WEST SHORE LAKE PONTCHARTRAIN
HURRICANE AND STORM DAMAGE RISK REDUCTION STUDY
FINAL INTEGRATED FEASIBILITY REPORT
AND
ENVIRONMENTAL IMPACT STATEMENT

ENGINEERING
APPENDIX B

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General

The Study area is located west of the Bonnet Carre Spillway between the Mississippi River and Lakes Pontchartrain and Maurepas in Southeast Louisiana. The project's purpose is to provide hurricane and storm damage risk reduction to developed areas of St. Charles, St. John the Baptist and St. James Parishes. Three structural levee alignments (Levee Alignments A, C and D) were evaluated (each with several features, including levees, floodwalls, floodgates and pumping stations) in order to select the best approach to reduce hurricane/tropical storm surge (hereafter “storm surge”) in communities throughout the study area. Each alternative also evaluated environmental measures designed to protect and/or minimize the impacts to nearby wetlands and transportation evacuation routes (such as I-10 and U.S. 61) located in the study area. More information on the alternatives that were considered can be found in the **Screening Phase (Background) Information** section of this Appendix.

Information provided herein describes the details of the Levee System of the Recommended Plan (drawings of the alignment, known as Alignment C, can be found in Annex 3 of this Appendix). Details on the final design of the localized storm surge risk reduction system are incorporated into Chapter 5 of the main report and at the end of the Plan Formulation Appendix. The Recommended Plan is based on modeling for a 100-year level of risk reduction in the Baseline Year of 2020. This is also known as the base year and is part of a 50 year planning horizon that is generally used for U. S. Army Corps of Engineers (USACE) projects. The year 2020 was decided as the base year for economic and hydraulic conditions since it is possible that the proposed levee could be designed and constructed by then with sufficient funding and authorization.

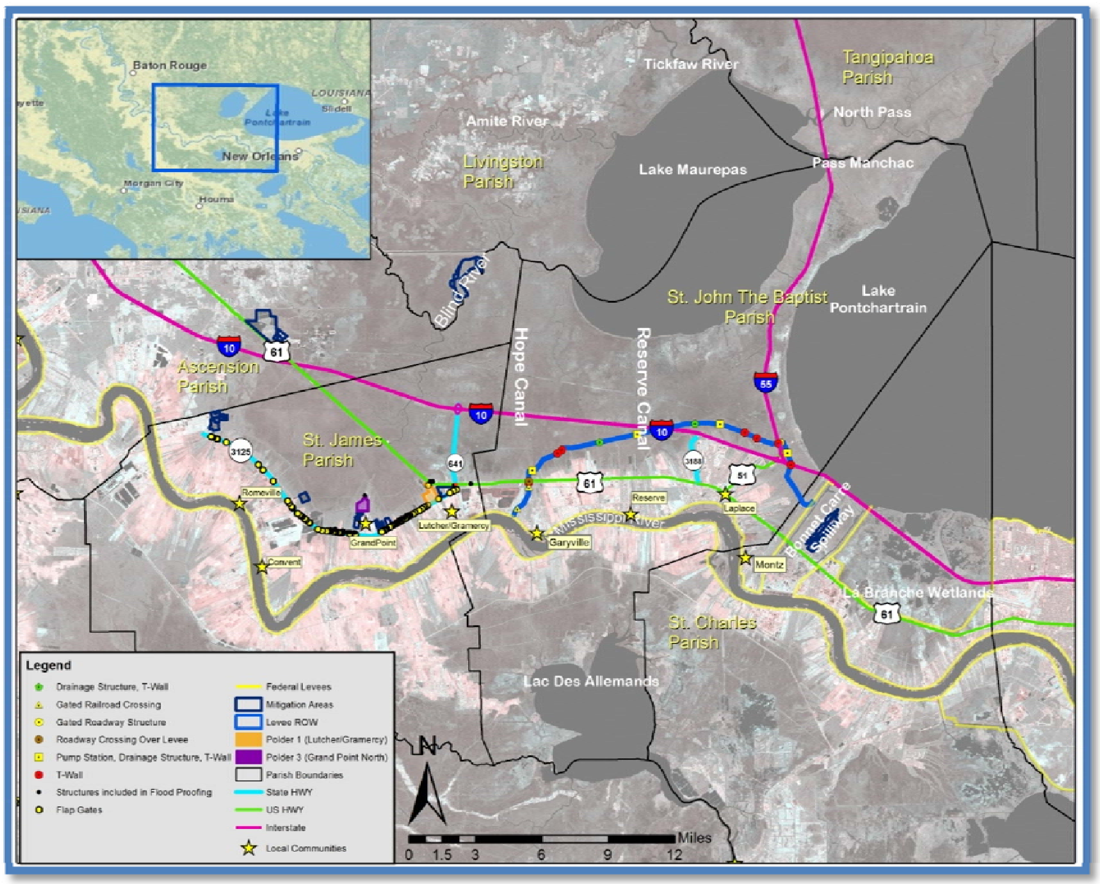


Figure 1: Overview of Risk Reduction System (Alignment C)

The Recommended Plan (known as Alignment C) for the West Shore-Lake Pontchartrain (WSLP) study includes the construction of an 18.27-mile (96,500 ft) levee system around the communities of Montz, LaPlace, Reserve and Garyville. This system also includes the construction of localized storm surge risk reduction measures in St. James Parish. An overview of the entire risk reduction system is shown in Figure 1.

Levee System

The levee system would begin at the upper guide levee of the Bonnet Carre Spillway, north of an underground utility pipeline right of way and US-61. The levee would head northwest paralleling the pipeline right of way and pass under I-10. Past I-10, the levee would enclose the I-10 and I-55 interchange and cross US-51. It would then track north of I-10 and a pipeline transmission corridor. Past the Belle Terre/I-10 exit, the levee would pass back under I-10 and parallel the pipeline corridor through wetlands until it crosses Hope Canal. The levee would then turn south; cross the pipeline transmission corridor and then extend to the Mississippi River Levee System (MRL)

The levee system would reduce the risk of flooding for over 7,000 structures and four miles of I-10 located in the system. Inclusion of this segment of I-10 could allow for an earlier re-entry route for residents and emergency responders in southeast Louisiana, including residents in the New Orleans metropolitan area.

The construction of the structural component of the project, hereafter referred to as the “levee system”, would be based on a 1% probability storm level of risk reduction and a 2020 intermediate RSLR condition. In order to maintain the 1% probability storm level of risk reduction system over the period of evaluation (50 years) the levee system would include future levee lifts based on the 2070 intermediate RSLR conditions. For example, at the starting point of the upper guide levee of the Bonnet Carre Spillway the levee would be constructed to a top of levee elevation of 15 ft. NAVD88 in 2020. In the future, the levee at this point would be lifted to a final elevation of 19.5 ft. NAVD88 based on the 2070 intermediate RSLR conditions. This is the highest elevation point of the constructed levee system. The levee would start at this height and taper down to a final top levee elevation of 8.5 ft. NAVD88 near the MRL. The final 2070 top levee elevation near the MRL would be 16 ft. NAVD88.

