, the 1% AEP Alternative, and the 3% AEP Alternative. The OSE analysis evaluated the differential impact that each alternative is expected to have on the socioeconomic characteristics and other social factors of the study area. After first providing a description of the existing and future without-project socioeconomic characteristics and other social factors of the study area, an analysis of the impacts to population and housing, medical/emergency facilities, civic infrastructure, and recreational areas under the three alternatives was conducted. The analysis was conducted based on without-project overflow and depth-of-flooding data for the years 2035 and 2085. Results show significant differences between the alternatives with important implications for the overall social well-being of the study area.

The No Action Alternative would not reduce the risk associated with hurricane and tropical storm damage to residents of the Morganza study area. Therefore, there is a high potential for extensive hurricane and tropical storm damage to continue occurring in the area. The apparent subsidence, or relative sea level rise, that has been taking place in the Morganza to the Gulf area, coupled with the anticipated population growth, is expected to magnify the flooding problems in the future. As a result, subsequent flooding events could cause even more damage to housing units, public facilities, and commercial structures than has previously been experienced. Under this alternative, residents would remain at a higher risk for adverse health impacts such as loss of life and injury, as well as post flood health hazards. The area would remain vulnerable to flooding, and long term resiliency would be hampered by the continued local efforts necessary to prepare for, and react to, flood events.

The 1% AEP would result in the greatest potential for reduced flooding in the Morganza study area. This alternative would reduce the risks associated with damages to housing units, public facilities, and commercial structures for 1% (and more frequent) ACE events as well as provide increased protection to the health and safety of residents living within the study area. The area's social vulnerability would be reduced under this alternative, and thus, the potential for long-term growth and sustainability would be enhanced. Also, under this alternative, the area would be at a reduced risk of incurring the costs associated with clean-up, debris removal, and building and infrastructure repair as a result of flood events.

The 3% AEP would also reduce the risk of flooding in the Morganza study area. However, this alternative would only provide risk reduction for 3% (and more frequent) ACE events. The area would still face risks associated with less frequent (more damaging) events.

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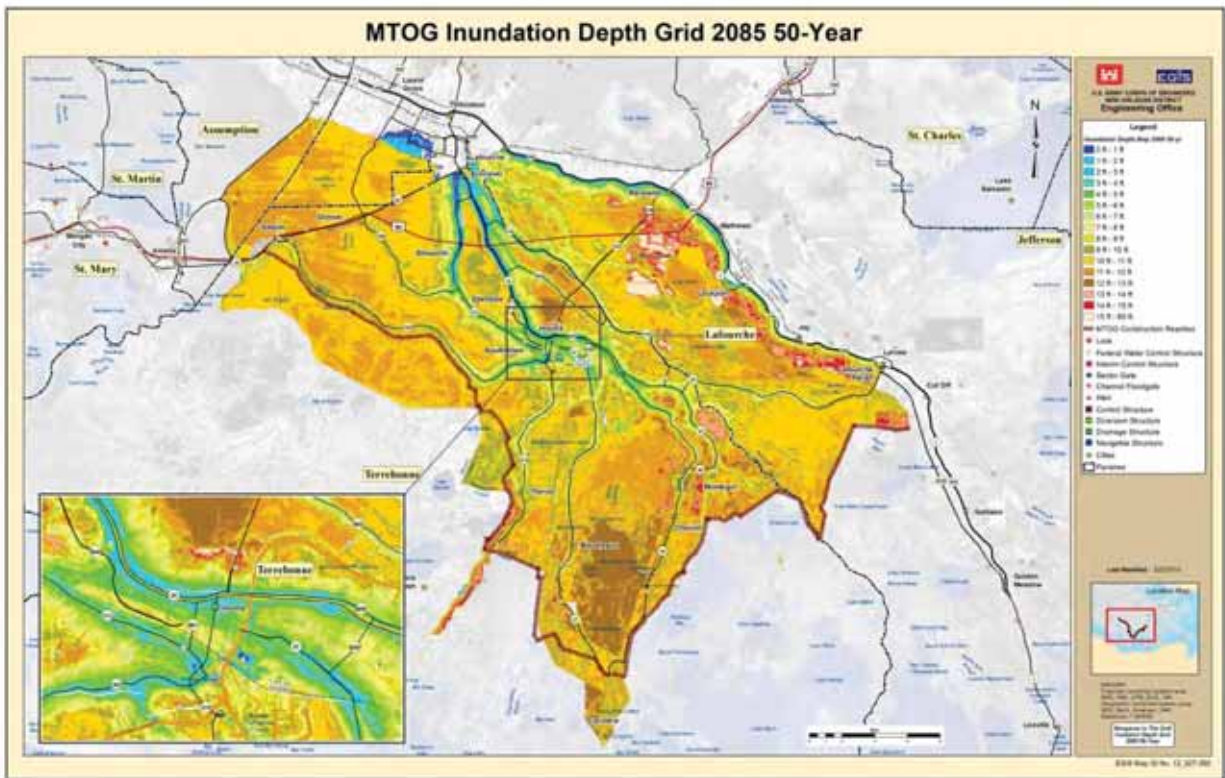
Figure 1. Morganza Study Area Estimated Inundation for 2% ACE Event (50-Year) in 2085

Figure 2. Morganza Study Area Estimated Inundation for 1% ACE Event (100-Year) in 2085
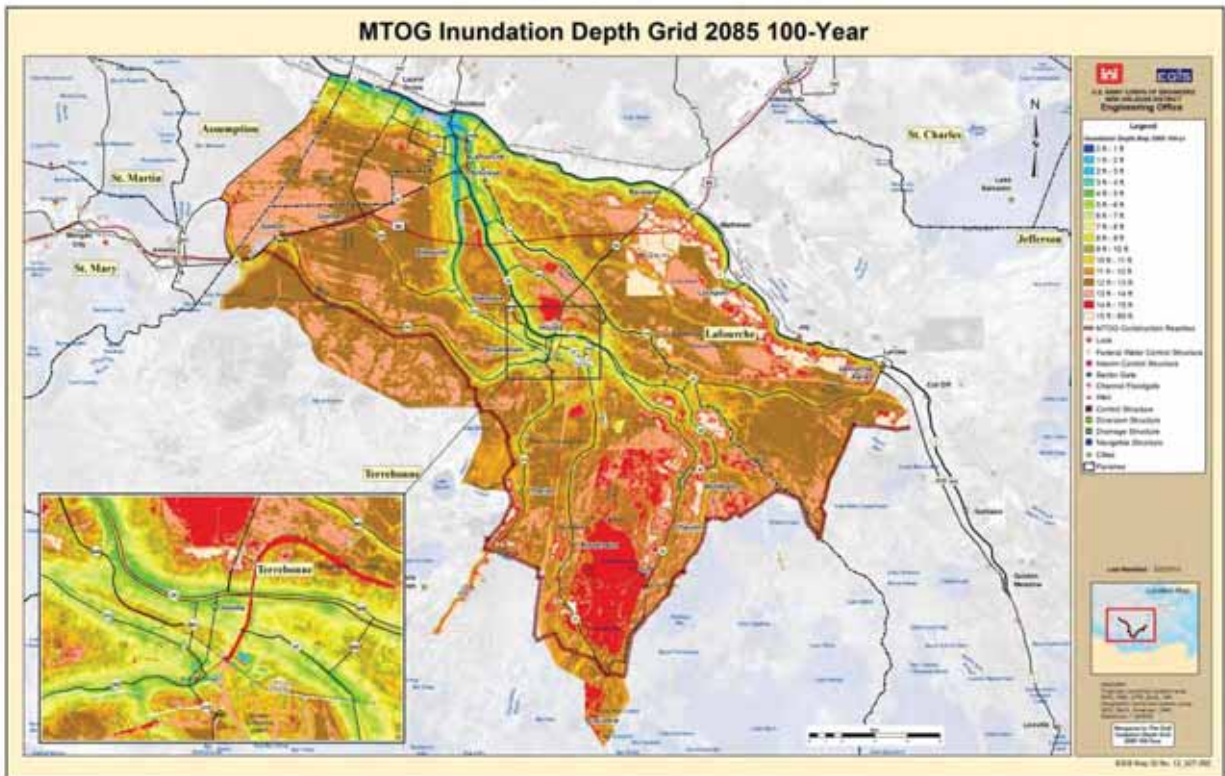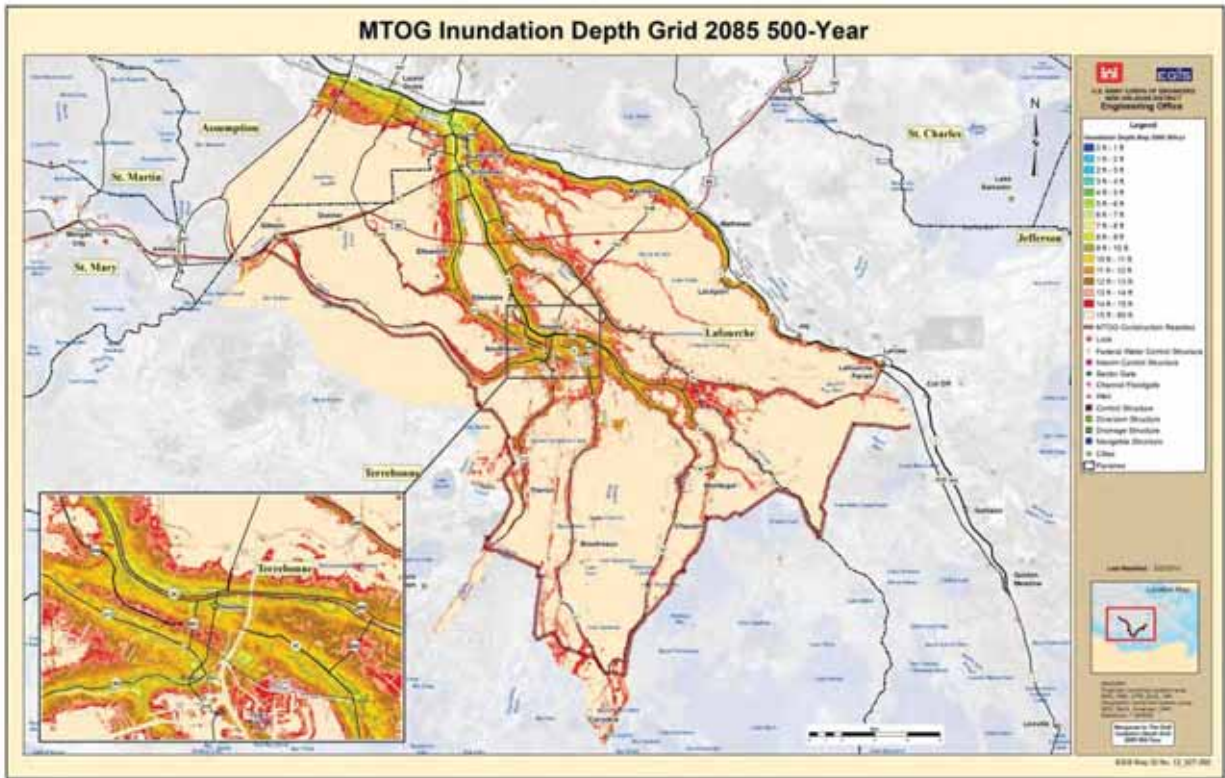
Figure 3. Morganza Study Area Estimated Inundation for 0.2% ACE Event (500-Year) in 2085

Table 1
Historical and Projected Parish Population (1,000s)
2010
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	69.1	83.5	85.8	90	96.3	97.9	104.2
Terrebonne	76.2	95.1	97	104.5	112.0	120.9	142.8
Total	145.2	178.6	182.9	194.4	208.3	218.8	247.0

Source: U.S. Census data, Moody's Country Forecast Database, and discussions with local officials.

Table 2
Number of Households by Parish (1,000s)
2010
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	18.0	25.7	28.8	32.1	33.7	36.3	38.1
Terrebonne	19.6	29.5	31.9	36.0	38.2	43.4	50.4
Total	37.6	55.2	60.7	68.1	71.9	79.7	88.5

Source: U.S. Census data, Moody's Country Forecast Database, and discussions with local officials.

Table 3
Total Nonfarm Employment (1,000s)
2010
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

Parish	1970	1980	1990	2000	2010	2035	2085
Lafourche	15.1	24.4	22.1	30.4	37.5	40.7	44.2
Terrebonne	24.6	42.4	35.8	47.3	58.9	67.3	81.3
Total	39.7	66.8	57.9	77.7	96.4	108.0	125.5

Source: Based on Moody's Forecast and discussions with local officials

Table 4
Race and Ethnic Composition Morganza to the Gulf PAC
2010
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

	Lafourche Parish		Terrebonne Parish	
	Number	%	Number	%
Total	96,318		111,860	
Hispanic	3,647	3.8	4,421	4
Non-Hispanic	92,671	96.2	107,439	96
White alone	75,080	78	76,789	68.6
Black or African American alone	12,679	13.2	21,046	18.8
American Indian and Alaska Native alone	2,623	2.7	6,226	5.6
Asian alone	707	0.7	1,127	1
Native Hawaiian and Other Pacific Islander alone	26	0	40	0
Some Other Race alone	62	0.1	93	0.1
Two or More Races	1,494	1.6	2,118	1.9

Source: 2010 U.S. Census

Table 5
Evaluation of Alternatives
2035
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

	No Action Alternative			1% AEP Alternative			3% AEP Alternative		
	50	100	500	50	100	500	50	100	500
Population and Housing									
Population	180.2	181.5	206.7	0	0	206.7	180.2	181.5	206.7
Housing Units	62.1	62.6	71.3	0	0	71.3	62.1	62.6	71.3
Social Factor									
Health and Safety									
Medical Facilities	13	16	20	0	0	20	13	16	20
Police Stations	4	6	12	0	0	12	4	6	12
Fire Stations	17	18	22	0	0	22	17	18	22
Social Connectedness									
Libraries	5	6	7	0	0	7	5	6	7
Places of Worship	34	36	40	0	0	40	34	36	40
Schools	38	43	56	0	0	56	38	43	56
Leisure and Recreation									
Recreational Areas									
	4	5	8	0	0	8	4	5	8

Source: U.S. Census data, Moody's Country Forecast Database, ESRI data.
Based on without-project overflow and depth-of-flooding data provided by Engineering Division
Population/housing figures are based on without-project overflow data and not depth-of-flooding data.
Facilities are considered impacted if depth-of-flooding data show two feet or more of flooding.