The system would consist of earthen levees, floodwalls (T-Walls), floodgates, drainage canals, a flood-side ditch for hydraulic connectivity for wetlands north and south of the system, drainage structures and pump stations along the alignment, and mitigation measures (Figure 5-2). Structures through the levee would be built to the 2070 intermediate RSLR condition, to prevent costly future retrofits required for anticipated changing sea levels.

Starting at the upper guide levee of the Bonnet Carre Spillway and heading west along the levee, the project would construct a 646 linear foot (hereafter “LF”) T-Wall to pass under the existing I-10 overpass. Past this point, an 1,100 cubic feet per second (cfs) pump station with three 68" outfalls would be built at Montz Canal, which is very near the I-55 northbound entrance ramp. The pump station, when the system is closed, would mainly remove rainwater flows from the Woodland, the River Forest and the Prescott Canals. A 267 LF T-Wall and two 6' x 18' x 27' gated drainage structures would also be constructed at this location. This location and all locations with pump stations or drainage structures would be connected to a flood side ditch and a protected side canal that would parallel the entire levee length. The canals would be used to maintain the existing connection between swamps located inside and outside of the levee system. The protected side canal would also serve as a redundancy connection if one of the pump stations failed during an event.

Past the Montz Canal, at the location of US-51, a 188 LF gated structure would be placed through the levee. Directly west of US-51, a 247 LF T-Wall would cross under I-55. The levee would continue to the west until the levee intercepts the first pipeline crossings near Vicknair Canal. Two sections of T-Walls would be used for these pipeline crossing, a 550 LF T-Wall, and a 623 LF T-Wall. Half of the 35 required pipeline relocations would be at these two locations. For purposes of this report, it is expected that all of the pipeline relocations would be compensable. Relocations are expected to take place in the proposed levee right-of-way (ROW) or existing pipeline ROW. Determination of the compensability of these relocations will be determined during the engineering and design phase of this project if it is authorized.

Continuing west, the levee would then cross Ridgefield Canal. Ridgefield Canal is located between the I-10 Louisiana Department of Transportation and Development (LADOTD) weigh station and the I-10/LA 3188 exit. A 200 cfs pump station with three 30" outfalls would be built at Ridgefield Canal. The pump station, when the system is closed, would mainly remove rainfall flows from Laplace Plantation, Perriloux, Ridgefield, Tebo and Vicknair canals. A 244 LF T-Wall with two 6' x 18' x 267' gated drainage structures would also be constructed at this location.

West of the Ridgefield Canal, a 100 LF floodgate would be constructed at the location of the Perriloux Canal to allow rainfall flows to flow through the levee when the system is not closed.

West of the I-10/LA 3188 exit, a 247 LF T-Wall would be constructed to cross back under I-10. The levee would continue to parallel the pipeline corridor through wetlands until it reaches Reserve canal. A 400 cfs pump station with three 48" outfalls would be built at this location. The structures at this location would also include two 6' x 20' x 25' drainage structures with a boat bay and 335 LF of T-Walls. Small boats would still be able to pass through the drainage structure when the system is open.

Continuing west, the levee would then cross Mississippi Bayou. A 6' x 10' x 25' drainage structure with a 267 LF T-Wall would be constructed at this location.

The levee would then continue west toward Hope Canal, until it reaches the next major set of pipeline crossings. All of the remaining major pipeline relocations would be at this location. Two sections of T-Walls would be used for these pipeline crossings: a 400 LF T-Wall and a 300 LF T-Wall. As with the other pipelines, for purposes of this report, it is expected that the pipeline relocations would be compensable. Relocations are expected to take place in the proposed levee ROW or existing pipeline ROW at this location. Determination of the compensability of these relocations will be determined during the engineering and design phase of this project if it is authorized.

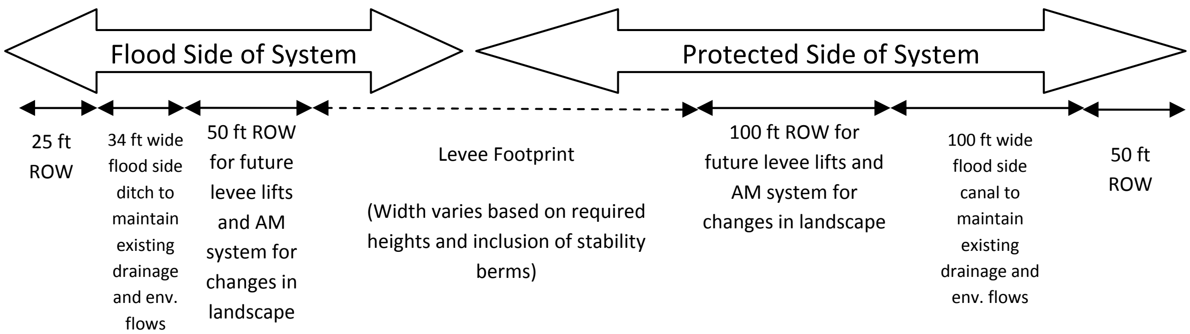
The levee would then continue west until it reaches Hope Canal. A 450 cfs pump station with three 54" outfalls would be constructed at this location. Currently, the design and cost includes a 6' x 20' x 25' drainage structure and a 247 LF T-Wall, but the Hope Canal location is also the same location of the State of Louisiana's proposed Mississippi Reintroduction into Maurepas Swamp diversion. The WSLP project has been coordinating activities between the project development teams, but for the purposes of the WSLP feasibility design, we do not consider the diversion project as a future landscape feature, since the State has not identified funding and has filed an incomplete permit application to USACE for construction of the project. USACE would continue to monitor the status of the diversion project. The team expects that if the diversion project moves forward it would be constructed on the flood side of the levee and would parallel the levee from Hope Canal to the MRL.

When the levee turns south, past Hope Canal to tie into the MRL, the levee would cross US-61, a pipeline ROW, and two railroad tracks. US-61 would be raised to hump over the levee at the crossing point. The pipeline crossing would include a 301 LF T-Wall, while the two railroad crossings would include a 150 LF gate structure and a 50 LF gate structure.

In all, there would be a total of 5,001 LF of T-Walls, 4 pump stations with associated drainage structures, 2 drainage structures, one gated road crossing, and 2 gated railroad crossings.

4.69 miles of the upper guide levee of the Bonnet Carre Spillway from the spillway control structure to the WSLP tie-in point would be included in the WSLP levee system, but there would be no construction activities associated with this Bonnet Carre levee. Existing levee heights are high enough to prevent 1% probability storm surge from entering the WSLP system during storms. The construction of the WSLP tie-in point would be to set to elevation of 15 ft. NAVD88 while the current upper guide levee elevation is 15.5 ft. NAVD88. The upper guide levee heights in the future would be monitored to determine if sections of the Bonnet Carre Spillway levee would need future lifts to prevent overtopping of storm surges into the WSLP system.

All levee right-of-ways would have the following typical dimensions:



The 50-ft. and 100-ft. rights-of-way adjacent to the levee footprints would be used for future levee lifts. The levee would be lifted five times over the period of evaluation. The first two lifts would be used to obtain a 1% probability storm level of risk reduction system in 2020. Additional levee lifts to maintain a 1% probability storm level of risk reduction system would take place in years 2030, 2045 and in 2060.

9,000,000 cubic yards (cy) of compacted fill and un-compacted fill would be required to create and maintain the levee over the period of evaluation. A portion of the initial fill material, if suitable, would be obtained from the canals and ditch, approximately 1,678,000 cy. Borings indicate that the top 4 ft of the cross section of these features would not be suitable as levee fill material. The top 4 ft of material; approximately 1,685,000 cy, would be used beneficially at mitigation plan sites, or disposed of appropriately by the contractor. The remaining fill for the levee, approximately 7,322,000 cy, would be obtained from the Bonnet Carre Spillway.

The levee footprint would vary based on the designed cross section and required top of levee heights by each levee section. The top of the levee would have a 10’ wide crown and the protected side of the levee system would be based on a 1:3 side slope, with some reaches including a geotechnical stability berm. 3,400,000 square yards of geotextile fabric would be placed under the levee footprint and approximately 80,000 cubic yards of aggregate limestone would be used to build a road on the levee crown.

The total levee construction ROW would be 1,235 acres. Real Estate agreements would be acquired on all features. A perpetual flood protection levee easement would be acquired for the 669 acres of the levee and floodwall features. A perpetual flood protection levee easement would be acquired for the 33 acres of the T-Walls. For the two canals, a 519-acre perpetual drainage ditch easement would be acquired. For the remaining features, the 4 pump stations would require 9 acres and the 3 gated crossings would require 5 acres (to be acquired based on fee, excluding minerals). In addition to the permanent easements, 49 acres of temporary access easements and 12 acres of temporary work area easements would be acquired. These temporary access and work access areas would be on existing roadways or developed areas of the project area and would not be in environmentally sensitive areas.

All of the impacts from the constructed features would be to either swamp habitats or Bottom Land Hardwood (BLH). There would be a direct removal of 1,112 acres of swamp habitats and 123 acres of BLH habitats. Using a wetland value assessment (WVA) under the intermediate sea level rise scenario the project would be required to mitigate for a direct loss of 595.3 average annual habitat units (AAHUs) of swamp and 95.5 AAHUs of BLH. In addition to the direct removal of acres of habitat due to construction, the project would enclose 8,432 acres of swamp and 89 acres of BLH.

Hydrologic connectivity would be maintained to the extent practicable through water control structures except during closure for hurricanes or tropical storms. When the system is closed, pumps would operate on average for 1.7 storms per year, which equates to a closure of structures on average 8.5 days per year. This expected rate of closure would be the same regardless of the actual rate of RSLR as closure of the system is tied to tropical storm events and the elevation trigger would be adjusted as sea level rises. The risk reduction system is only authorized to address storm surge caused by hurricane and tropical storm events. It is not authorized to mitigate for or reduce impacts caused by higher day-to-day water levels brought about by increases in sea level rise alone. Any operational changes implemented to address changing SLR conditions or for any other non-project-related purpose would be considered a separate project purpose requiring separate authorization, new NEPA documentation, and / or permit approvals.

The levee is designed to maintain hydrologic connectivity to the extent practicable. In order to minimize a reduction in efficiency of drainage affecting water quality and increased impoundment on the protected side of the system, the levee design includes drainage structures and canals located on both the flood side and protected side of the levee. In order to mitigate for any impacts caused by the potential delay in water movement, the team developed a WVA that accounts for delays in water movement. Because 366 acres of the total 455 acres of enclosed BLH is already impacted by existing roadways and railroad tracks, the BLH indirect impacts were calculated to total 89 acres. Using a WVA under the intermediate RSLR scenario, the project would have to mitigate for the indirect loss of 494.5 AAHUs of swamp and 3.1 AAHUs of BLH. The project would also be required to mitigate for a direct loss of 595.3 AAHUs of swamp and 95.5 AAHUs of BLH. The total required mitigation for both the direct and indirect impacts from the construction of the risk reduction levee system is 1,188.03 AAHUs.

Localized Storm Surge Risk Reduction Measures

The Recommended Plan includes localized storm surge risk reduction measures for structures in the communities of Gramercy, Lutcher and Grand Point, which are located outside of the proposed levee system (Figure 5-2). These localized storm surge risk reduction measures focused on addressing existing damages in St. James Parish, while still being economically justified and environmentally compliant. See Chapter 3.9 and Appendix E for information concerning plan formulation and design of the localized storm surge risk reduction measures. These measures include berms and flapgates on existing drainage and roadway features. Floodproofing measures (e.g., raising of certain residential structures and construction of smaller berms around certain individual non-residential structures) are limited to a few structures located outside of the larger localized storm surge risk reduction measures. All of the measures focus on providing a risk reduction above the 1% probability storm stages in Year 2020. The Non-Federal Sponsor (NFS) will be required to maintain these features to their initially-constructed design height for as long as the project remains authorized. The future level of risk reduction is dependent on the actual rate of RSLR.

Gramercy Area

In the Gramercy and Lutcher area, north of LA Hwy. 3125, a 10,100 LF berm would be built to provide risk reduction to 275 structures, herein referred to as "**Polder 1 (Gramercy Berm)**." The berm would be constructed to a 6.5' NAVD88 elevation. The berm in Year 2020 would provide risk reduction above the 1% probability storm stages. Storm stages in St. James Parish are below +6.5' NAVD88 elevation in Year 2020. As discussed in Chapter 3, in the future, the berm's effectiveness depends on the actual rate of RSLR.

The berm would parallel both sides of LA Hwy. 20, and parallel the railroad track along US-61 (Airline Highway). To the south, the berm would tie into LA Hwy. 3125 to close off the system. LA Hwy. 3125 is key feature for all of the localized storm surge risk reduction features. The entire roadway is above 6.5' NAVD88 elevation and will be used as a tie-in point for the berm. The design of the berm is based on a 4' wide crown and 3:1 side slopes. Using local Light Detection and Ranging (LiDAR) data, it was assumed that the existing ground elevation under the berm would be at an elevation of approximately 4.3' NAVD88. Using this assumption, the proposed berm would have an average height of 2.2' with an average width of 18', and require 237,000 cy of compacted fill for construction. The berm would also include two floodgates to allow existing drainage to flow through the berm when not under surge events. A pump system to operate and remove rainwater during tropical / hurricane storm events will be included in the features. The pump system will be approximately 217 cfs. The berm would be placed in a location so as not to interfere with existing local drainage.