Table 6
Evaluation of Alternatives
2085
Morganza to the Gulf of Mexico, La.
Post-Authorization Change Report

	No Action Alternative			1% AEP Alternative			3% AEP Alternative		
	50	100	500	50	100	500	50	100	500
Population and Housing									
Population	215.9	217.2	242.4	0	0	242.4	215.9	217.2	242.4
Housing Units	74.5	74.9	83.6	0	0	83.6	74.5	74.9	83.6
Social Factor									
Health and Safety									
Medical Facilities	16	18	20	0	0	20	16	18	20
Police Stations	5	8	12	0	0	12	5	8	12
Fire Stations	17	19	22	0	0	22	17	19	22
Social Connectedness									
Libraries	6	7	7	0	0	7	6	7	7
Places of Worship	35	39	40	0	0	40	35	40	40
Schools	43	56	56	0	0	56	43	56	56
Leisure and Recreation									
Recreational Areas									
	4	5	8	0	0	8	4	5	8

Source: U.S. Census data, Moody's Country Forecast Database, ESRI data.
Based on without-project overflow and depth-of-flooding data provided by Engineering Division
Population/housing figures are based on without-project overflow data and not depth-of-flooding data.
Facilities are considered impacted if depth-of-flooding data show two feet or more of flooding.

Real Estate Plan

Morganza to the Gulf of Mexico, Louisiana

Morganza to the Gulf Project Vicinity

TABLE OF CONTENTS

PURPOSE OF THE REAL ESTATE PLAN AND
PROJECT DESCRIPTION

This Real Estate Appendix presents the real estate requirements and costs for the Post Authorization Change (PAC) Report for the Morganza to the Gulf of Mexico, Louisiana hurricane and storm damage reduction project. The information contained herein is tentative in nature for planning purposes only.

A previous Real Estate Plan was prepared in August, 2000 in support of the Final Feasibility Report, which was approved in March, 2002.

The Water Resources and Development Act of 2007 (WRDA 2007), Section 1001 (24) authorized construction of the Morganza to the Gulf of Mexico, Louisiana hurricane and storm damage reduction project (commonly referred to as "Morganza to the Gulf"). The authorization was based on reports of the Chief of Engineers dated August 23, 2002 and July 22, 2003. The project was intended to provide for the 100-year level of risk reduction (based on a storm surge elevation that has a 1% chance of being equaled or exceeded in a given year); however, the authorized project cost estimates do not reflect post-Katrina 100-year design elevations, criteria, borrow standards, or construction costs that must now be incorporated into the Morganza to the Gulf project. Changes to the authorized project have exceeded the 20% cost increase limit specified in WRDA 1986, Section 902, and must be documented in a Post Authorization Change Report (PAC REPORT).

This project analyzes the same levee alignment at two levels of risk reduction (1% and 3% probability). The 1% Annual Exceedance Probability (AEP) Storm Surge Risk Reduction System is a hurricane levee system that provides risk reduction for water levels that have a one percent probability of occurring each year. This alternative has the same intended level of risk reduction as the pre-Katrina authorized project but is modified to be consistent with post-Katrina design standards.

The 98-mile Morganza to the Gulf levee system primarily follows existing hydrologic barriers such as natural ridges, roadbeds, and existing local levees. The western extent ties into high ground along US 90 near the town of Gibson, and the eastern extent ties into Hwy 1 near Lockport, LA. Levee elevations range from 15.5 to 24.0 feet for base year (2035) conditions and from 19.5 to 26.5 feet NAVD88 for future year conditions. Structure elevations range from elevation 17.5 to 33.0 NAVD88.

The project includes 1 lock, 22 navigable floodgates, 23 environmental water control structures, 9 road gates, and fronting protection for 6 existing pumping stations. Structures on Federally maintained navigation channels include the Houma Navigation Canal Lock Complex (and 250-ft sector gate) and two 125-ft sector gates on the GIWW east and west of Houma. In addition, fourteen (14) 56-ft sector gates and five (5) 20- to 30-ft stop log gates are located on various waterways that cross the levee system.

A summary map showing the location of the project reaches is included as Exhibit A to this Real Estate Appendix.

The levees will be constructed in lifts. There will be a maximum of four lifts in various thicknesses. (Larose C-North 1% AEP plan includes a 5th lift.) It is assumed that the lifts will be constructed between 2015 and 2071, with final settlement being reached in the year 2085.

Figure 1-1 below is a summary map of the project reaches.

Figure 1-1 Morganza PAC Alignment

PROJECT LOCATION

The project/study area is generally located in coastal Louisiana about 60 miles southwest of New Orleans. The area includes all of Terrebonne Parish and portionsof Lafourche Parish. The study area extends south to the saline marshes bordering the Gulf of Mexico and encompasses approximately 1,891 square miles. Bayou Lafourche forms the eastern study boundary and Bayou Black and Louisiana Highway 311 forms the western boundary. The Gulf Intracoastal Waterway (GIWW) passes through the northern part of the study area in an east-west direction, and the Houma Navigation Channel (HNC) extends due south from Houma to the Gulf of Mexico.

A map of the project area is located within Exhibit A. Preliminary design plates showing the location of each Reach are located in the Engineering Appendix of the Post Authorization Change Report.

PROJECT AUTHORIZATION

The Morganza to the Gulf of Mexico project was conditionally authorized in WRDA 2000 at a cost of $550 million subject to having a favorable Chief of Engineer's Report completed by December 2000; however, the terms of this conditional authorization were not met. The PED phase on the HNC lock complex was initiated in advance of the PED phase for the Morganza to the Gulf of Mexico hurricane and storm damage reduction project. The PED Agreement for the HNC lock was signed in January 2000.
The Morganza to the Gulf feasibility study and PEIS were completed in March, 2002, and a PED agreement was signed in May, 2002. In August 2002, USACE issued a Chief of Engineers report. In July 2003, USACE issued a supplemental Chief of Engineers report, which made changes to the Non-Federal Sponsor's in-kind services. In accordance with the 2002 and 2003 reports of the Chief of Engineers, the Morganza project is authorized as a feature of the Mississippi River and Tributaries (MR&T). Section 158 of the Energy and Water Development Appropriations Act, 2004 (P.L. 108-137) authorized construction on Reach J-1, which had been previously identified as work-in-kind.

The PED Amendment 1 executed in March 2005 combined the two PED efforts into one and allowed the non-Federal sponsor to advance funds on the combined PED effort. WRDA 2007 authorized the Morganza to the Gulf of Mexico, Louisiana project for hurricane and storm damage reduction at a total cost of $886.7 million. The PED Amendment 2 executed in January 2011 increased the funding ceiling and changed the name of the Non-Federal Sponsor from Louisiana Department of Transportation and Development (DOTD) to the Louisiana Coastal Protection and Restoration Authority Board (CPRAB).

The Morganza to the Gulf Project was authorized by WRDA 2007 (P.L. 110-114, Section 1001(24)) on November 9, 2007. In accordance with the project authorization contained in Section 1001(24) of WRDA 2007, construction of the project requires a 65 percent cost share by the Federal government and a 35 percent cost share by the non-Federal sponsor. The operation, maintenance, repair, rehabilitation, and replacement of the HNC lock complex and the GIWW floodgate features of the project that provide for inland waterway transportation are a Federal responsibility in accordance with section 102 of WRDA 1986 (33 U.S.C. 2212). The Operation and Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) of all other project features is the Non-Federal Sponsor's responsibility.

NON-FEDERAL SPONSOR

The Louisiana Coastal Protection and Restoration Authority (CPRAB) and the Terrebonne Levee and Conservation District (TLCD) intend to be the non-Federal co-sponsors for the Morganza to the Gulf project (hereafter referred to as the Non-Federal Sponsor). Section 1001(24) of WRDA 2007 specifies Federal responsibility for OMRR&R of the HNC lock complex and the GIWW floodgate features that provide for

inland waterway transportation in accordance with Section 102 of WRDA 1986, as amended. The Non-Federal Sponsor is responsible for OMRR&R of all other project features.

CPRAB and TLCD, as the Non-Federal Sponsors, are charged with responsibility for the provision of all lands, easements, and rights-of-way, including those required for relocations, the borrowing of material, and the disposal of dredged or excavated material; performing or ensuring the performance of all relocations; and constructing all improvements required on lands, easements, and rights-of-way to enable the disposal of dredged or excavated material all as determined by the Government to be required or to be necessary for the construction, operation, and maintenance of the project (LERRDs).

While CPRAB has condemnation authority, it does not, at this time, possess quick-take condemnation authority. If that condition continues when the Real Estate acquisition process commences, TLCD will have responsibility for the acquisition of real estate interests necessary for construction of the project.

There may be areas which the TLCD does not have the authority to acquire (lands located in Lafourche Parish). If that is the case, CPRAB will enter into third party agreements with the South Lafourche Levee District (SLLD) and the North Lafourche Conservation, Levee and Drainage District (NLCLDD) to acquire real estate rights in Lafourche Parish on behalf of CPRAB. Although neither NLCLDD nor SLLD will be a party to the Project Partnership Agreement, they will utilize their statutory authority on behalf of CPRAB. For that reason, an Assessment of the Non-Federal Sponsor's Real Estate Acquisition Capability has been obtained for NLCLDD and SLLD, as well as CPRAB and TLCD.

Due to the need for a Post Authorization Change Report and Congressional reauthorization of the project, no PPA has been developed at this time. At such time as the project is reauthorized and funds appropriated for the construction of the project, MVN District will commence the PPA development and negotiation process.

Assessments of the Non-Federal Sponsor's Real Estate Acquisition Capability for CPRAB, TLCD, SLLD, and NLCLDD are attached as Exhibit B. The Non-Federal Sponsors have been found to be fully capable of performing acquisition of the LER required for the project.

The TLCD has started work on reaches that were initially proposed to be a part of the Morganza to the Gulf project, at their own expense, acknowledging that there is no signed PPA in place. The TLCD has substantially completed approximately 9 miles of first lift levees and a few floodgates, which are located along the proposed Morganza to the Gulf project. Discussion of those features is included in section 2 of the PAC Report.

In the absence of an executed PPA, the locally constructed levees do not form an integral part of the Morganza to the Gulf Project, and the work performed by the TLCD is not eligible for consideration and approval of work-in-kind credit. If the Morganza to the Gulf project is reauthorized, the Non-Federal Sponsor will be entitled to LERRD credit for the real estate acquired for those local levees, only to the extent that the Non-Federal

Sponsor is required to provide authorization for entry to the LERRDs necessary for any future Morganza to the Gulf project work that is conducted on the locally constructed levees.

LANDS, EASEMENTS & RIGHTS-OF-WAY

The majority of the acreage affected by the project consists of marsh or wooded wetlands. Other lands impacted include woodland, agricultural (cane land), industrial waterfront on the GIWW, residential waterfront lots at the community of Waterproof, and mixed-use waterfront lots on Bayou Petite Caillou and Bayou du Large. (Mixed-use refers to recreational "camps" or residential waterfront lots.) The Right-of-Way in Lafourche Parish consists mostly of marsh and open water, with the exception of 3 acres of industrial property along the GIWW at the location of a proposed floodgate in the town of Larose. These 3 acres consist of industrial property that appears to be vacant. It is not anticipated that a business relocation will be necessary in this area.

This report references LERRDs for three different types of acquisitions: 1) The LERRDs required for construction and OMRR&R of the project, 2) the LERRDs required for future lift borrow, and 3) the LERRDs required for induced flooding (refer to the Section entitled Induced Flooding below). Because the location of future lift borrow sites has not yet been determined, and this project feature will be addressed in the future, this acquisition was not included in the LERRDs required for project construction (first lift). The LERRDs required for potential induced flooding are an assumed mitigation feature, and not a part of project construction. The PDT determined that these LERRDs should be included for cost purposes, should it be determined in the PED phase that it is necessary to acquire these properties for mitigation purposes. The chart below shows the estimated number of ownerships affected by each acquisition:

	# Ownerships
Project/Levee Alignment	580
Future Lift Borrow	325
Induced Flooding	1,010

PROJECT LEVEE ALIGNMENT

The project will affect approximately 580 ownerships*. Project features and proposed estates by Reach are shown on Table 1.1 below:

Table 1.1 Project Features, Proposed Estates and Estimated Acres

Reach	Land Class	Est. Owners	Miles	Fee Exc. Minerals Acres	Perpetual Levee Easmt. Acres	Temp. Work Area (Borrow) Acres	Temp. Work Area (Access) Acres	Temp. Work Area (Staging) Acres	Temp. Work Area (Staging) HNC Acres
Barrier Alignment	W/F/C/WFR/R/I	75	15.7		732	434	5	20	
A	M/W/F/C/WMA	26	8.2		373	226	7		
B	M	33	5.1	259	398	118		2	
E-2	M/R	10	2.3	120	207	68	0.25		
E-1	M	10	2.1	151	253	93	0.25	2	
F-2	M	3	1.9		139	77			
F-1	M/C	18	2.2	1287	75	51	1	16	298
G-1	M	5	2.2	0	158	57	0.25		
G-2	M	5	1.7	150	88	25		2	
G-3	M	5	1.9	0	87	23	0.25		
H-1	M/W/C/WFR	15	1.9	99	120	68		2	
H-2	M	20	2.6	142	205	100			
H-3	M	35	3.4	212	253	142			
I-1	M	10	1.7		156	61			
I-2	M/C/WFR	33	2.1	161	164	105	0.25		
I-3	M	43	1.9	175	170	117			
J-2	C/WMA	15	4.9	632	488	194			
J-1	WMA	13	3.1	389	292	126			
J-3	W/WFR	10	1.3	183	132	49		2	
K	M/C/WMA/WFR	12	5.1	782	418	308			
L	M/C/WMA	2	5.9	0	499	184		2	
Larose Floodgate	I	2		3.5				2	
Lockport to Larose	M/W/F	75	14		412				
Larose C North	M/F/R/I	105	7		166				
TOTALS:		**580**	**98.2**	**4,746**	**5,985**	**2,626**	**14.25**	**50**	**298**

M=Marshland, **W**=Woodland, **F**=Farmland, **C**=Canals, **WMA**=Wildlife Management Areas, **WFR**=Water Front Residential, **R**=Residential, **I**=Industrial

ACCESS

Access to the construction area will be over existing public roads and navigable waters throughout the project area. In some areas, access will be via existing levee Right-of-Way. However, Temporary Access Easements are proposed in small areas where access is needed on privately owned lands. Table 1.1 shows the acreages over which a Temporary Access Easement is proposed. Access areas for Larose to Lockport and Larose C North will be determined during PED.

STAGING

The majority of staging areas for construction of this project will be located within the Right-of-Way for the levee footprint or existing Right-of-Way. Additional Right-of-Way will be required within a few reaches. Table 1.1 shows the acreages over which a Temporary Work Area Easement is proposed for staging.

BORROW

Borrow material for the first lift will be obtained adjacent to the levees in several of the Reaches. Table 1.1 demonstrates the anticipated acreage for the adjacent borrow areas. A Temporary Work Area Easement (Borrow) will be acquired in these reaches.

Borrow areas for the levee extensions (Lockport to Larose and Larose C North) have not yet been identified. As noted in the Project Description Section, additional refinement of designs and costs will need to continue through PED if the project is re-authorized.

Future Lift Borrow

Material for the remaining lifts will be hauled in from remote locations which have not yet been identified. A separate Chart of Accounts was prepared for the acquisition of LERRDs for borrow for future lifts.