In reviewing the berm footprint, there is a risk of affecting approximately 0.29 acres of forested wetlands. Attempts would be made to avoid these areas during construction. Due to the current uncertainty in avoiding these areas, we have included costs for mitigating for these forested wetlands in the total construction cost.

Grand Point Area

In the Grand Point area, north of LA Hwy. 3125, the Recommended Plan includes one berm, "**Polder3 (Grand Point North**)". Polder3 (Grand Point North) would provide risk reduction to 71 structures. The berm would be a complete ring around the structures in the northern portion of Grand Point, near the Grand Point Boat Launch. The berm would be 10,400 LF, and would include a 4' wide crown and 3:1 side slopes. The berm would be constructed to a 6.5' NAVD88 elevation. Initially, in Year 2020, the berm would provide risk reduction above the 1% probability storm stages. Storm stages in St. James Parish are below a 6.5' NAVD88 elevation in Year 2020. Future level of risk reduction is dependent on the actual rate of RSLR.

Using local LiDAR data, it was assumed that the existing ground elevation under the berm would be approximately 4' NAVD88. Using this assumption, the proposed berm would have an average height of 2.5' with an average width of 20', and require 286,800 cy of compacted fill for construction. The berm would also include one floodgate to allow existing drainage to flow through the berm when not under surge events. A pump system to operate and remove rain water during tropical / hurricane storm events will be included in the features. The pump system will be approximately 140 cfs. The berm would be placed in a location so as not to interfere with existing local drainage. The berm would also be placed very near the edge of the property owners' parcels where feasible. This would minimize the loss of use of any property.

In reviewing the berm footprint, there is a risk of affecting approximately 0.81 acres of forested wetlands. Attempts would be made to avoid these areas during construction. Due to the current uncertainty in avoiding these areas, we have included costs for mitigating for these forested wetlands in the total construction cost.

Flood Risk Reduction Under LA Highway 3125

In addition to the berms north of LA Hwy. 3125, the Recommended Plan is to use 13 miles of LA Hwy. 3125 and its existing foundation as a localized storm surge risk reduction feature. Currently, the roadway elevation is above 6.5' NAVD88 in elevation. At present, the 1% probability storm stages in Year 2020 flow through the culverts under the roadway in the opposite direction from natural drainage. By closing off the culverts with one-way flap gates and a drainage canal with a floodgate during surge events, the plan would provide risk reduction to 19,500 acres and 4,295 structures south of LA Hwy. 3125. Although there are a limited number of structures that are impacted by the 1% probability storm stages, this closure reduces the risk of a large portion of the Parish's critical sugarcane crops from flooding from this type of storm surge event. If the Parish in the future makes improvements to LA Hwy. 3125, any additional height added to the entire highway could add to the structures risk reduction level behind the highway. Due to the fact that the roadway is being used as a flood risk reduction feature, the local sponsor will be required to maintain the system's initial level of risk reduction. This includes the berm tie-in points to the roadway and 13 miles of the roadway itself. If the roadway requires maintenance and would be degraded below its original elevation, the work should take place outside of hurricane season. If it is not possible to work outside of hurricane season, interim flood risk measures should be implemented to maintain the original level of risk reduction provided by the roadway.

The Recommended Plan includes 145 flap gate closures, two floodgates and two small berms (Noranda and Uncle Sam). The Noranda berm ties the highway into high ground east of Gramercy. The Uncle Sam berm divides the developed area behind LA Hwy. 3125 from an area that is primarily agricultural land. By dividing these two areas, the local community can focus its reduction efforts in the future. Future improvements could be focused on sections of the highway that have structures behind the highway, approximately 7 miles vs. 13 miles. The area west of the Uncle Sam berm includes an area of 8,175 acres, but only includes one structure that has a first floor elevation below the 1% probability storm stages. The total length of the berms is approximately 645 LF.

Due to the nature of the flooding south of LA Hwy. 3125, it is assumed that the 19,500 acres would have ample storage capacity to hold any rainfall during the surge events. Even if some acres of crops are flooded from rainfall, it would be much less severe than if storm surge was allowed to flow under LA Hwy. 3125.

Remaining Structures in St. James Parish

Eighty structures were evaluated outside of the economically-justified and unjustified berms. Only 23 of the 80 structures have a first floor elevation below the 1% probability storm stages in Year 2020. Based on this evaluation, the Recommended Plan includes 14 residential structures that would be raised to the stage associated with the Year 2070 intermediate RSLR 1% probability storm stages; 4 non-residential structures would be floodproofed to 3 feet above the ground elevation; and smaller berms would be constructed for 5 light industrial/warehouse facilities. The 14 residential structures are being raised to the Year 2070 height because it is more cost effective to raise a home once.

The incremental first cost for the levee system in the Recommended Plan is $676,598,000. The incremental first cost for the localized storm surge risk reduction system in the Recommended Plan is $41,493,000. The total first cost for the Recommended Plan is $718,091,000.

Hydraulics and Hydrology

Interior Modeling Methodology (Without-Project and With-Project - Alignment C)

Hydrology

General. The hydrologic model was developed utilizing HEC-HMS 3.5. Rainfall runoff hydrographs were generated throughout the system for synthetic rain events. Synthetic flood events of a magnitude that are expected to be equaled or exceeded once on the average during any 1-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year (recurrence interval) have been selected as having special significance for floodplain management. As an example, the 10-year and 100-year floods have a 10 percent and 1 percent chance of occurring and of being equaled or exceeded during any year, respectively. Rainfall totals for these frequency events were derived from the NOAA Atlas 14 Precipitation Frequency Data Server (PFDS).

Drainage Basin Area Delineation. The drainage basin areas were directly taken from the ADvanced CIRCulation (ADCIRC) model. ADCIRC is a system of computer programs for solving time-dependent, free-surface circulation and transport problems in two and three dimensions. These programs utilize the finite element method in space allowing the use of highly flexible, unstructured grids. In areas where newer models were available, the ADCIRC basins were supplemented with newer and more relevant basin delineation. Those particular areas were in St. John the Baptist Parish. The basin areas from that model were overlaid and merged with the original ADCIRC basins.

Basin Parameter Determination. For each of the drainage areas delineated within the watersheds, estimates were made of the homogeneous surface characteristics and soil properties needed to characterize the runoff potential. These data define the individual characteristics of each of the drainage areas as direct input parameters for the hydrologic model.

Each sub-basin (storage area) requires an entry of seven pieces of data, or hydrologic parameters, in order to compute a hydrograph: the name of the sub-basin; the sub-basin size, the initial loss rate, the percent of the sub-basin that is impervious; the Soil Conservation Service (SCS) curve number (CN); the lag time and the base flow in cubic feet per second.. The model is made up of several sub-basin elements, each containing these seven pieces of data, and an outflow channel system that can carry the runoff downstream where it may combine with other runoff to generate a flood wave in the watercourse.

Soil Type and Land Use. The SCS curve number is related to soil type, land use and antecedent moisture conditions. More information about the background and use in the SCS curve number method can be found at USDA Natural Resource Conservation Service (NRCS). The curve number is a non-dimensional value that ranges between 1 and 100 that the SCS method uses to represent the potential for surface runoff from a watershed. Higher CN values indicate higher potential runoff, corresponding with a lower amount of rainfall "losses". The major factors that determine CN are the hydrologic soil group, cover type, treatment, hydrologic condition and antecedent runoff condition.

The curve numbers listed in Appendix A of the HEC-HMS Technical Reference Manual for every land use type are sub-classified into different hydrologic soil groups. SCS soil maps were downloaded from the web site http://soildatamart.nrcs.usda.gov and clipped with the watershed borders in GIS. Two different hydrologic soil types are found in the modeled watersheds (C &

D).

The digitized land use shape file was intersected with the soil shape file using ArcGIS, resulting in 22 different CN surface types: 11 land use types, each with two different hydrologic soil types. In the attribute table, the area of every sub-element was calculated using the area formula in the field calculator. These areas were then tabulated in ArcGIS, using the watershed sub-basin name as row theme and the surface types as columns. The resulting table was exported to Microsoft Excel, where the corresponding CNs were assigned to every soil-land use category.

Initial Losses. Initial losses were computed using the SCS loss rate method. In this method, 20% of the maximum retention is taken to be the initial abstraction or "initial loss in inches". Runoff losses for the model were determined by the SCS CN method. The equation is as follows:

$$I_a = 0.2 * (\frac{1000}{CN} - 10)$$

Lag Time Calculations. Some sub-basins are extremely low-lying, offering little change in slope and have large areas available for the storage of water. Modeling these areas utilizing mostly traditional hydrologic engineering methods could be inaccurate based on the fact that most methods do not compensate for such small slopes and such large areas available for storage. Sub-basins were modeled in HEC-HMS utilizing the SCS unit hydrograph procedure. The SCS method can be used for urban areas that are less than 2,000 acres or 3.1 sq. mi. Lag Time calculations were computed for each sub-basin using the SCS lag time equation which includes the slope of the sub-basin, the length of travel and the SCS curve number. The Lag Time calculation equation used is as follows:

$$T_L = L^{0.8} * \frac{(S+1)^{0.7}}{(1900 * y^{0.5})}$$

Where:

T_L= Sub-basin Lag Time (hr)

L = Hydraulic Length (ft)

S = (1000/CN) – 10

CN = Sub-basin Average Curve Number

y = Average Sub-basin Land Slope (%)

Flow path lengths were measured from the farthest point in the sub-basin to the lowest point in the sub-basin. These paths were determined by visual inspection of the Light Detection and Ranging (LiDAR) imagery. Using ArcGIS Spatial Analyst Slope feature, a slope grid was produced from the LiDAR imagery. The grid is the slope of a particular pixel in relation to its eight neighboring pixels. The average sub-basin land slope of each sub-basin was calculated using Zonal Statistics from Spatial Analyst Tools. After calculating the necessary input data, the data was entered into the SCS Lag Time equation for each sub-basin. Next, the Lag Time was then entered into the appropriate sub-basin in HEC-HMS. HEC-HMS was run and rainfall runoff hydrographs were computed for each sub-basin. Then, the rainfall runoff hydrographs were entered as input to the HEC-RAS hydraulic model.

Table 1 lists the parameters described above used in the existing conditions hydrologic model.

Storage	Lag (min)	Area (Sq Mi)	CN	Impervious (%)	Initial (in)
SA 1	79.07	0.739	81.78	13.81	0.446
SA 10	109.61	1.230	82.43	15.76	0.426
SA 11	55.47	0.401	89.44	78.92	0.236
SA 12	108.97	1.669	80.59	18.49	0.482
SA 13	70.18	0.696	82.17	12.72	0.434
SA 14	84.86	1.181	83.36	4.45	0.399
SA 15	104.20	0.756	80.88	30.63	0.473
SA 16	78.88	0.562	77.05	9.10	0.596
SA 17	74.48	1.017	75.81	1.07	0.638
SA 18	56.13	0.150	77.35	6.77	0.586
SA 19	85.94	0.532	81.08	8.55	0.467
SA 2	124.66	1.519	80.66	18.30	0.479
SA 20	63.46	0.183	79.50	6.77	0.516
SA 21	138.33	1.122	81.81	26.05	0.445
SA 22	50.64	0.487	81.42	6.93	0.456
SA 23	57.82	0.480	81.56	32.69	0.452
SA 24	63.83	0.504	78.16	25.96	0.559
SA 25	36.27	0.243	76.94	6.04	0.599
SA 26	69.25	1.190	80.42	3.98	0.487
SA 27	77.24	0.929	81.06	5.78	0.467
SA 28X	37.89	0.461	82.16	7.28	0.434
SA 28Y	55.94	0.309	81.72	3.95	0.447
SA 29	86.07	0.921	81.52	12.92	0.453
SA 29C	120.99	0.571	82.83	3.64	0.415
SA 3	37.62	0.503	84.63	42.58	0.363
SA 30	63.53	0.701	80.98	35.08	0.470
SA 30C	61.66	0.297	81.42	3.83	0.457
SA 31	93.81	1.823	81.02	29.86	0.469
SA 31C	35.27	0.153	82.23	7.87	0.432
SA 32	116.31	1.342	78.94	31.48	0.534
SA 33	111.25	0.801	78.80	32.40	0.538
SA 34	62.32	0.839	79.41	34.82	0.519
SA 35	87.93	0.460	78.59	27.91	0.545
SA 36	89.92	0.690	77.57	20.23	0.578
SA 37	150.33	1.346	78.77	2.88	0.539
SA 38	59.33	0.404	78.63	30.99	0.544
SA 39	79.40	0.595	77.26	11.42	0.589
SA 39C	97.01	0.949	76.04	0.66	0.630
SA 4	64.38	0.859	81.94	28.75	0.441
SA 40P	277.53	8.502	81.94	0.74	0.441
SA 41	119.15	0.347	76.54	17.51	0.613
SA 41P	203.21	5.770	82.01	0.89	0.439
SA 42P	163.17	2.514	77.71	2.60	0.574
SA 43P	120.44	2.503	76.09	0.69	0.629
SA 44C	25.66	0.156	79.64	25.37	0.511

SA 5	78.19	1.105	83.53	22.72	0.394
SA 6	59.25	0.304	81.91	28.90	0.442
SA 7	48.51	0.508	78.86	21.18	0.536
SA 8	138.25	2.073	77.05	13.32	0.596
SA 9	65.84	0.785	79.61	4.22	0.512

Table 1: HEC-HMS parameters for Existing Conditions

Table 2 lists the parameters described above used in the with-project hydrologic model.