Although sites have not yet been identified, the Chart of Accounts (Exhibit D) shows acquisition costs for estimated future lift borrow, assuming 325 landowners affected over 3,250 acres. A Temporary Work Area Easement (Borrow) will be acquired over these areas. While these costs are accounted for, they are estimates and the actual number of ownerships/acres will not be known until future lift borrow sites are identified.

The 580 ownerships referred to in Table 1.1 is an estimate for the proposed features within the chart, and does not include the 325 landowners assumed for future lift borrow. The actual number of landowners affected by the future lift borrow feature is not known, and this rough estimate was not added to the number of landowners referenced in the chart. Total number of affected landowners including future lift borrow (a rough estimate) is 905.

MITIGATION

There will be additional real estate required for mitigation. As noted in Table 1.1, approximately 4,700 acres** of Fee, Excluding Minerals will be acquired for mitigation areas. Please refer to this table for acres required in Fee by Reach. Costs for lands required for mitigation acres are included in costs for land payments in Exhibit D, Chart of Accounts. Real estate costs for mitigation lands are eligible for LERRD credit.

*The estimate of 4,700 acres was based on a preliminary mitigation analysis. Actual acreage may be less for this feature. A detailed mitigation plan and acreage for each of the programmatic features will be developed in the future.

INDUCED FLOODING

Existing pump stations are used to drain the project area. These pump stations, along with water control structures and navigation structures, will be operated so that the construction of the project features will not induce flooding on the protected side of the project.

Induced Flooding – Larose to Golden Meadow Project Area

Figure 2-2 below demonstrates the Larose to Golden Meadow project reaches:
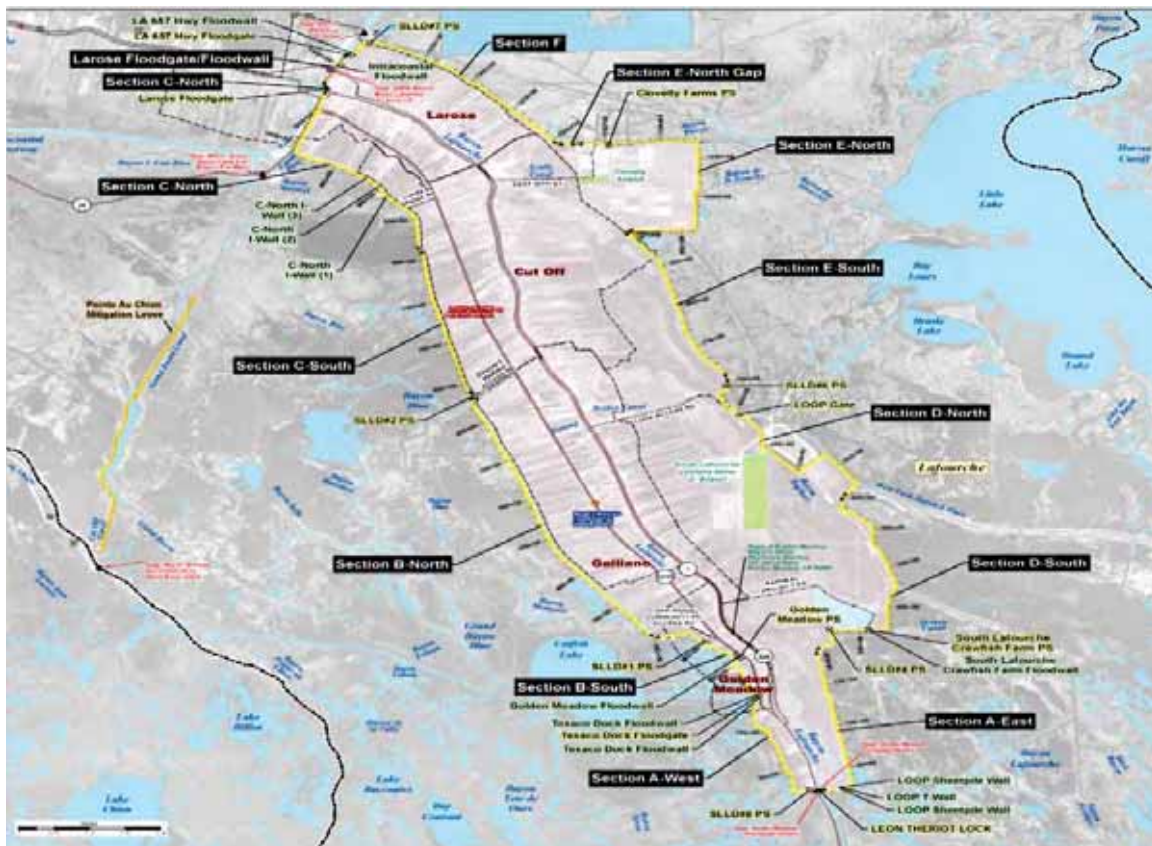
Figure 2-1 Larose to Golden Meadow Project Reaches

It was determined that there would be inducements in Larose levee Sections C-South, B-North, B-South and A-West (Refer to Figure 1.1). The existing levee would be raised in increments varying from 1-3 ft elevation.   The impacts on the Larose to Golden Meadow project are discussed in greater detail in Section 6.5.2 of the PAC Report. It is assumed that the lifts in these sections could be constructed within existing Right-of-Way, and no additional LERRDs will be required for those sections.

Induced Flooding - Areas Outside of Levees

Construction of the levee system has the potential to raise water levels in areas immediately outside the levees by several feet during storm events. These areas include portions of the communities of Gibson, Bayou Dularge, Dulac, and Cocodrie. is the impacts outside the risk reduction system are discussed in greater detail in **Section 6.5.1** of the PAC report. At the current time, information is not available regarding the differences in frequency, depth, and duration of the flooding between the Future Without Project (FWOP) and Future With Project (FWP) conditions. This detailed information typically would be assessed in light of the uses to which the particular land is zoned, and the appropriate mitigation methods, if any, would be implemented to address the effects of the Federal project. Hydrologic modeling estimates that the Future With Project conditions could potentially increase the level of flooding to approximately 1,010 structures. Because of the vast scope of this project and the limited amount of available information at this time, the PDT was not able to look at each affected parcel individually to determine what the level of impact will be, and whether that impact would be categorized as a taking of property rights.   To ensure that the public is informed of all potential impacts of the project and to prevent future delays to project schedule, the PDT, for purposes of this report, has assumed the worst case scenario (most expensive option), a 100% buy-out of all of the structures in the impacted areas.

This cost has been incorporated into the total project cost. These costs were not included as a part of the Gross Appraisal for this project, as they are based on an estimated average cost and not an actual appraisal of the properties involved.

The potential induced damages and mitigation for economic damages would be further addressed during detailed design and supplemental NEPA documents. Individual investigation and devising mitigation for each structure, if appropriate, will be done during Pre Construction Engineering & Design (PED).

Additional factors (height of structures vs. induced stages, type of residential structure, social concerns, etc.) would have to be investigated under PED. Each structure would have to be evaluated under PED to determine if mitigation is appropriate. Further modeling will be performed during PED to determine whether there is a potential taking. A Takings Analysis will be prepared during PED to address this issue, and at that time, it will be determined what real estate interest, if any, will be acquired.

For the purposes of this PAC report, it is assumed that there would be a taking, and that the properties will be acquired in fee excluding minerals. This acquisition will impact 876 residential structures and 134 non-residential structures. The estimate of acquisition costs for residential structures includes the value of the improvement, the value of the land,

moving costs, differential housing payment, payment of last resort, and administrative costs. The estimate of acquisition for the non-residential structures includes the depreciated value of the improvements, land value, moving costs, re-establishment costs, necessary and reasonable incidental costs, and administrative costs. Administrative costs associated with acquisition include the costs of preparing/conducting the following for each ownership: maps and legal descriptions, title, appraisals, negotiations, relocations, closings and condemnation. The total real estate cost associated with this acquisition is estimated to be $305,115,300. These costs/assumptions are broken out in the chart below:

	# Owners	Cost Per Owner (Rounded)	Subtotal	25% Contingency	TOTAL
Acquisition by LS	1,010	9,500	9,595,000	2,398,750	11,993,750
Review of LS Acq	1,010	5,000	5,050,000	1,262,500	6,312,500
Condemnations	500	10,000	5,000,000	1,250,000	6,250,000
Appraisal by LS	1,010	3,000	3,030,000	757,500	3,787,500
Review of Appraisal	1,010	1,000	1,010,000	252,500	1,262,500
P.L. 91-646 Assistance					
Commercial	134	80,300	10,760,200	2,690,050	13,450,250
Residential	876	48,011	42,057,800	10,514,450	52,572,250
Improvement/ Land Value (Average)*	1010	165,430	167,084,240	41,771,060	208,855,300
LERRD Crediting Admin Cost	1010	500	505,000	126,250	631,250
TOTAL		**$322,741**	**$244,092,240**	**$61,023,060**	**$305,115,300**

*For purposes of the Chart of Accounts, total improvement and land value was divided by the number of structures to yield an average cost per owner.

The decision to buyout the 1,010 structures was made within two weeks of the deadline for the final report. That time frame is not sufficient for the preparation of a feasibility level cost estimate. Given the urgency to have the report completed by the deadline, the District real estate office decided to use existing data for real estate costs associated with this acquisition.

The value of the improvements was prepared by URS Group, Inc. under contract with the Corps of Engineers for the economic analysis. URS is mainly an engineering firm. For this assignment, they hired cost estimators to estimate the value of the improvements. URS used similar methodology to estimate the value of improvements as what Corps appraisers would utilize at the feasibility level. The company physically inspected the exterior of each structure, took photographs and cataloged the structure's physical characteristics including the effective age, quality of construction, and condition of structure. URS estimated the size of each structure utilizing an aerial photograph on which the structure's width and length were measured. The value was then estimated by the use of Marshall & Swift Valuation Service and Residential Handbook.

The estimate of land value for each structure is based on an average lot value for residential land and for commercial/industrial land. The estimate of land value is consistent with the values estimated in the gross appraisal for similar properties that will be acquired for the construction of the levee.

The value of the improvements and land value associated with this acquisition is not included in the gross appraisal for this project given that the information utilized to estimate the improvement values was not prepared by an appraiser. The District recognizes that this is not the conventional way to estimate value and that the inclusion of these costs in the REP is contrary to Corps regulations. However, in order to meet the required deadline this was the only alternative. The costs are included in this real estate plan in order to ensure that Congressional authorization of this project includes the necessary funds to construct the project. Once the project is authorized, a relocation plan will be prepared. In accordance with Public Law 91-646, each ownership to be acquired due to potential induced flooding will be appraised individually prior to acquisition of the property.

Estimated costs for this acquisition are included in the Chart of Accounts in Exhibit D to this Real Estate Plan.

Certain areas impacted by induced flooding may be disproportionately impacted by the project. During PED, further investigations will be performed regarding those communities. If those investigations reveal that the communities need to be relocated in a different manner than as addressed herein, the Real Estate Plan will be supplemented.

NON-FEDERAL SPONSOR OWNED LER

Portions of Reach J-2, J-3, K and L are located within the Point Aux Chenes State Wildlife Management Area, which is owned by the Louisiana Department of Wildlife and Fisheries. Pointe Aux Chenes Wildlife Management Area encompasses 35,000 acres in Terrebonne and Lafourche Parishes, approximately 15 miles southeast of Houma. The state will issue a Grant of Particular Use to CPRAB for the lands required for the Morganza to the Gulf Project.

Several of the structures proposed for the project are located within state claimed water bodies. The State of Louisiana will provide authorization for entry for construction of these features. Below is a list of these structures:

Reach	Structure/Location
BA	56-ft sector gate on Bayou Black
B	56-ft sector gate on Bayou Du Large
F-1	56-ft sector gate on Bayou Grand Caillou
G	30-ft sector gate on Bayou Four Points
H-1	56-ft sector gate on Bayou Petit Caillou
I-1/I-2	56-ft sector gate on Bayou Terrebonne
J-3/K	56-ft sector gate on Bayou Pointe Aux Chenes
L	56-ft sector gate on Grand Bayou

As discussed in Section 1.8 of the PAC Report, the Non-Federal Sponsors have constructed or plan to construct several portions of a levee which follows the alignment of this project.
If the Morganza to the Gulf project is reauthorized, the Non-Federal Sponsor will be entitled to LERRD credit for the real estate acquired for those local levees, only to the extent that the Non-Federal Sponsor is required to provide authorization for entry to the LERRDs necessary for any future Morganza to the Gulf project work that is conducted on the locally constructed levees.

The advanced property acquisition and relocation details have been provided to USACE for review and consideration. The Non-Federal Sponsor plans to request credit for the acquisition.   However, for the purpose of providing the estimated total real estate costs for the project, the current LER acquisition being performed by TLCD is not included or accounted for in this Real Estate Plan. In other words, real estate costs are based on acquisition of the entire project alignment. The PPA states that after it has been signed, the Non-Federal Sponsor will be eligible for credit for lands that were acquired and necessary for the project, plus any incidental costs that were spent in acquiring these lands, within 5 years of the date the PPA is signed. Once the PPA is signed and the Non-Federal Sponsor submits a credit package for review, USACE will determine what credit they are entitled to receive.

The Non-Federal Sponsor has been notified in writing of the risks of acquiring LERRDs before execution of the PPA.

ESTATES

The following standard estates will be required for the project:

FEE EXCLUDING MINERALS (With Restriction on Use of the Surface).

The fee simple title to the land, subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines; excepting and excluding all (coal) (oil and gas), in and under said land and all appurtenant rights for the exploration, development, production and removal of said (coal) (oil and gas), but without the right to enter upon or over the surface of said land for the for the purpose of exploration, development, production and removal therefrom of said (coal) (oil and gas).

FLOOD PROTECTION LEVEE EASEMENT

A perpetual and assignable right and easement in (the land described in Schedule A) (Tracts Nos, ____, ____ and ____) to construct, maintain, repair, operate, patrol and replace a flood protection (levee) (floodwall)(gate closure) (sandbag closure), including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

TEMPORARY WORK AREA EASEMENT

A temporary easement and right-of-way in, on, over and across (the land described in Schedule A) (Tracts Nos. _____, _____ and _____), for a period not to exceed _____, beginning with date possession of the land is granted to the United States, for use by the United States, its representatives, agents, and contractors as a (borrow area) (work area), including the right to (borrow and/or deposit fill, spoil and waste material thereon) (move, store and remove equipment and supplies, and erect and remove temporary structures on the land and to perform any other work necessary and incident to the construction of the _____ Project, together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions, and any other vegetation, structures, or obstacles within the limits of the right-of-way; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

TEMPORARY ACCESS EASEMENT (Non-Material Deviation from Standard Estate)

A non-exclusive and assignable temporary easement for a period not to exceed _____years beginning with date possession of the land is granted to the United States, for use by the United States, its representatives, agents, and contractors as an access route and/or right-of-way in, on, over and across (the land described in Schedule A) (Tracts Nos. _____, _____ and _____); together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions and other vegetation, structures, or obstacles within the limits of the right-of-way, reserving, however, to the owners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired, including the right to cross over the right-of-way as access to their adjoining land; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

Approval of the Temporary Access Easement (Non-Material Deviation from Standard Estate) is attached as Exhibit C.

EXISTING FEDERAL PROJECTS WITHIN THE LER REQUIRED FOR THE PROJECT

Gulf Intracoastal Waterway

The Gulf Intracoastal Waterway (GIWW) crosses the northern portion of the project area. The proposed flood protection system will require two floodgates across the GIWW. The eastern floodgate will be in the town of Larose and will cross the Harvey Canal No. 2 segment of the GIWW, approximately 35.2 miles west of Harvey Lock (WHL). The Government has Fee ownership of the channel at this location. The western floodgate will be located about two miles west of the city of Houma, and will cross the Bourg Canal to Bayou Chene segment of the GIWW at mile 62 WHL. The Government owns a perpetual channel and disposal easement at this location. These Federal interests are sufficient for the construction of the floodgates. A small amount of additional fee land will be required on each side of the channel at the eastern floodgate.

Coastal Wetlands Planning Protection Restoration Act (CWPPRA)

CWPRRA Project TE-41, Mandalay Bank Protection Demonstration Project, lies within the footprint of Reach A of the Morganza to the Gulf Project. The sponsors for TE-41 are U.S. Fish and Wildlife Service and Louisiana Department of Natural Resources. The shoreline protection project is located within the boundaries of the Mandalay National Wildlife Refuge, which is owned by the U.S. Fish and Wildlife Service. The project was complete in 2003. Refer to the Federally Owned Lands Section below for more information.

As mentioned in the PAC Report, there are numerous other Federal projects which are being proposed within the Morganza to the Gulf Project area, including Coastal Wetlands Planning, Protection, Restoration Act (CWPPRA), Coastal Impact Assistance Program (CIAP) projects, the Houma Navigation Canal Lock, and the Larose to Golden Meadow project. These projects are currently in the planning phase, and LER has not yet been acquired for the projects. For most of these projects, except the Larose to Golden Meadow project, LER required is not expected to lie within or overlap the Morganza to the Gulf project footprint.

FEDERALLY OWNED LANDS WITHIN THE LER FOR THE PROJECT

The United States owns fee title to lands within the Mandalay National Wildlife Refuge, located within Reach A. Approximately 80 acres within the Wildlife Refuge will be required for the project. The U.S. Fish and Wildlife Service (USFWS) is the managing agency for these lands. The Non-Federal Sponsor will acquire from the USFWS the necessary real estate interests required for the project.

NAVIGATION SERVITUDE

Portions of the project water control structures lie within the navigable waters of the United States, and therefore the Federal Navigational Servitude will be invoked for those portions of the project.

The navigation servitude is the dominant right of the Government under the Commerce Clause of the U.S. Constitution to use, control and regulate the navigable waters of the United States and submerged lands thereunder. The applicability of the navigation servitude depends on both legal and factual determinations. First, it must be determined whether the project feature serves a purpose which is in the aid of commerce. Because this is a hurricane and storm damage risk reduction project, it would serve such a purpose. Secondly, it must be determined whether the land required for the project is located below the mean high water mark (in tidal areas), or below the ordinary high water mark of a navigable watercourse (in non-tidal areas).

The project requires crossing several navigable watercourses. Water control structures will be built within these streams to provide the hurricane and storm damage risk reduction measures for those openings. These portions of the project right-of-way lie below the ordinary high water mark in an inland watercourse that interstate or foreign commerce has either used, is presently using, or is susceptible to use. Therefore, it is within the Navigable Waters of the United States (33 CFR, Part 329). The Federal Navigational Servitude is available for those portions of the project.

BASELINE COST ESTIMATES/CHART OF ACCOUNTS (COAs)

A Chart of Accounts for each Reach is included in Exhibit D of this Real Estate Plan. The total cost for Real Estate Acquisition by Reach is listed below:

Reach Name	Total Real Estate Costs
Barrier Alignment	$ 3,702,000
A	$ 896,000
B	$ 1,045,000
E-1	$ 435,000
E-2	$ 565,000
F-1 (includes HNC Lock Complex)	$ 1,139,000
F-2	$ 150,000
G-1	$ 198,000
G-2	$ 233,000
G-3	$ 154,000
H-1	$ 2,929,000
H-2	$ 611,000
H-3	$ 1,088,000
I-1	$ 322,000
I-2	$ 1,901,000
I-3	$ 1,203,000
J-1	$ 0
J-2	$ 0
J-3	$ 1,120,000
K	$ 1,136,000
L	$ 282,000
Larose Floodgate	$ 159,000
Future Lift Borrow	$ 17,424,000
Induced Damages Flood Side	$305,115,000
Lockport to Larose	$ 2,931,000
Larose C North	$ 10,088,000
TOTAL ALL REACHES	**$354,826,000**

As noted above, costs for the HNC Lock Complex are included in Reach F-1.

The costs include land payments as well as administrative costs and incremental costs associated with acquiring the real estate interests. These estimates include costs of acquiring mitigation lands. The gross appraisal for LER was reviewed and approved at the Division level. However, a revised gross appraisal is currently under review.

Refer to Exhibit D for the Baseline Cost Estimate/Chart of Accounts estimate for each Reach.

Note: The cost estimates do not reflect the costs for facilities/utilities relocations. Refer to the section entitled "Facility/Utility Relocations" for more information.

UNIFORM RELOCATION ASSISTANCE (PL 91-646, Title II as amended)

The benefits of Title II of the Uniform Relocation Assistance and Real Property Acquisition Policy Act of 1970 (P.L. 91-646), as amended, are applicable for this project. Title II requires that persons and businesses displaced by a Federal project be given advisory services and assistance in the location of replacement dwellings and businesses.

Under Title II, displaced persons are entitled to reimbursement for actual and reasonable moving of personal property, differential housing payment, and incidental costs associated with the relocation.   Differential housing payment is a payment made by the Government when the compensation paid for the property being acquired is not sufficient to cover the costs of a replacement dwelling for the displaced persons. Differential payments are capped at $22,500 for homeowners and $5,280 for tenants. However, in cases where the difference between the compensation and the cost of a replacement dwelling is greater than that specified in the regulation, the agency may request approval for payment of last resort. Payment of last resort is calculated on a case by case basis and has no predetermined amount. For this study, the estimate of Title II payment assumes that all displaced persons are homeowners and that the agency will need to request approval for payment of last resort.   This assumption is made because there is not sufficient time to survey residents in order to determine their residency status, and it allows for a higher estimate of Title II payments.   Title II costs were estimated to be approximately $60,000 per displaced family (including contingencies).

The benefits of Title II for displaced businesses are not as lucrative as they are for displaced persons.   Businesses are entitled to receive advisory services, reimbursement for actual reasonable moving costs, re-establishment costs which are capped at $10,000 and certain reasonable and necessary incidental costs associated with the relocation. For purposes of this study, the estimate of relocation for business includes all of these costs and was estimated to be approximately $100,000 per business (including contingencies).

The chart below demonstrates the estimated number of displaced landowners within the levee alignment and the type of property to be displaced. The estimate of relocation assistance benefits are included within the Chart of Accounts provided for each Reach within Exhibit D of this document.

Reach	# Displaced	Type of Property to be Displaced
H	7	Residential
I-2	1	Residential
J-3	1	Residential
K	1	Residential
Flood-side Potential Induced Flooding	1010	134 Non-Residential, 876 Residential
Larose C North	51	51 Residential

It is likely that comparable dwellings will be available to relocate persons displaced as a result of the levee alignments. However, the team recognizes the difficulty associated with relocating 1,010 displaced families and business impacted by the potential induced flooding. It is likely that there will not be sufficient replacement dwellings and business establishments on the market to meet the demand. It may be necessary for homeowners to construct new dwellings and for businesses to construct new buildings or to move to areas that are 25 miles from their current location. The project area is rural, and there is ample vacant land to accommodate new construction. Relocations will be accomplished in phases along with project construction and every effort will be made to relocate displaced persons and business as close to the current communities as possible. The Non-Federal Sponsor will perform these relocations as a part of its responsibility under the project authority. These relocation costs are eligible for LERRD credit. These conclusions are preliminary only.

If the Morganza to the Gulf project is reauthorized, the Non-Federal Sponsor will be entitled to LERRD credit for the real estate acquired for those local levees, only to the extent that the Non-Federal Sponsor is required to provide authorization for entry to the LERRDs necessary for any future Morganza to the Gulf project work that is conducted on the locally constructed levees.

TIMBER/MINERAL/ROW CROP ACTIVITY

The Louisiana Department of Natural Resources provides a Strategic Online Natural Resources Information System (SONRIS), which contains up-to-date information on oil & gas activity in the state of Louisiana. Review of this information indicated that there are several active oil and gas wells within the vicinity of the project. The PDT determined that oil and gas wells will not be relocation items, and the levee alignment would be changed, or T-walls used, during the project Plans and Specifications (P&S) phase to avoid them.

With the exception of the acquisition of the standard Fee Excluding Minerals (With Restrictions on the Use of the Surface) estate over certain lands, the Government will not acquire mineral rights to any of the LER required for the project. Over lands where the fee estate is being acquired, mineral rights will be subordinated. Mineral right owners can still explore for minerals through directional drilling. The timber on most of the wooded right-of-way has little or no value. Any timber present is included in the overall appraised value of the land. For properties impacted by the project which are in agricultural use, the owner will be allowed to harvest crops prior to acquisition.

OYSTER LEASES

Southern Terrebonne and Lafourche Parishes are abundant with oysters. Several oyster leases exist within the project study area. For the Tentatively Selected Plan, seven (7) oyster leases will be impacted by the project. The table below provides information regarding the oyster leases within the project footprint:

REACH	STATIONS	LOCATION	LEASE #	STATUS
H-3	4040-4050	MIT/BORROW/LEVEE	2980904	EXPIRES 2019, TERRY NETTLETON
H-3	4055-4070	BORROW AREA	3041904	EXPIRES 2019, NETTLETON OYSTERS
I-1	4250-4275	MITIGATION AREA	3127005	EXPIRES 2020, BAY NEGRESSE, INC.
I-2	4345-4350	MITIGATION AREA	3303408	EXPIRES 2023, BAY NEGRESSE, INC.
I-2/I-3	4385-4400	MITIGATION/BORROW	2757100	EXPIRES 2015, SANDRA & TERRY, INC.
I-3	4400-4405	MITIGATION AREA	3234007	EXPIRES 2022, TERRY'S SEAFOOD, INC.
I-3	4435-4465	MIT/BORROW/LEVEE	3366109	EXPIRES 2024, COON OYSTERS, LLC

In accordance with State statute (LA R.S. 56:432.1 and Act 523 (2009)), all oyster lease acreage determined to be directly impacted by a project feature shall be acquired. The area of impact to oyster leases is generally considered to be the footprint of the project feature plus an approximately 150 foot buffer. Considering State statute, it is anticipated that at least a portion of seven (7) oyster leases will need to be acquired. The total estimated cost of acquisition of these oyster leases is $316,300 (these costs are included in the Chart of Accounts Section above).

At this time, no estimate is included for possible business relocation costs associated with oyster lease acquisition. It is, however, anticipated that minimal moving costs may be associated with the moving of markers (personal property) that delineate the leased areas.

ZONING ORDINANCES

There will be no application or enactment of zoning ordinances in lieu of, or to facilitate, acquisition in connection with this project.

ACQUISITION SCHEDULE

The following acquisition schedule is based on the premise that the project will impact approximately 580 landowners for the levee alignment. It is assumed that the project will be constructed in sections. A detailed acquisition schedule will be prepared during PED once the 95% plans and specifications are prepared for each section of the project. The schedule below provides the total amount of time to complete the acquisition of real

estate rights for mitigation and for the construction of the levee alignment based on the preliminary information available at this time. This schedule is only for purposes of the feasibility study.

1)   TOD, Mapping   3 years

2)   Obtain Title & Appraisals (begin 1 year after   6 years
        mapping begins then run concurrently with mapping)

3)   Negotiations (begins 1 year after title and Appraisals   10 years
        begin and then run concurrently with those tasks)

4)   Closing/Condemnation   (begins 1 year after   10 years
        negotiations begin and runs concurrently with negotiations)

5)   Eminent Domain Proceedings (begins 1 year after   12 years
        negotiations begin and ends 1 year after the end of
        negotiations)

The following acquisition schedule is for the acquisition of the areas potentially impacted by induced flooding. This schedule assumes acquisition of 1,010 properties. The time frame indicated below is in addition that calculated for acquisition of properties impacted by construction of the levee.

1)   TOD, Mapping   5 years

2)   Obtain Title & Appraisals (begin 1 year after   10 years
        mapping begins then run concurrently with mapping)

3)   Negotiations (begins 1 year after title and Appraisals   15 years
        begin and then run concurrently with those tasks)

4)   Closing/Condemnation   (begins 1 year after   15 years
        negotiations begin and runs concurrently with negotiations)

5)   Eminent Domain Proceedings (begins 1 year after   15 years
        negotiations begin and ends 1 year after the end of
        negotiations)

FACILITY/UTILITY RELOCATIONS

Relocation data was collected and detailed by the USACE New Orleans District, Engineering Division, Design Services Branch Relocations Team, to a feasibility level of design. A separate Relocations Report, containing relocations costs, was submitted as a

reference to the Engineering Appendix of the PAC Report. Maps of potential relocations can be referenced in that appendix. Those relocation costs represent a feasibility level of design and will be further refined during the development of the project P&S.

The project is traversed by numerous crude oil and natural gas pipelines, and by utilities and minor public highways. These facilities are listed and discussed in detail in the "Preliminary Attorney's Investigation and Report of Compensable Interest" (CIR), which was prepared on March 12, 2012 for the Post Authorization Change Report by District Office of Counsel. Based on the limited information available, all of the relocations were found to be compensable.

The Non-Federal Sponsor will perform these relocations as a part of its responsibility under the project authority. The conclusions are preliminary only. The Government will make a final determination of the relocations necessary for the construction, operation or maintenance of the project after further analysis, and completion and approval of the Final Attorney's Opinion of Compensability for each of the impacted utilities and facilities.

There are numerous schools, cemeteries and churches within the project area, but none will be adversely impacted by this project. It is not known at this time whether areas impacted by induced flooding include schools, cemeteries and churches.