Storage Area	Lag Time (min)	Area (sq mi)	CN	Impervious (%)	Initial Abstraction (in)
SA 1	80.51	0.612	81.58	15.63	0.452
SA 10	109.56	1.230	82.47	15.04	0.425
SA 100	12.72	0.096	82.78	2.38	0.416
SA 101	59.64	0.541	76.77	4.76	0.605
SA 102	32.95	0.083	81.98	7.14	0.440
SA 11	55.64	0.401	89.44	79.80	0.236
SA 12	108.66	1.669	80.60	18.60	0.481
SA 13	70.24	0.696	82.20	13.23	0.433
SA 14	84.86	1.181	83.35	4.21	0.400
SA 15	104.33	0.756	80.87	30.76	0.473
SA 16	78.82	0.562	77.05	9.22	0.596
SA 17	74.44	1.017	75.80	1.06	0.639
SA 18	17.48	0.049	80.83	4.71	0.474
SA 19	85.78	0.532	81.08	8.41	0.467
SA 2	125.22	1.519	80.67	17.79	0.479
SA 20	62.99	0.183	79.50	7.52	0.516
SA 21	138.37	1.122	81.82	26.17	0.444
SA 22	50.63	0.487	81.42	6.83	0.456

SA 23	57.73	0.480	81.59	32.44	0.451
SA 24	63.24	0.504	78.14	24.13	0.560
SA 25	36.56	0.243	76.86	5.20	0.602
SA 26	69.54	1.190	80.42	3.70	0.487
SA 27	77.30	0.929	81.08	5.67	0.467
SA 28X	37.73	0.461	82.18	6.99	0.434
SA 28Y	56.51	0.309	81.72	4.08	0.447
SA 29	85.78	0.921	81.50	12.65	0.454
SA 29C	120.72	0.571	82.84	3.91	0.414
SA 3	37.22	0.503	84.62	42.44	0.363
SA 30	63.38	0.701	80.99	33.76	0.469
SA 30C	62.05	0.297	81.38	3.42	0.458
SA 31	94.23	1.823	81.00	30.00	0.469
SA 31C	35.32	0.153	82.23	5.67	0.432
SA 32	116.40	1.342	78.94	31.52	0.534
SA 33	110.80	0.801	78.78	33.27	0.539
SA 34	62.92	0.839	79.46	33.69	0.517
SA 35	88.29	0.460	78.61	28.45	0.544
SA 36	90.34	0.690	77.58	21.87	0.578
SA 37	150.02	1.346	78.78	2.95	0.539
SA 38	59.37	0.404	78.66	30.82	0.543
SA 39	79.16	0.595	77.30	12.89	0.587
SA 39C	97.03	0.949	76.04	0.59	0.630
SA 4	64.38	0.859	81.96	29.11	0.440
SA 40P	103.36	2.212	81.59	0.37	0.451
SA 41	119.20	0.347	76.54	17.47	0.613

SA 41P	155.34	4.550	81.99	0.48	0.439
SA 42P	161.39	2.397	77.79	3.00	0.571
SA 43P	118.85	2.362	75.99	0.77	0.632
SA 44C	25.66	0.156	79.65	26.03	0.511
SA 5	78.40	1.105	83.53	22.66	0.394
SA 6	59.78	0.304	81.97	28.63	0.440
SA 7	48.60	0.508	78.90	21.46	0.535
SA 8	138.66	2.073	77.06	13.76	0.595
SA 9	37.06	0.310	80.77	6.86	0.476

Table 2: HEC-HMS parameter for With-Project (Alignment C)

Reach Parameter Calculation. The model is tied together by a series of routing reaches and junctions where several flow paths join into one channel as the flood wave moves downstream. A reach represents a portion of the natural channel that carries the flood. The velocity of the water moving through the reach and the amount of channel storage available to the water determines the rate or speed of translation of the flood wave. The more storage that is available, the less speed of translation and the longer duration of flood effects that are observed. The model parameters can be selected to account for channel and overbank storage using several routing techniques that are options in the software. As the base flow is negligible in modeling large events, no base flow method was used.

Rainfall. Frequency-based synthetic rainfall (Table 3) was used for each sub-basin in the model. The rainfalls were taken from NOAA Atlas 14 Precipitation Frequency Data Server (PFDS). A 24-hour storm duration (total rain time) was chosen based on time of concentration and to remain consistent with other studies conducted by the U.S. Army Corps of Engineers.

Duration	2 Yr	5 Yr	10 Yr	25 Yr	50 Yr	100 Yr	200 Yr	500 Yr
5 minutes	0.59	0.73	0.84	1.00	1.13	1.26	1.40	1.58
15 minutes	1.06	1.3	1.51	1.79	2.02	2.25	2.50	2.82
1 hour	2.14	2.68	3.16	3.84	4.40	4.98	5.59	6.43
2 hours	2.68	3.39	4.01	4.93	5.70	6.50	7.36	8.56
3 hours	3.03	3.84	4.58	5.70	6.64	7.65	8.75	10.3
6 hours	3.65	4.67	5.61	7.05	8.27	9.60	11.05	13.12

12 hours	4.34	5.57	6.69	8.39	9.83	11.38	13.06	15.46
24 hours	5.07	6.47	7.74	9.65	11.25	12.96	14.82	17.45

Table 3: Frequency-Based Synthetic Rainfall Distributions for St. John the Baptist Parish

Since HEC-HMS only has probability for the 50 percent to the 0.2 percent rainfall, the SCS storm (NRCS) total rainfall depth was used for the 1-year rainfall event. The total depth for the 1-year rainfall according to NOAA Atlas 14 Precipitation Frequency Data Server is 4.50 inches. The time distribution selected for this area was Type 1.

Hydraulics

Geometry

Topographic Data Used. The 5x5 meter LiDAR field data (downloaded from http://atlas.lsu.edu/LiDAR) was used to define topographic features because it gave a better resolution than the 30x30 DEM (Digital Elevation Model) method. The field point data was categorized into the following sets: a set of raw points, a set of edited points and a contour line shapefile. The metadata file, which was also included, describes the projection of the data points and their level of accuracy.

The contour line shapefile, consisted of vector lines with elevation data at two-foot intervals. This contour vector data was then used in a GIS (Geographic Information System) program to display any desired projection.

Datum. The Datum used for the modeling project is NAVD88 (Epoch 2004.65). This Datum was used throughout the development of the model and all stages and elevations reported in this document are to this datum. No conversions of data due to datum discrepancies were required in the model.