HAZARDOUS, TOXIC AND RADIOACTIVE WASTE
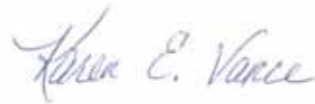
As the result of project changes such as larger levee footprints, all project-related benefits and impacts must be reviewed, including benefits and impacts to environmental habitat, navigation and industry, commercial and recreational fishing, salinity intrusion, and freshwater and sediment diversion.   A Final Phase I Environmental Site Assessment was conducted in May, 2011 for the Morganza to the Gulf Project. A Revised Programmatic Environmental Impact Statement (RPEIS) was prepared to document the environmental changes. In summary, there is a low probability that HTRW would alter the project design or alignment in the PAC levee reaches. No environmental studies have been completed for the additional levee features associated with Larose to Lockport or Larose C-North. Based on the land types/uses impacted by the additional right-of-way (wetlands, agricultural, woodlands and residential), HTRW is not suspected. If the project is re-authorized, Phase I studies would be conducted in these areas. There is one area in the Larose C-North which is industrial. If the project is re-authorized, Phase II investigations would be conducted in this area during PED.

Environmental studies have not been conducted over areas to be acquired as a result of induced flooding. These studies will be conducted during PED phase.

LANDOWNER CONCERNS
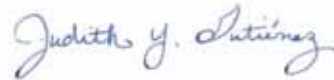
There have been many public meetings regarding the original PAC alignment, and the project has received wide-spread support from the community; however, the attitudes of the landowners who will be directly affected by its construction is not known. The Non-Federal Sponsor is confident that landowner support will be high, and they will be able to acquire the LER required for the project. However, it is anticipated that there will not be strong landowner support for acquisition of properties outside the levee areas, as a result of induced flooding nor for the acquisition of areas impacted by Larose C-North which displace residences.

Prepared by:

_____

Karen E. Vance
Realty Specialist
Real Estate Region South Division
March 26, 2013

Recommended for Approval By:

_____

Judith Y. Gutierrez
Chief, Appraisal & Planning Branch
Real Estate Region South Division
March 26, 2013

EXHIBIT A

PROJECT MAPS

PROJECT STUDY AREA

Figure 3-1. Study Area

Morganza, Louisiana to the Gulf of Mexico
Revised Programmatic Environmental Impact Statement

Image: USDA Prime Imagery

**Eastern Tie In Footprint
LaFourche Parish, Louisiana**

Eastern Tie in Footprint BERM
Eastern Tie in Footprint ROW
Townships
Sections

Feet
10,000
5,000
2,500
0

ROW: 784.51 Acres

BERM: 684.79 Acres

16S 20E

16S 19E

17S 20E

17S 19E

H:\ReeJo Kemper\JUNE6_JUDI_ Footprint For Eastern Tie-In - Morg To The Gulf

100' Width

14,426' Length

LAROSE C NORTH

23,884' Length

705' Width

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

LEGEND

CENTERLINE OF LEVEE

RIGHT-OF-WAY (100 YR, LORR)

TOP OF LEVEE SLOPE (100 YR, LORR)

SCALE : 1" = 400'

400'   0   400'   800'   1200'   1600'

N

TERREBONNE PARISH, LA.

T 16 S
R 19 E

FLOOD SIDE

PROTECTED SIDE

MATCH LINE
B/L STA. 2245+00

SECT. 65

SECT. 63

SECT. 66

SECT. 16

SECT. 18

LIMIT OF WORK
B/L SAT. 2160+70

MARK	DATE	APPR.	DESCRIPTION	MARK	APPR.	DATE	DESCRIPTION	APPR.	DATE

LEGEND

— · — · — CENTERLINE OF LEVEE

— — — — RIGHT-OF-WAY (100 YR, LORR)

————— TOP OF LEVEE SLOPE (100 YR, LORR)

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

TERREBONNE PARISH, LA.

SECT. 54

SECT. 37

SECT. 63

SECT. 65

SECT. 67

SECT. 16

SECT. 67

FLOOD SIDE

PROTECTED SIDE

T 16 S
R 19 E

MATCH LINE
B/L STA. 2320+00

PI 2315+90
PI 2313+41
PI 2311+33
PI 2303+99
PI 2301+69
PI 2299+14
PI 2298+18
PI 2294+16
PI 2294+48

MATCH LINE
C/L STA. 2245+00

N

MATCH LINE B/L STA. 2400+00

MATCH LINE B/L STA. 2320+00

TERREBONNE PARISH, LA.

T 16 S R 20 E

T 16 S R 19 E

SECT. 37

SECT. 67

SECT. 100

SECT. 36

SECT. 63

FLOOD SIDE

PROTECTED SIDE

PI 2393+68
PI 2380+55
PI 2383+78
PI 2382+67
PI 2380+86
PI 2378+21
PI 2377+32
PI 2374+06
PI 2372+32
PI 2366+52
PI 2364+74
PI 2362+44
PI 2361+29
2360+00
PI 2358+16
PI 2356+22
PI 2354+76
PI 2352+76
PI 2345+06
PI 2344+27
PI 2340+11
PI 2336+46
PI 2335+32
PI 2326+00
PI 2321+23
PI 2320+34

LEGEND

CENTERLINE OF LEVEE

RIGHT-OF-WAY (100 YR, LORR)

TOP OF LEVEE SLOPE (100 YR, LORR)

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

N

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

LEGEND

CENTERLINE OF LEVEE

RIGHT-OF-WAY (100 YR, LORR)

TOP OF LEVEE SLOPE (100 YR, LORR)

MATCH LINE
B/L STA. 2520+00

PI 2485+54

2480+00

PI 2448+00

PROTECTED SIDE

T 17 S
R 20 E

FLOOD SIDE

SECT. 100

TERREBONNE PARISH, LA.

2440+00

PI 2434+21

PI 2431+96

PI 2425+75

PI 2420+58

PI 2415+90

PI 2410+24

PI 2402+43

MATCH LINE
B/L STA. 2400+00

T 16 S
R 20 E

N

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

MARK	DATE	APPR	DESCRIPTION	MARK	APPR	DATE	DESCRIPTION	DATE	APPR

ANSI D
SIZE:
FILE NAME: M2G_L5_05.dgn

FILE NUMBER: 4021
PLOT SCALE: X=X
PLOT DATE: 02/06/2012
SUBMITTED BY: J. GAUTREAUX
CONTRACT NO.: X
DWN BY: JAY SHAH
CKD BY: B.R.L.
SOLICITATION NO.: X
DESIGNED BY: BRIAN JOHNSON
DATE: MAY/25/2012

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA.

100 YR. EVENT PLAN
B/L STA. 2520+00 TO B/L STA. 2640+00

SHEET
IDENTIFICATION
M2G-L-05

LEGEND

CENTERLINE OF LEVEE

RIGHT-OF-WAY (100 YR, LORR)

TOP OF LEVEE SLOPE (100 YR, LORR)

MATCH LINE
B/L STA. 2640+00

TERREBONNE PARISH, LA.

T 16 S
R 20 E

2600+00

FLOOD SIDE

PROTECTED SIDE

PI 2568+02

2560+00

MATCH LINE
B/L STA. 2520+00

PI 2553+72

N

SCALE: 1" = 400'

400' 0 400' 800' 1200' 1600'

LEGEND

- - - - - - CENTERLINE OF LEVEE

— - — RIGHT-OF-WAY (100 YR, LORR)

———— TOP OF LEVEE SLOPE (100 YR, LORR)

MATCH LINE
B/L STA. 2760+00

PI 2755+67

2720+00

PI 2717+17

T 16 S
R 20 E

TERREBONNE PARISH, LA.

FLOOD SIDE

PROTECTED SIDE

2680+00

PI 2684+01

MATCH LINE
B/L STA. 2640+00

N

SCALE: 1" = 400'

400'   0   400'   800'   1200'   1600'

MORGANZA, LOUISIANA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE PARISH, LA

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

LEGEND

— · — · — · —   CENTERLINE OF LEVEE

— — — — —   RIGHT-OF-WAY (100 YR, LORR)

————————   TOP OF LEVEE SLOPE (100 YR, LORR)

SECT. 100

SECT. 63

MATCH LINE
B/L STA. 2760+00

2800+00

LIMIT OF WORK
L2G BASELINE STA. 2816+79

PI 2776+11

PI 2798+79

TERREBONNE PARISH, LA.

T 17 S
R 20 E

SECT. 102

SECT. 62

SCALE : 1" = 400'

400'   0   400'   800'   1200'   1600'

US Army Corps
of Engineers®
NEW ORLEANS DISTRICT

LEGEND

CENTERLINE OF LEVEE
RIGHT-OF-WAY (100 YR, LORR)
TOP OF LEVEE SLOPE (100 YR, LORR)

SCALE 1" = 400'

400' 0 400' 800' 1200' 1600'

N

SECT. 3

SECT. 43

SECT. 42

SECT. 2

SECT. 12

T 16 S
R 20 E

T 18 S
R 21 E

TERREBONNE PARISH, LA.

PROTECTED SIDE

FLOOD SIDE

LIMIT OF WORK
LGM B/L STA. 1042+90

22°28'31.1"

999+94.99

995+14.82

988+30.00

970+00.00

973+63.52

973+63.51

956+89.94

956+89.94

943+73.22

930+57.54

293°58'9.5"

293°58'9.5"

288°10'17.6"

296°43'24.0"

300°12'59.2"

306°6'27.1"

MATCH LINE
LGM B/L 925+48.62

EXHIBIT B

ASSESSMENT OF NON-FEDERAL SPONSOR'S
ACQUISITION CAPABILITY

ASSESSMENT OF NON-FEDERAL SPONSOR'S
REAL ESTATE ACQUISITION CAPABILITY

COASTAL PROTECTION AND
RESTORATION AUTHORITY (CPRA)

I. Legal Authority:

    a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes? **YES, if property title is required for the purpose of this project.**

    b. Does the sponsor have the power of eminent domain for this project?
NO. Although the Sponsor does not have eminent domain authority, if this should be needed for the project, the Sponsor may partner with a Levee District or Parish Government which has that authority (Act 225 RS38:301.1 and Act 320), if they agree.

    c. Does the sponsor have "quick-take" authority for this project?
NO. Although the Sponsor does not have quick take authority, if this should be needed for the project, the Sponsor may partner with a Levee District or Parish Government which has that authority (Act 225 RS38:301.1 and Act 320), if they agree.

    d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? **NO**

    e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **YES**

    The United States owns fee title to lands within the Mandalay National Wildlife Refuge, located within Reach A. The U.S. Fish and Wildlife Service is the managing agency for these lands. Some of the LER required for the project within Reach A is located within this area. It is expected that the U.S. Fish and Wildlife Service will support the project and grant the necessary real estate interests required for the project.

II. Human Resource Requirements:

    a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? **NO**

b. If the answer to II.a. is "yes," has a reasonable plan been developed to provide such training? **N/A**

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **YES**

d. Is the sponsor's projected in-house staffing level sufficient considering its other workload, if any, and the project schedule? **Not at this time. However, CPRA has numerous contracts in place which provide ample resources.**

   CPRA is presently under development. It is expected that the staff will continue to grow in the upcoming months/years, provided sufficient budget and proper legal authorities.

e. Can the sponsor obtain contractor support, if required in a timely fashion? **YES, contracts are in place now.**

f. Will the sponsor likely request USACE assistance in acquiring real estate? **It is not likely that the Sponsor will request assistance.**

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **YES**

b. Has the sponsor approved the project/real estate schedule/milestones? **At the feasibility level, there are too many unknowns to develop a definite project schedule. Once project designs are finalized, the Sponsor will be requested to provide an acquisition schedule.**

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? **YES**

b. With regard to this project, the sponsor is anticipated to be: highly capable/fully capable/moderately capable/marginally capable/insufficiently capable. **Highly capable**

V. Coordination:
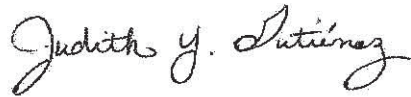
    a.   Has this assessment been coordinated with the sponsor?
       YES

    b.   Does the sponsor concur with this assessment?
       YES

Prepared by: Approved by:

Judith Y. Gutierrez   Cynthia Wallace
Chief, Appraisal & Planning Branch   Chief, Real Estate/Land Rights Division
Real Estate Region South Division   Office of Coastal Protection & Restoration
USACE

ASSESSMENT OF NON-FEDERAL SPONSOR'S
REAL ESTATE ACQUISITION CAPABILITY
<u>TERREBONNE LEVEE AND CONSERVATION DISTRICT</u>

I.   <u>Legal Authority</u>:

    a.   Does the sponsor have legal authority to acquire and hold title to real property for project purposes? (yes/no)

       Yes

    b.   Does the sponsor have the power of eminent domain for this project? (yes/no)

       Yes

    c.   Does the sponsor have "quick-take" authority for this project? (yes/no)

       Yes

    d.   Are any of the lands/interests in land required for the project located outside the sponsor's political boundary (yes/no)

       No

    e.   Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? (yes/no)

       Yes - The United States owns fee title to lands within the Mandalay National Wildlife Refuge, located within Reach A. The U.S. Fish and Wildlife Service is the managing agency for these lands. Some of the LER required for the project within Reach A is located within this area. It is expected that the U.S. Fish and Wildlife Service will support the project and grant the necessary real estate interests required for the project.

II.   <u>Human Resource Requirements</u>:

    a.   Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? (yes/no)

       No

    b.   If the answer to II.a. is "yes," has a reasonable plan been developed to provide such training? (yes/no)

       N/A

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? (yes/no)

   Yes

d. Is the sponsor's projected in-house staffing level sufficient considering its other workload, if any, and the project schedule? (yes/no)

   Yes

e. Can the sponsor obtain contractor support, if required in a timely fashion? (yes/no)

   Yes

f. Will the sponsor likely request USACE assistance in acquiring real estate? (yes/no – If yes provide description)

   No

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? (yes/no)

   Yes

b. Has the sponsor approved the project/real estate schedule/milestones? (yes/no)

   Yes

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? (yes/no/not applicable)

   Yes

b. With regard to this project, the sponsor is anticipated to be: highly capable/fully capable/moderately capable/marginally capable/insufficiently capable. (If sponsor is believed to be "insufficiently capable," provide explanation.)

   Fully Capable

V. Coordination:

a. Has this assessment been coordinated with the sponsor? (yes/no)

   Yes

b.   Does the sponsor concur with this assessment? (yes/no)

   Yes

Prepared by:

_3-11-11_
Date

Karen Vance
Realty Specialist

Reviewed by:

_3-11-11_
Date

Al Levron
Parish Manager
Terrebonne Parish Consolidated Govt.

Approved by:

_3-17-11_
Date

Linda C. LaBure
Chief, Real Estate Division

ASSESSMENT OF NON-FEDERAL SPONSOR'S
REAL ESTATE ACQUISITION CAPABILITY

North Lafourche Conservation, Levee & Drainage District

June 2012

I.   Legal Authority:

   a.   Does the sponsor have legal authority to acquire and hold title to real property for project purposes? **YES**

   b.   Does the sponsor have the power of eminent domain for this project? **YES**

   c.   Does the sponsor have "quick-take" authority for this project? **YES**

   d.   Are any of the lands/interests in land required for the project located outside the sponsor's political boundary ? **NO**

   e.   Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **YES**

      The United States owns fee title to lands within the Mandalay National Wildlife Refuge, located within Reach A. The U.S. Fish and Wildlife Service is the managing agency for these lands. Some of the LER required for the project within Reach A is located within this area.

II. Human Resource Requirements:

   a.   Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? **NO**

   b.   If the answer to II.a. is "yes," has a reasonable plan been developed to provide such training? **N/A**

   c.   Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **YES**

   d.   Is the sponsor's projected in-house staffing level sufficient considering its other workload, if any, and the project schedule? **NO**

e. Can the sponsor obtain contractor support, if required in a timely fashion? **YES, contracts are in place now.**

f. Will the sponsor likely request USACE assistance in acquiring real estate?
    It is not likely that the sponsor will request assistance.

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **YES**

b. Has the sponsor approved the project/real estate schedule/milestones?

    At the feasibility level there are too many unknowns to develop a definite project schedule. Once project designs are finalized, the sponsor will be requested to provide an acquisition schedule.

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? **YES**

b. With regard to this project, the sponsor is anticipated to be: highly capable/fully capable/moderately capable/marginally capable/insufficiently capable.

    Fully capable

V. Coordination:

    a.   Has this assessment been coordinated with the sponsor?
          YES

    b.   Does the sponsor concur with this assessment?
          YES

Prepared by: Approved by:

Judith Y. Gutierrez   Dwayne Bourgeois
Chief, Appraisal & Planning Branch   Executive Director
Real Estate Region South Division   North Lafourche Conservation,
USACE   Levee & Drainage District

ASSESSMENT OF NON-FEDERAL SPONSOR'S
REAL ESTATE ACQUISITION CAPABILITY

South Lafourche Levee District

June 2012

I. Legal Authority:

a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes? **YES**

b. Does the sponsor have the power of eminent domain for this project? **YES**

c. Does the sponsor have "quick-take" authority for this project? **YES**

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary ? **NO**

e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **NO**

II. Human Resource Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? **NO**

b. If the answer to II.a. is "yes," has a reasonable plan been developed to provide such training? **N/A**

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **YES**

d.   Is the sponsor's projected in-house staffing level sufficient considering its other workload, if any, and the project schedule? **NO**

e. Can the sponsor obtain contractor support, if required in a timely fashion? **YES, contracts are in place now.**

f. Will the sponsor likely request USACE assistance in acquiring real estate?
It is not likely that the sponsor will request assistance.

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **YES**

b. Has the sponsor approved the project/real estate schedule/milestones?

At the feasibility level there are too many unknowns to develop a definite project schedule. Once project designs are finalized, the sponsor will be requested to provide an acquisition schedule.

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? **YES**

b. With regard to this project, the sponsor is anticipated to be: highly capable/fully capable/moderately capable/marginally capable/insufficiently capable.

Fully capable

V. Coordination:

   a. Has this assessment been coordinated with the sponsor?
       YES

   b. Does the sponsor concur with this assessment?
       YES

Prepared by: Approved by:

Judith Y. Gutierrez   Windell Curole
Chief, Appraisal & Planning Branch   General Manager
Real Estate Region South Division   South Lafourche Levee District
USACE

EXHIBIT C

NON-MATERIAL DEVIATION FROM
STANDARD ESTATE

TEMPORARY ACCESS EASEMENT

TEMPORARY ACCESS EASEMENT

A non-exclusive and assignable temporary easement for a period not to exceed _____ years beginning with date possession of the land is granted to the United States, for use by the United States, its representatives, agents, and contractors as an access route and/or right-of-way in, on, over and across (the land described in Schedule A) (Tracts Nos. _____, _____ and _____); together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions and other vegetation, structures, or obstacles within the limits of the right-of-way, reserving, however, to the owners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired, including the right to cross over the right-of-way as access to their adjoining land; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

In accordance with paragraph 12-9 c. of ER 405-1-12, the District Chief of Real Estate may approve a non-standard estate if it serves the intended project purpose, substantially conforms with and does not materially deviate from a corresponding standard estate, and does not increase the costs or potential liability of the Government. The foregoing estate complies with those requirements as it achieves the project purpose in as narrow a manner as practical, and is a minor modification of the standard Road Easement, adding language for a temporary term and replacing the word "road" with the words "access route and/or right of way.

Reviewed by:

Marco Rosamano
Assistant District Counsel
New Orleans District

Approved by:

Linda Labure
Chief, Real Estate Division
New Orleans District

EXHIBIT D

BASELINE COST ESTIMATES/
CHARTS OF ACCOUNTS

					AMOUNT	CONTINGENCY	PROJECT COST
TOTAL ALL REACHES							
						ROUNDED	**354,826,000**
	TOTAL PROJECT COSTS				282,436,700	72,389,090	354,825,790
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	282,436,700	72,389,090	354,825,790
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	15,430,900	3,857,780	19,288,680			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	8,135,119	2,033,820	10,168,939			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	7,611,000	1,902,750	9,513,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	5,661,000	1,415,250	7,076,250			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	1,887,000	471,750	2,358,750			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	53,063,500	13,265,880	66,329,380			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**	0	0	0			
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	185,260,934	48,095,230	233,356,164			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01R2	**PL 91-646 ASSISTANCE PAYMENTS**						
01R2A	BY GOVERNMENT	0	0	0			
01R2B	BY LS	4,191,200	1,047,800	5,239,000			
01R2C	BY GOVT ON BEHALF OF LS	0	0	0			
01R2D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	253,050	63,270	316,320			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	348,500	87,180	435,680			

					AMOUNT	CONTINGENCY	PROJECT COST
REACH - BARRIER ALIGNMENT							
						ROUNDED	**3,702,000**
	TOTAL PROJECT COSTS				2,961,500	740,380	3,701,880
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	2,961,500	740,380	3,701,880
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	425,000	106,250	531,250			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	225,000	56,250	281,250			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	25,000	6,250	31,250			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	225,000	56,250	281,250			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	75,000	18,750	93,750			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	1,949,000	487,250	2,436,250			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	37,500	9,380	46,880			
ASSUMES 75 OWNERS							

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
REAL ESTATE DIVISION   REAL ESTATE CHART OF ACCOUNTS SUMMARY
(SEPARATED BY REACH)

REACH - A					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**896,000**
	TOTAL PROJECT COSTS				716,420	179,110	895,530
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	716,420	179,110	895,530
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	154,700	38,680	193,380			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	82,719	20,680	103,399			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	78,000	19,500	97,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	78,000	19,500	97,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	26,000	6,500	32,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	284,000	71,000	355,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	13,000	3,250	16,250			
ASSUMES 26 OWNERS							

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
REAL ESTATE DIVISION   REAL ESTATE CHART OF ACCOUNTS SUMMARY
(SEPARATED BY REACH)

REACH - B					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**1,045,000**
	TOTAL PROJECT COSTS				835,800	208,960	1,044,760
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	835,800	208,960	1,044,760
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	196,350	49,090	245,440			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	103,950	25,990	129,940			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	99,000	24,750	123,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	99,000	24,750	123,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	33,000	8,250	41,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	288,000	72,000	360,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	16,500	4,130	20,630			
ASSUMES 33 OWNERS							

					AMOUNT	CONTINGENCY	PROJECT COST
REACH E-1							
						ROUNDED	**435,000**
	TOTAL PROJECT COSTS				348,000	87,010	435,010
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	348,000	87,010	435,010
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	59,500	14,880	74,380			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	31,500	7,880	39,380			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	30,000	7,500	37,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	30,000	7,500	37,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	10,000	2,500	12,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	182,000	45,500	227,500			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	5,000	1,250	6,250			
ASSUMES 10 OWNERS							

REACH E-2					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**566,000**
	TOTAL PROJECT COSTS				453,000	113,260	566,260
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	453,000	113,260	566,260
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	59,500	14,880	74,380			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	31,500	7,880	39,380			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	30,000	7,500	37,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	30,000	7,500	37,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	10,000	2,500	12,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	287,000	71,750	358,750			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	5,000	1,250	6,250			
ASSUMES 10 OWNERS							

REACH F-1					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**1,139,000**
	TOTAL PROJECT COSTS				910,800	227,710	1,138,510
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	910,800	227,710	1,138,510
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	107,100	26,780	133,880			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	56,700	14,180	70,880			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	54,000	13,500	67,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	54,000	13,500	67,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	18,000	4,500	22,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	612,000	153,000	765,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	9,000	2,250	11,250			
ASSUMES 18 OWNERS							

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
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REACH F-2				AMOUNT	CONTINGENCY	PROJECT COST
					ROUNDED	**150,000**
	TOTAL PROJECT COSTS			119,800	29,950	149,750
01	**LANDS AND DAMAGES**	CONTINGENCY	PROJECT COST	119,800	29,950	149,750
01B	**ACQUISITIONS**					
01B10	BY GOVERNMENT	0	0	0		
01B20	BY LOCAL SPONSOR (LS)	17,850	4,460	22,310		
01B30	BY GOVT ON BEHALF OF LS	0	0	0		
01B40	REVIEW OF LS	9,450	2,360	11,810		
01C	**CONDEMNATIONS**					
01C10	BY GOVERNMENT	0	0	0		
01C20	BY LS	9,000	2,250	11,250		
01C30	BY GOVT ON BEHALF OF LS	0	0	0		
01C40	REVIEW OF LS	0	0	0		
01E	**APPRAISAL**					
01E10	BY GOVT (IN HOUSE)	0	0	0		
01E20	BY GOVT (CONTRACT)	0	0	0		
01E30	BY LS	9,000	2,250	11,250		
01E40	BY GOVT ON BEHALF OF LS	0	0	0		
01E50	REVIEW OF LS	3,000	750	3,750		
01F	**PL 91-646 ASSISTANCE**					
01F10	BY GOVERNMENT	0	0	0		
01F20	BY LS	0	0	0		
01F30	BY GOVT ON BEHALF OF LS	0	0	0		
01F40	REVIEW OF LS	0	0	0		
01R	**REAL ESTATE PAYMENTS**					
01R1	**LAND PAYMENTS**					
01R1A	BY GOVERNMENT	0	0	0		
01R1B	BY LS	70,000	17,500	87,500		
01R1C	BY GOVT ON BEHALF OF LS	0	0	0		
01R1D	REVIEW OF LS	0	0	0		
01T	**LERRD CREDITING**					
01T20	ADMINISTRATIVE COSTS	1,500	380	1,880		
ASSUMES 3 OWNERS						

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
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REACH G-1					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**198,000**
	TOTAL PROJECT COSTS				158,000	39,510	197,510
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	158,000	39,510	197,510
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT		0	0	0		
01B20	BY LOCAL SPONSOR (LS)		29,750	7,440	37,190		
01B30	BY GOVT ON BEHALF OF LS		0	0	0		
01B40	REVIEW OF LS		15,750	3,940	19,690		
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT		0	0	0		
01C20	BY LS		15,000	3,750	18,750		
01C30	BY GOVT ON BEHALF OF LS		0	0	0		
01C40	REVIEW OF LS		0	0	0		
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)		0	0	0		
01E20	BY GOVT (CONTRACT)		0	0	0		
01E30	BY LS		15,000	3,750	18,750		
01E40	BY GOVT ON BEHALF OF LS		0	0	0		
01E50	REVIEW OF LS		5,000	1,250	6,250		
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT		0	0	0		
01F20	BY LS		0	0	0		
01F30	BY GOVT ON BEHALF OF LS		0	0	0		
01F40	REVIEW OF LS		0	0	0		
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT		0	0	0		
01R1B	BY LS		75,000	18,750	93,750		
01R1C	BY GOVT ON BEHALF OF LS		0	0	0		
01R1D	REVIEW OF LS		0	0	0		
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS		2,500	630	3,130		
ASSUMES 5 OWNERS							

CEMVN-RE-E
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REACH G-2					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**233,000**
	TOTAL PROJECT COSTS				186,000	46,510	232,510
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	186,000	46,510	232,510
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	29,750	7,440	37,190			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	15,750	3,940	19,690			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	15,000	3,750	18,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	15,000	3,750	18,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	5,000	1,250	6,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	103,000	25,750	128,750			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	2,500	630	3,130			
ASSUMES 5 OWNERS							

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REACH G-3					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**154,000**
	TOTAL PROJECT COSTS				123,000	30,760	153,760
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	123,000	30,760	153,760
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	29,750	7,440	37,190			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	15,750	3,940	19,690			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	15,000	3,750	18,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	15,000	3,750	18,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	5,000	1,250	6,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	40,000	10,000	50,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	2,500	630	3,130			
ASSUMES 5 OWNERS							

REACH H-1					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**2,929,000**
	TOTAL PROJECT COSTS				2,343,200	585,800	2,929,000
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	2,343,200	585,800	2,929,000
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	89,250	22,310	111,560			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	47,250	11,810	59,060			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	45,000	11,250	56,250			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	45,000	11,250	56,250			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	15,000	3,750	18,750			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	21,000	5,250	26,250			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	1,430,000	357,500	1,787,500			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01R2	**PL 91-646 ASSISTANCE PAYMENTS**						
01R2A	BY GOVERNMENT	0	0	0			
01R2B	BY LS	643,200	160,800	804,000			
01R2C	BY GOVT ON BEHALF OF LS	0	0	0			
01R2D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	7,500	1,880	9,380			
ASSUMES 15 OWNERS							

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					AMOUNT	CONTINGENCY	PROJECT COST
REACH H-2							
						ROUNDED	**611,000**
	TOTAL PROJECT COSTS				489,000	122,250	611,250
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	489,000	122,250	611,250
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	119,000	29,750	148,750			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	63,000	15,750	78,750			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	60,000	15,000	75,000			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	60,000	15,000	75,000			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	20,000	5,000	25,000			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	157,000	39,250	196,250			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	10,000	2,500	12,500			
ASSUMES 20 OWNERS							

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REACH H-3					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**1,088,000**
	TOTAL PROJECT COSTS				870,900	217,470	1,088,370
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	870,900	217,470	1,088,370
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	208,250	52,060	260,310			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	110,250	27,560	137,810			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	105,000	26,250	131,250			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	105,000	26,250	131,250			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	35,000	8,750	43,750			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	212,000	53,000	265,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	78,400	19,600	98,000			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	17,000	4,000	21,000			
ASSUMES 35 OWNERS							

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					AMOUNT	CONTINGENCY	PROJECT COST
REACH I-1							
						ROUNDED	**322,000**
	TOTAL PROJECT COSTS				257,850	64,470	322,320
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	257,850	64,470	322,320
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	59,500	14,880	74,380			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	31,500	7,880	39,380			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	30,000	7,500	37,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	30,000	7,500	37,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	10,000	2,500	12,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	74,000	18,500	92,500			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	17,850	4,460	22,310			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	5,000	1,250	6,250			
ASSUMES 10 OWNERS							