Once the HEC-HMS hydrological model was completed, the runoff hydrographs were placed as input into the HEC-RAS hydraulic model. The hydraulic model consists of canals, storage areas and structures (such as bridges, pumping stations, inline weirs and lateral weirs).

Canal Alignments and Connections. The basic alignment of canals, storage areas and connections was taken from the USACE ADCIRC West Shore-Lake Pontchartrain model. It was then modified by adding lateral weirs representing areas conducive to bank overflow into the various parts (Storage Areas) of the model. LiDAR imagery was used to establish top-of-bank elevations for lateral weirs.

Canal Cross Sections. The detailed area of St. John the Baptist Parish was taken from the HEC-RAS model developed by Burk-Kleinpeter, Inc. (BKI). The dimensions of other canals were determined by conducting a reconnaissance-level survey of the most important canals and their related crossings.

The Manning's "N" values were taken from the HEC-RAS Technical Reference Manual for typical canal sections with earthen, concrete and rip-rap bottoms.

Pump Stations. Existing pumping stations were modeled in HEC-RAS using the existing pump curves, as described below. Proposed pumping station pump curves were taken from pump curves of commonly-used centrifugal pumps of appropriate size.

Calibration. Calibration is a process whereby the model is adjusted to better simulate the actual drainage system and a storm event with recorded data. This is usually accomplished by analyzing the performance of the model when an historical rainfall is provided as input. For this model, no historical storm events with recorded data were available. A common method of calibration / validation (when no such data is available) is simulating a 10% recurrence storm and plotting its inundation over the study area. Once that is done, the inundation map is given to the drainage department of the area of study and comments are provided on the extent of the mapping.

For this project, an inundation map of the 10% recurrence simulation was provided to the St. John the Baptist Parish Drainage Department. Comments were provided on several areas that didn't seem to match historical inundation for the 10% storm. Even though a detailed channel network was not available for all areas of the project area, parameters were adjusted so the new inundation more closely matched the historical inundation.

With-Project Model. The levee alignment (Alignment C) was overlaid on the existing conditions sub-basin (storage area) map to determine which sub-basins (storage areas) would be affected by the alignment. The affected sub-basins (storage areas) were then edited to reflect the reduced elevation-volume and gross area. New parameters for the HEC-HMS models were calculated (Table 4). New elevation-volume curves were also calculated and modified in HEC-RAS.

Lateral Structures (weirs) were placed in the model to simulate overflow from canals to and from storage areas.

Gates and pumps were added to the with-project HEC-RAS model. The gates are to promote normal tidal exchange and allow rainwater to move out of the system during normal or low tide conditions. During elevated Lake conditions attributable to hurricane and tropical storm events when the elevation of the lake reaches approximately +1.7 ft. NAVD88, the gates would close and the pumps would evacuate the rain water as it moves through the system. This is expected to occur 8.5 days per year.

The gravity drainage gates and pumps would be placed in the new levee alignment at the following canals:

1. Hope Canal
2. Reserve Relief Canal
3. Ridgefield Canal
4. Montz Canal / Woodland Canal

The gravity drainage gates would be placed in the new levee alignment at the following canals:

1. Mississippi Bayou
2. Perriloux Canal

The storage areas for the proposed model were developed in the same way as those in the existing model. Additional storage areas were created along Alignment C. The levee alignment bisects some storage areas and produces the need to add some new storage areas.

RESULTS AND EVALUATION

As stated in Chapter 5 of the Main Report, hydrologic connectivity would be maintained to the extent practicable through water control structures except during closure for hurricanes or tropical storms. When the system is closed, pumps would operate on average for 1.7 storms per year, which equates to a closure of structures on average 8.5 days per year. This expected rate of closure would be the same regardless of the actual rate of SLR as closure of the system is tied to tropical storm events and the elevation trigger would be adjusted as sea level rises. The risk reduction system is only authorized to address storm surge caused by hurricane and tropical storm events. It is not authorized to mitigate for or reduce impacts caused by higher day-to-day water levels brought about by increases in sea level rise. Any operational changes implemented to address changing SLR conditions or for any other non-project-related purpose would be considered a separate project purpose requiring separate authorization, new NEPA documentation, and/or permit approvals.

An assessment was conducted to analyze the water levels in the surrounding lakes. A hydraulic analysis was performed using HEC-RAS and synthetic frequency rainfall. An initial condition run was established and simulated. The objective of the initial condition simulation is to establish the interior stages to an elevation equal to the actual elevations after high lake elevations for 5 to 7 days. The elevations related to the last profile in the initial conditions simulation are used to begin the synthetic frequency rainfall simulations. This method ensures the model starts with the same interior basin stages that would occur before the gates are closed.

In this section of the Engineering Appendix, the 10% recurrence interval rainfall event (10-Year) for the existing condition (year 2013) and future development condition (Year 2070) are compared. Note - no future land development was considered for the future development condition in the hydrology simulation (HEC-HMS). The only difference is the addition of relative sea level rise (SLR) at the downstream boundaries of the model. The SLR values were added directly to the original downstream boundary.

Table 4 below is the comparison of storage area stages for without-project (Year 2013) and the with-project (Year 2013).

	Without-Project				**With-Project**			
SA	**RT1yr**	**RT5yr**	**RT10yr**	**RT25yr**	**RT1yr**	**RT5yr**	**RT10yr**	**RT25yr**
SA18	2.05	2.20	2.31	2.44	2.05	2.20	2.31	2.44
SA9	3.23	4.24	4.45	4.73	3.23	4.24	4.45	4.73
SA1	5.09	5.81	6.14	6.49	5.09	5.81	6.11	6.52
SA2	6.00	6.83	7.23	7.69	6.02	6.85	7.25	7.70
SA3	13.57	14.00	14.27	14.64	13.58	14.01	14.28	14.65
SA4	11.18	11.67	11.98	12.41	11.19	11.68	12.00	12.42
SA5	11.28	12.00	12.36	12.54	11.31	12.03	12.39	12.56
SA6	12.06	12.63	12.97	13.28	12.06	12.63	12.99	13.28
SA7	4.95	5.87	6.36	6.94	4.97	5.89	6.38	6.96
SA8	6.28	7.26	7.67	7.93	6.30	7.28	7.69	7.94
SA10	3.59	3.81	3.92	4.07	3.62	3.83	3.98	4.12
SA11	6.75	6.96	7.11	7.58	6.76	6.97	7.13	7.60
SA12	5.78	6.72	7.11	7.58	5.81	6.75	7.13	7.60