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					AMOUNT	CONTINGENCY	PROJECT COST
REACH I-2							
						ROUNDED	**1,901,000**
	TOTAL PROJECT COSTS				1,520,990	380,260	1,901,250
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	1,520,990	380,260	1,901,250
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	196,350	49,090	245,440			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	103,950	25,990	129,940			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	99,000	24,750	123,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	99,000	24,750	123,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	33,000	8,250	41,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	3,000	750	3,750			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	864,694	216,170	1,080,864			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01R2	**PL 91-646 ASSISTANCE PAYMENTS**						
01R2A	BY GOVERNMENT	0	0	0			
01R2B	BY LS	37,600	9,400	47,000			
01R2C	BY GOVT ON BEHALF OF LS	0	0	0			
01R2D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	67,900	16,980	84,880			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	16,500	4,130	20,630			
ASSUMES 33 OWNERS							

REACH I-3						AMOUNT	CONTINGENCY	PROJECT COST
							ROUNDED	**1,203,000**
	TOTAL PROJECT COSTS					962,700	240,680	1,203,380
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST		962,700	240,680	1,203,380
01B	**ACQUISITIONS**							
01B10	BY GOVERNMENT	0	0	0				
01B20	BY LOCAL SPONSOR (LS)	255,850	63,960	319,810				
01B30	BY GOVT ON BEHALF OF LS	0	0	0				
01B40	REVIEW OF LS	135,450	33,860	169,310				
01C	**CONDEMNATIONS**							
01C10	BY GOVERNMENT	0	0	0				
01C20	BY LS	129,000	32,250	161,250				
01C30	BY GOVT ON BEHALF OF LS	0	0	0				
01C40	REVIEW OF LS	0	0	0				
01E	**APPRAISAL**							
01E10	BY GOVT (IN HOUSE)	0	0	0				
01E20	BY GOVT (CONTRACT)	0	0	0				
01E30	BY LS	129,000	32,250	161,250				
01E40	BY GOVT ON BEHALF OF LS	0	0	0				
01E50	REVIEW OF LS	43,000	10,750	53,750				
01F	**PL 91-646 ASSISTANCE**							
01F10	BY GOVERNMENT	0	0	0				
01F20	BY LS	0	0	0				
01F30	BY GOVT ON BEHALF OF LS	0	0	0				
01F40	REVIEW OF LS	0	0	0				
01R	**REAL ESTATE PAYMENTS**							
01R1	**LAND PAYMENTS**							
01R1A	BY GOVERNMENT	0	0	0				
01R1B	BY LS	160,000	40,000	200,000				
01R1C	BY GOVT ON BEHALF OF LS	0	0	0				
01R1D	REVIEW OF LS	0	0	0				
01O	**OYSTER LEASES**							
01O20	OYSTER LEASE ACQUISITION	88,900	22,230	111,130				
01T	**LERRD CREDITING**							
01T20	ADMINISTRATIVE COSTS	21,500	5,380	26,880				
ASSUMES 43 OWNERS								

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REACH J-1					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	0
	TOTAL PROJECT COSTS				0	0	0
01	LANDS AND DAMAGES		CONTINGENCY	PROJECT COST	0	0	0
01B	ACQUISITIONS						
01B10	BY GOVERNMENT		0	0	0		
01B20	BY LOCAL SPONSOR (LS)		0	0	0		
01B30	BY GOVT ON BEHALF OF LS		0	0	0		
01B40	REVIEW OF LS		0	0	0		
01C	CONDEMNATIONS						
01C10	BY GOVERNMENT		0	0	0		
01C20	BY LS		0	0	0		
01C30	BY GOVT ON BEHALF OF LS		0	0	0		
01C40	REVIEW OF LS		0	0	0		
01E	APPRAISAL						
01E10	BY GOVT (IN HOUSE)		0	0	0		
01E20	BY GOVT (CONTRACT)		0	0	0		
01E30	BY LS		0	0	0		
01E40	BY GOVT ON BEHALF OF LS		0	0	0		
01E50	REVIEW OF LS		0	0	0		
01F	PL 91-646 ASSISTANCE						
01F10	BY GOVERNMENT		0	0	0		
01F20	BY LS		0	0	0		
01F30	BY GOVT ON BEHALF OF LS		0	0	0		
01F40	REVIEW OF LS		0	0	0		
01R	REAL ESTATE PAYMENTS						
01R1	LAND PAYMENTS						
01R1A	BY GOVERNMENT		0	0	0		
01R1B	BY LS		0	0	0		
01R1C	BY GOVT ON BEHALF OF LS		0	0	0		
01R1D	REVIEW OF LS		0	0	0		
01O	OYSTER LEASES						
01O20	OYSTER LEASE ACQUISITION		0	0	0		
01T	LERRD CREDITING						
01T20	ADMINISTRATIVE COSTS		0	0	0		

ASSUMES ALL ROW IS OWNED BY POINTE AUX CHENES WMA AND
A GRANT OF PARTICULAR USE WILL BE ISSUED BY STATE OF LOUISIANA.

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
REAL ESTATE DIVISION   REAL ESTATE CHART OF ACCOUNTS SUMMARY
          (SEPARATED BY REACH)

REACH J-2				AMOUNT	CONTINGENCY	PROJECT COST
					ROUNDED	0
	TOTAL PROJECT COSTS			0	0	0
01	LANDS AND DAMAGES	CONTINGENCY	PROJECT COST	0	0	0
01B	ACQUISITIONS					
01B10	BY GOVERNMENT	0	0	0		
01B20	BY LOCAL SPONSOR (LS)	0	0	0		
01B30	BY GOVT ON BEHALF OF LS	0	0	0		
01B40	REVIEW OF LS	0	0	0		
01C	CONDEMNATIONS					
01C10	BY GOVERNMENT	0	0	0		
01C20	BY LS	0	0	0		
01C30	BY GOVT ON BEHALF OF LS	0	0	0		
01C40	REVIEW OF LS	0	0	0		
01E	APPRAISAL					
01E10	BY GOVT (IN HOUSE)	0	0	0		
01E20	BY GOVT (CONTRACT)	0	0	0		
01E30	BY LS	0	0	0		
01E40	BY GOVT ON BEHALF OF LS	0	0	0		
01E50	REVIEW OF LS	0	0	0		
01F	PL 91-646 ASSISTANCE					
01F10	BY GOVERNMENT	0	0	0		
01F20	BY LS	0	0	0		
01F30	BY GOVT ON BEHALF OF LS	0	0	0		
01F40	REVIEW OF LS	0	0	0		
01R	REAL ESTATE PAYMENTS					
01R1	LAND PAYMENTS					
01R1A	BY GOVERNMENT	0	0	0		
01R1B	BY LS	0	0	0		
01R1C	BY GOVT ON BEHALF OF LS	0	0	0		
01R1D	REVIEW OF LS	0	0	0		
01O	OYSTER LEASES					
01O20	OYSTER LEASE ACQUISITION	0	0	0		
01T	LERRD CREDITING					
01T20	ADMINISTRATIVE COSTS	0	0	0		

ASSUMES ALL ROW IS OWNED BY POINTE AUX CHENES WMA AND
A GRANT OF PARTICULAR USE WILL BE ISSUED BY STATE OF LOUISIANA.

CEMVN-RE-E
REAL ESTATE DIVISION

MORGANZA TO THE GULF PROJECT
REAL ESTATE CHART OF ACCOUNTS SUMMARY
(SEPARATED BY REACH)

OCTOBER 17, 2012

REACH J-3					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**1,120,000**
	TOTAL PROJECT COSTS				896,000	224,010	1,120,010
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	896,000	224,010	1,120,010
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	59,500	14,880	74,380			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	31,500	7,880	39,380			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	30,000	7,500	37,500			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	30,000	7,500	37,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	10,000	2,500	12,500			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	3,000	750	3,750			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	635,000	158,750	793,750			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01R2	**PL 91-646 ASSISTANCE PAYMENTS**						
01R2A	BY GOVERNMENT	0	0	0			
01R2B	BY LS	92,000	23,000	115,000			
01R2C	BY GOVT ON BEHALF OF LS	0	0	0			
01R2D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	5,000	1,250	6,250			
ASSUMES 10 OWNERS							

REACH K						AMOUNT	CONTINGENCY	PROJECT COST
							ROUNDED	**1,136,000**
	TOTAL PROJECT COSTS					908,600	227,150	1,135,750
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST		908,600	227,150	1,135,750
01B	**ACQUISITIONS**							
01B10	BY GOVERNMENT	0	0	0				
01B20	BY LOCAL SPONSOR (LS)	71,400	17,850	89,250				
01B30	BY GOVT ON BEHALF OF LS	0	0	0				
01B40	REVIEW OF LS	37,800	9,450	47,250				
01C	**CONDEMNATIONS**							
01C10	BY GOVERNMENT	0	0	0				
01C20	BY LS	36,000	9,000	45,000				
01C30	BY GOVT ON BEHALF OF LS	0	0	0				
01C40	REVIEW OF LS	0	0	0				
01E	**APPRAISAL**							
01E10	BY GOVT (IN HOUSE)	0	0	0				
01E20	BY GOVT (CONTRACT)	0	0	0				
01E30	BY LS	36,000	9,000	45,000				
01E40	BY GOVT ON BEHALF OF LS	0	0	0				
01E50	REVIEW OF LS	12,000	3,000	15,000				
01F	**PL 91-646 ASSISTANCE**							
01F10	BY GOVERNMENT	0	0	0				
01F20	BY LS	12,000	3,000	15,000				
01F30	BY GOVT ON BEHALF OF LS	0	0	0				
01F40	REVIEW OF LS	0	0	0				
01R	**REAL ESTATE PAYMENTS**							
01R1	**LAND PAYMENTS**							
01R1A	BY GOVERNMENT	0	0	0				
01R1B	BY LS	567,000	141,750	708,750				
01R1C	BY GOVT ON BEHALF OF LS	0	0	0				
01R1D	REVIEW OF LS	0	0	0				
01R2	**PL 91-646 ASSISTANCE PAYMENTS**							
01R2A	BY GOVERNMENT	0	0	0				
01R2B	BY LS	130,400	32,600	163,000				
01R2C	BY GOVT ON BEHALF OF LS	0	0	0				
01R2D	REVIEW OF LS	0	0	0				
01O	**OYSTER LEASES**							
01O20	OYSTER LEASE ACQUISITION	0	0	0				
01T	**LERRD CREDITING**							
01T20	ADMINISTRATIVE COSTS	6,000	1,500	7,500				
ASSUMES 12 OWNERS								

CEMVN-RE-E   MORGANZA TO THE GULF PROJECT   OCTOBER 17, 2012
REAL ESTATE DIVISION   REAL ESTATE CHART OF ACCOUNTS SUMMARY
(SEPARATED BY REACH)

	ESTIMATED FUTURE LIFT BORROW				AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**17,424,000**
	TOTAL PROJECT COSTS				12,515,000	4,908,760	17,423,760
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	12,515,000	4,908,760	17,423,760
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	1,933,750	483,440	2,417,190			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	1,023,750	255,940	1,279,690			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	975,000	243,750	1,218,750			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	975,000	243,750	1,218,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	325,000	81,250	406,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	0	0	0			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	7,120,000	3,560,000	10,680,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01O	**OYSTER LEASES**						
01O20	OYSTER LEASE ACQUISITION	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	162,500	40,630	203,130			
	ASSUMES 3,250 ACRES ADDITIONAL BORROW REQUIRED AND ESTIMATED 325 LANDOWNERS						

CEMVN-RE-E
REAL ESTATE DIVISION

MORGANZA TO THE GULF PROJECT
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						AMOUNT	CONTINGENCY	PROJECT COST
LAROSE FLOODGATE								
							ROUNDED	**159,000**
	TOTAL PROJECT COSTS					127,200	31,810	159,010
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST		127,200	31,810	159,010
01B	**ACQUISITIONS**							
01B10	BY GOVERNMENT	0	0	0				
01B20	BY LOCAL SPONSOR (LS)	11,900	2,980	14,880				
01B30	BY GOVT ON BEHALF OF LS	0	0	0				
01B40	REVIEW OF LS	6,300	1,580	7,880				
01C	**CONDEMNATIONS**							
01C10	BY GOVERNMENT	0	0	0				
01C20	BY LS	6,000	1,500	7,500				
01C30	BY GOVT ON BEHALF OF LS	0	0	0				
01C40	REVIEW OF LS	0	0	0				
01E	**APPRAISAL**							
01E10	BY GOVT (IN HOUSE)	0	0	0				
01E20	BY GOVT (CONTRACT)	0	0	0				
01E30	BY LS	6,000	1,500	7,500				
01E40	BY GOVT ON BEHALF OF LS	0	0	0				
01E50	REVIEW OF LS	2,000	500	2,500				
01F	**PL 91-646 ASSISTANCE**							
01F10	BY GOVERNMENT	0	0	0				
01F20	BY LS	3,000	750	3,750				
01F30	BY GOVT ON BEHALF OF LS	0	0	0				
01F40	REVIEW OF LS	0	0	0				
01R	**REAL ESTATE PAYMENTS**							
01R1	**LAND PAYMENTS**							
01R1A	BY GOVERNMENT	0	0	0				
01R1B	BY LS	91,000	22,750	113,750				
01R1C	BY GOVT ON BEHALF OF LS	0	0	0				
01R1D	REVIEW OF LS	0	0	0				
01O	**OYSTER LEASES**							
01O20	OYSTER LEASE ACQUISITION	0	0	0				
01T	**LERRD CREDITING**							
01T20	ADMINISTRATIVE COSTS	1,000	250	1,250				
ASSUMES 2 OWNERS								

CEMVN-RE-E
REAL ESTATE DIVISION

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INDUCED FLOODING BUY-OUT ESTIMATE					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**305,115,000**
	TOTAL PROJECT COSTS				244,092,240	61,023,060	305,115,300
01	LANDS AND DAMAGES		CONTINGENCY	PROJECT COST	244,092,240	61,023,060	305,115,300
01B	ACQUISITIONS						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	9,595,000	2,398,750	11,993,750			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	5,050,000	1,262,500	6,312,500			
01C	CONDEMNATIONS						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	5,000,000	1,250,000	6,250,000			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	APPRAISAL						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	3,030,000	757,500	3,787,500			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	1,010,000	252,500	1,262,500			
01F	PL 91-646 ASSISTANCE						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	52,818,000	13,204,500	66,022,500			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	REAL ESTATE PAYMENTS						
01R1	LAND PAYMENTS						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	167,084,240	41,771,060	208,855,300			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01T	LERRD CREDITING						
01T20	ADMINISTRATIVE COSTS	505,000	126,250	631,250			
	ASSUMES 1010 OWNERS						

REACH - LAROSE TO LOCKPORT - 1% ALTERNATIVE				AMOUNT	CONTINGENCY	PROJECT COST
					ROUNDED	**2,931,000**
	TOTAL PROJECT COSTS			2,345,000	586,260	2,931,260
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	2,345,000	586,260	2,931,260
01B	**ACQUISITIONS**					
01B10	BY GOVERNMENT	0	0	0		
01B20	BY LOCAL SPONSOR (LS)	712,500	178,130	890,630		
01B30	BY GOVT ON BEHALF OF LS	0	0	0		
01B40	REVIEW OF LS	375,000	93,750	468,750		
01C	**CONDEMNATIONS**					
01C10	BY GOVERNMENT	0	0	0		
01C20	BY LS	300,000	75,000	375,000		
01C30	BY GOVT ON BEHALF OF LS	0	0	0		
01C40	REVIEW OF LS	0	0	0		
01E	**APPRAISAL**					
01E10	BY GOVT (IN HOUSE)	0	0	0		
01E20	BY GOVT (CONTRACT)	0	0	0		
01E30	BY LS	225,000	56,250	281,250		
01E40	BY GOVT ON BEHALF OF LS	0	0	0		
01E50	REVIEW OF LS	75,000	18,750	93,750		
01F	**PL 91-646 ASSISTANCE**					
01F10	BY GOVERNMENT	0	0	0		
01F20	BY LS	0	0	0		
01F30	BY GOVT ON BEHALF OF LS	0	0	0		
01F40	REVIEW OF LS	0	0	0		
01R	**REAL ESTATE PAYMENTS**					
01R1	**LAND PAYMENTS**					
01R1A	BY GOVERNMENT	0	0	0		
01R1B	BY LS	620,000	155,000	775,000		
01R1C	BY GOVT ON BEHALF OF LS	0	0	0		
01R1D	REVIEW OF LS	0	0	0		
01R2	**PL 91-646 ASSISTANCE PAYMENTS**					
01R2A	BY GOVERNMENT	0	0	0		
01R2B	BY LS	0	0	0		
01R2C	BY GOVT ON BEHALF OF LS	0	0	0		
01R2D	REVIEW OF LS	0	0	0		
01T	**LERRD CREDITING**					
01T20	ADMINISTRATIVE COSTS	37,500	9,380	46,880		
ASSUMES 75 OWNERS						

CEMVN-RE-E
REAL ESTATE DIVISION

MORGANZA TO THE GULF PROJECT
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REACH - LAROSE C NORTH					AMOUNT	CONTINGENCY	PROJECT COST
						ROUNDED	**10,088,000**
	TOTAL PROJECT COSTS				8,070,500	2,017,640	10,088,140
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST	8,070,500	2,017,640	10,088,140
01B	**ACQUISITIONS**						
01B10	BY GOVERNMENT	0	0	0			
01B20	BY LOCAL SPONSOR (LS)	997,500	249,380	1,246,880			
01B30	BY GOVT ON BEHALF OF LS	0	0	0			
01B40	REVIEW OF LS	525,000	131,250	656,250			
01C	**CONDEMNATIONS**						
01C10	BY GOVERNMENT	0	0	0			
01C20	BY LS	420,000	105,000	525,000			
01C30	BY GOVT ON BEHALF OF LS	0	0	0			
01C40	REVIEW OF LS	0	0	0			
01E	**APPRAISAL**						
01E10	BY GOVT (IN HOUSE)	0	0	0			
01E20	BY GOVT (CONTRACT)	0	0	0			
01E30	BY LS	315,000	78,750	393,750			
01E40	BY GOVT ON BEHALF OF LS	0	0	0			
01E50	REVIEW OF LS	105,000	26,250	131,250			
01F	**PL 91-646 ASSISTANCE**						
01F10	BY GOVERNMENT	0	0	0			
01F20	BY LS	203,500	50,880	254,380			
01F30	BY GOVT ON BEHALF OF LS	0	0	0			
01F40	REVIEW OF LS	0	0	0			
01R	**REAL ESTATE PAYMENTS**						
01R1	**LAND PAYMENTS**						
01R1A	BY GOVERNMENT	0	0	0			
01R1B	BY LS	2,164,000	541,000	2,705,000			
01R1C	BY GOVT ON BEHALF OF LS	0	0	0			
01R1D	REVIEW OF LS	0	0	0			
01R2	**PL 91-646 ASSISTANCE PAYMENTS**						
01R2A	BY GOVERNMENT	0	0	0			
01R2B	BY LS	3,288,000	822,000	4,110,000			
01R2C	BY GOVT ON BEHALF OF LS	0	0	0			
01R2D	REVIEW OF LS	0	0	0			
01T	**LERRD CREDITING**						
01T20	ADMINISTRATIVE COSTS	52,500	13,130	65,630			
ASSUMES 105 OWNERS							

CEMVN-RE-E
REAL ESTATE DIVISION

MORGANZA TO THE GULF PROJECT
REAL ESTATE CHART OF ACCOUNTS SUMMARY
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						AMOUNT	CONTINGENCY	PROJECT COST
REACH L								
							ROUNDED	**282,000**
	TOTAL PROJECT COSTS					225,200	56,310	281,510
01	**LANDS AND DAMAGES**		CONTINGENCY	PROJECT COST		225,200	56,310	281,510
01B	**ACQUISITIONS**							
01B10	BY GOVERNMENT	0	0	0				
01B20	BY LOCAL SPONSOR (LS)	11,900	2,980	14,880				
01B30	BY GOVT ON BEHALF OF LS	0	0	0				
01B40	REVIEW OF LS	6,300	1,580	7,880				
01C	**CONDEMNATIONS**							
01C10	BY GOVERNMENT	0	0	0				
01C20	BY LS	6,000	1,500	7,500				
01C30	BY GOVT ON BEHALF OF LS	0	0	0				
01C40	REVIEW OF LS	0	0	0				
01E	**APPRAISAL**							
01E10	BY GOVT (IN HOUSE)	0	0	0				
01E20	BY GOVT (CONTRACT)	0	0	0				
01E30	BY LS	6,000	1,500	7,500				
01E40	BY GOVT ON BEHALF OF LS	0	0	0				
01E50	REVIEW OF LS	2,000	500	2,500				
01F	**PL 91-646 ASSISTANCE**							
01F10	BY GOVERNMENT	0	0	0				
01F20	BY LS	0	0	0				
01F30	BY GOVT ON BEHALF OF LS	0	0	0				
01F40	REVIEW OF LS	0	0	0				
01R	**REAL ESTATE PAYMENTS**							
01R1	**LAND PAYMENTS**							
01R1A	BY GOVERNMENT	0	0	0				
01R1B	BY LS	192,000	48,000	240,000				
01R1C	BY GOVT ON BEHALF OF LS	0	0	0				
01R1D	REVIEW OF LS	0	0	0				
01O	**OYSTER LEASES**							
01O20	OYSTER LEASE ACQUISITION	0	0	0				
01T	**LERRD CREDITING**							
01T20	ADMINISTRATIVE COSTS	1,000	250	1,250				
ASSUMES 2 OWNERS								

EXHIBIT E

QUALITY CONTROL CHECKLIST

Quality Control Plan Checklist

**MISSISSIPPI RIVER AND TRIBUTARIES
MORGANZA TO THE GULF OF MEXICO
HURRICANE PROTECTION PROJECT
TERREBONNE AND LAFOURCHE PARISHES, LA**

REAL ESTATE PLAN

1. **Purpose of the REP**. __√___
    a. Describe the purpose of the REP in relation to the project document that it supports.
    b. Describe the project for the Real Estate reviewer.
    c. Describe any previous REPs for the project.

2. **Describe LER**. __√___
    a. Account for all lands, easements, and rights-of-way underlying and required for the construction, OMRR&R of the project, including mitigation, relocations, borrow material and dredged or excavated material disposal, whether or not it will need to be acquired or will be credited to the NFS.
    b. Provide description of total LER required for each project purpose and feature.
    c. Include LER already owned by the Government, the NFS and within the navigation servitude.
    d. Show acreage, estates, number of tracts and ownerships, and estimated value.
    e. Break down total acreage into fee and the various types and durations of easements.
    f. Break down acreage by Government, NFS, other public entity, and private ownership, and lands within the navigation servitude.

3. **NFS-Owned LER**.__√___
    a. Describe NFS-owned acreage and interest and whether or not it is sufficient and available for project requirements.
    b. Discuss any crediting issues and describe NFS views on such issues.

4. Include any proposed **Non-Standard Estates**. __√___
    a. Use Standard Estates where possible.
    b. Non-standard estates must be approved by HQ to assure they meet DOJ standards for use in condemnations.
    c. Provide justification for use of the proposed non-standard estates.
    d. Request approval of the non-standard estates as part of document approval.
    e. If the document is to be approved at MSC level, the District must seek approval of the non-standard estate by separate request to HQ. This should be stated in the REP.
    f. Exception to HQ approval is District Chiefs of RE approval of non-standard estate if it serves intended project purposed, substantially conforms with and does not materially deviate from the standard estates found in the RE Handbook, and does not increase cost or potential

liability to the Government. A copy of this approval should be included in the REP. (See Section 12-10c. of RE 405-1-12)

g. Although estates are discussed generally in topic 2, it is a good idea to also state in this section which standard estates are to be acquired and attach a copy as an appendix. The duration of any temporary estates should be stated.

5. **Existing Federal Projects**. __√__

a. Discuss whether there is any existing Federal project that lies fully of partially within LER required for the project.

b. Describe the existing project, all previously-provided interests that are to be included in the current project, and identify the sponsor.

c. Interest in land provided as an item of local cooperation for a previous Federal project is not eligible for credit.

d. Additional interest in the same land is eligible for credit.

6. **Federally-Owned Lands** __√__

a. Discuss whether there is any Federally owned land included within the LER required for the project.

b. Describe the acreage and interest owned by the Government.

c. Provide description of the views of the local agency representatives toward use of the land for the project and issues raised by the requirement for this land.

7. **Navigation Servitude**. __√__

a. Identify LER required for the project that lies below the Ordinary High Water Mark, or Mean High Water Mark, as the case may be, of a navigable watercourse.

b. Discuss whether navigation servitude is available

c. Will it be exercised for project purposes? Discuss why or why not.

d. Lands over which the navigation servitude is exercised are not to be acquired nor eligible for credit for a Federal navigation or flood control project or other project to which a navigation nexus can be shown.

e. See paragraph 12-7 of ER 405-1-12.

8. **Map** __√__

a. An aid to understanding

b. Clearly depicting project area and tracts required, including existing LER, LER to be acquired, and lands within the navigation servitude.

c. Depicts significant utilities and facilities to be relocated, any known or potential HTRW lands.

9. **Induced Flooding** can create a requirement for real estate acquisition. ___√__

a. Discuss whether there will be flooding induced by the construction and OMRR&R of the project.

b. If reasonably anticipated, describe nature, extent and whether additional acquisition of LER must or should occur.

c. Physical Takings Analysis (separate from the REP) must be done if significant induced flooding anticipated considering depth, frequency, duration, and extent of induced flooding.

.   d. Summarize findings of Takings Analysis in REP. Does it rise to the level of a taking for which just compensation is owed?

10. **Baseline Cost Estimate** as described in paragraph 12-18. __√__
        a. Provides information for the project cost estimates.
        b. Gross Appraisal includes the fair market value of all lands required for project construction and OMRR&R.
        c. PL 91-646 costs
        d. Incidental acquisition costs
        e. Incremental real estate costs discussed/supported.
        f. Is Gross Appraisal current? Does Gross Appraisal need to be updated due to changes in project LER requirements or time since report was prepared?

11. **Relocation Assistance Benefits** Anticipated. __√__
        a. Number of persons, farms, and businesses to be displaced and estimated cost of moving and reestablishment.
        b. Availability of replacement housing for owners/tenants
        c. Need for Last Resort Housing benefits
        d. Real Estate closing costs
        e. See current 49 CFR Part 24

12. **Mineral Activity.** __√__
        a. Description of present or anticipated mineral activity in vicinity that may affect construction, OMRR&R of project.
        b. Recommendation, including rationale, regarding acquisition of mineral rights or interest, including oil or gas.
        c. Discuss other surface or subsurface interests/timber harvesting activity
        d. Discuss effect of outstanding 3rd party mineral interests.
        e. Does estate properly address mineral rights in relation to the project?

13. **NFS Assessment** __√__
        a. Assessment of legal and professional capability and experience to acquire and provide LER for construction, OMRR&R of the Project.
        b. Condemnation authority
        c. Quick-take capability
        d. NFS advised of URA requirements
        e. NFS advised of requirements for documenting expenses for credit.
        f. If proposed that Government will acquire project LER on behalf of NFS, fully explain the reasons for the Government performing work.
        g. A copy of the signed and dated Assessment of Non-Federal Sponsor's Real Estate Acquisition Capability (Appendix 12-E) is attached to the REP.

14. **Zoning** in Lieu of Acquisition __√__
        a. Discuss type and intended purpose
        b. Determine whether the proposed zoning proposal would amount to a taking for which compensation will be due.

15. **Schedule** __√___
     a. Reasonable and detailed Schedule of land acquisition milestones, including LER certification.
     b. Dates mutually agreed upon by Real Estate, PM, and NFS. _____

16. **Facility or Utility Relocations** __√___
     a. Describe the relocations, identity of owners, purpose of facilities/utilities, whether owners have compensable real property interest.
     b. A synopsis of the findings of the Preliminary Attorney's Investigation and Report of Compensable Interest is included in the REP as well as statements required by Sections 12-17c.(5) and (6).
     c. Erroneous determinations can affect the accuracy of the project cost estimate and can confuse Congressional authorization.
     d. Eligibility for substitute facility
          1. Project impact
          2. Compensable interest
          3. Public utility or facility
          4. Duty to replace
          5. Fair market value too difficult to determine or its application would result in an injustice to the landowner or the public.
     e. See Sections 12-8, 12-17, and 12-22 of ER 405-1-12.

17. **HTRW** and Other Environmental Considerations __√___
     a. Discussion the impacts on the Real Estate acquisition process and LER value estimate due to known or suspected presence of contaminants.
     b. Status of District's investigation of contaminants.
     c. Are contaminants regulated under CERCLA, other statues, or State law?
     d. Is clean-up or other response required of non-CERCLA regulated material?
     e. If cost share, who is responsible for performing and paying cost of work?
     f. Status of NEPA and NHPA compliances
     g. See ER 1165-2-132, Hazardous, Toxic, and Radioactive Waste (HTRW) Guidance for Civil Works Projects.

18. **Landowner Attitude**. _√___
     a. Is there support, apathy, or opposition toward the project?
     b. Discuss any landowner concerns on issues such as condemnation, willing seller provisions, estates, acreages, etc.?

19. A statement that the **NFS has been notified in writing about the risks of acquiring LER before the execution of the PPA.** If not applicable, so state. _√___

20. **Other Relevant Real Estate Issues.** Anything material to the understanding of the RE aspects of the project. ___√___

A copy of the completed Checklist is attached to the REP. _√___

(Draft REPs must contain a draft checklist and draft Technical Review Guide)

I have prepared and thoroughly reviewed the REP and all information, as required by Section 12-16 of ER 405-1-12, is contained in the Plan.

Karen Vance   _1-24-12_
Preparer   Date

A copy of the Real Estate Internal Technical Review Guide for Civil Works Decision Documents is attached and signed by me as the Reviewer

Judith Lutro   _1/24/12_
RE Internal Technical Reviewer   Date

The REP has been signed and dated by the Preparer and the District Chief of Real Estate.
___√_____