SA13	4.86	5.31	5.53	5.83	4.88	5.33	5.55	5.85
SA14	8.09	8.88	9.11	9.29	8.12	8.91	9.12	9.30
SA15	8.83	9.77	10.05	10.28	8.85	9.79	10.07	10.29
SA16	4.95	5.81	6.24	6.66	4.96	5.83	6.25	6.68
SA17	3.20	4.36	4.94	5.71	3.23	4.38	4.96	5.73
SA25	3.11	4.29	4.88	5.61	3.15	4.30	4.89	5.63
SA22	4.83	5.73	5.99	6.15	4.85	5.75	6.00	6.15
SA21	3.27	3.87	4.36	4.83	3.33	3.89	4.39	4.84
SA19	2.07	2.28	2.42	2.50	2.07	2.28	2.42	2.50
SA20	3.20	3.49	3.62	3.74	3.22	3.51	3.63	3.86
SA43P	2.08	2.38	2.56	2.86	2.08	2.38	2.47	2.81
SA42P	1.86	2.03	2.12	2.30	1.71	1.84	1.92	2.38
SA26P	1.86	2.03	2.12	2.31	1.74	1.87	1.94	2.41
SA28Y	1.86	2.03	2.12	2.31	1.73	1.86	1.94	2.40
SA29C	1.86	2.03	2.12	2.31	1.73	1.86	1.94	2.40
SA30	1.86	2.03	2.12	2.31	1.74	1.87	1.94	2.40
SA44C	1.76	1.89	1.96	2.14	1.74	1.84	1.92	2.38
SA41P	1.86	2.03	2.12	2.30	1.73	1.86	1.94	2.40
SA40P	1.86	2.03	2.12	2.30	1.73	1.86	1.94	2.40
SA31	2.29	2.75	2.98	3.28	2.32	2.73	2.94	3.23
SA32	2.22	2.70	2.94	3.29	2.27	2.66	2.90	3.23
SA41	2.19	2.67	2.90	3.20	2.26	2.63	2.86	3.17
SA35	2.92	3.24	3.36	3.54	2.92	3.24	3.37	3.55
SA38	2.80	2.89	2.93	2.98	2.81	2.90	2.94	2.99
SA37	2.49	2.68	2.77	2.86	2.50	2.69	2.77	2.87
SA36	1.82	2.23	2.62	3.11	1.76	2.18	2.56	2.98
SA27	1.86	2.03	2.12	2.31	1.74	1.87	1.94	2.41
SA30C	1.86	2.03	2.12	2.31	1.74	1.87	1.94	2.40
SA23	5.95	6.72	6.99	7.40	5.98	6.74	7.02	7.42
SA24	4.58	5.77	6.22	6.64	4.59	5.79	6.23	6.65
SA28X	1.86	2.03	2.12	2.31	1.74	1.87	1.94	2.40
SA39	2.08	2.39	2.56	2.86	2.10	2.39	2.47	2.81
SA39C	2.08	2.39	2.56	2.86	2.08	2.38	2.47	2.81
SA34	3.03	3.67	3.96	4.26	3.02	3.67	3.97	4.27
SA33	3.72	4.04	4.17	4.31	3.73	4.05	4.18	4.32
SA31C	2.28	2.73	2.94	3.20	2.31	2.70	2.90	3.14
SA29Y	3.20	3.79	3.91	4.34	3.22	3.81	4.25	4.68

Table 4: Year 2013 Comparison of Stages: With-Project vs. Without-Project

Table 5 below is the comparison of storage area stages for Without-Project (Year 2070 Intermediate SLR) and the With-Project (Year 2070 Intermediate SLR).

	Without-Project				**With-Project**			
SA	**RT1yr**	**RT5yr**	**RT10yr**	**RT25yr**	**RT1yr**	**RT5yr**	**RT10yr**	**RT25yr**
SA18	2.07	2.86	2.86	3.04	2.07	2.86	2.86	3.04

SA9	3.24	4.30	4.50	4.77	3.24	4.30	4.50	4.77
SA1	5.09	5.81	6.14	6.49	5.23	5.81	6.11	6.52
SA2	6.00	6.83	7.23	7.69	6.00	6.85	7.25	7.70
SA3	13.57	14.00	14.27	14.64	13.57	14.01	14.28	14.65
SA4	11.18	11.67	11.98	12.41	11.18	11.68	12.00	12.42
SA5	11.28	12.00	12.36	12.54	11.28	12.03	12.39	12.56
SA6	12.06	12.63	12.98	13.28	12.06	12.63	12.98	13.28
SA7	4.96	5.89	6.37	6.94	4.95	5.89	6.38	6.96
SA8	6.28	7.26	7.67	7.93	6.28	7.28	7.69	7.94
SA10	3.65	4.13	4.16	4.21	3.57	3.83	3.98	4.12
SA11	6.75	6.96	7.12	7.58	6.76	6.97	7.13	7.60
SA12	5.78	6.72	7.12	7.58	5.78	6.75	7.13	7.60
SA13	4.86	5.31	5.53	5.83	4.86	5.33	5.55	5.85
SA14	8.09	8.88	9.11	9.29	8.09	8.91	9.12	9.30
SA15	8.84	9.78	10.05	10.28	8.83	9.79	10.07	10.29
SA16	4.95	5.83	6.25	6.67	4.95	5.83	6.25	6.68
SA17	3.25	4.39	4.96	5.72	3.29	4.39	4.97	5.73
SA25	3.16	4.32	4.90	5.62	3.19	4.32	4.91	5.64
SA22	4.84	5.78	6.02	6.18	4.83	5.75	6.00	6.15
SA21	3.80	4.69	4.93	5.30	3.23	3.89	4.39	4.84
SA19	3.46	4.08	4.10	4.11	1.88	2.81	3.28	3.84
SA20	3.45	4.08	4.10	4.12	3.20	3.51	3.63	3.84
SA43P	2.19	2.51	2.68	2.94	2.22	2.50	2.67	2.92
SA42P	3.44	3.95	3.99	4.02	0.87	1.27	1.51	1.76
SA26P	3.44	3.96	4.00	4.02	1.31	1.66	1.76	1.89
SA28Y	3.44	3.96	4.00	4.03	1.12	1.42	1.57	1.83
SA29C	3.45	3.96	4.01	4.03	1.50	1.70	1.78	1.88
SA30	3.44	3.96	4.00	4.03	1.47	1.74	1.85	2.00
SA44C	2.52	3.11	3.18	3.29	0.87	1.27	1.51	1.76
SA41P	3.43	3.95	4.00	4.02	0.87	1.29	1.55	1.83
SA40P	3.44	3.96	4.00	4.02	0.87	1.29	1.55	1.83
SA31	3.44	3.96	4.01	4.15	1.89	2.51	2.80	3.10
SA32	3.05	3.50	3.66	3.84	2.01	2.57	2.84	3.18
SA41	3.02	3.43	3.55	3.71	1.72	2.53	2.80	3.11
SA35	2.93	3.31	3.46	3.68	2.92	3.24	3.37	3.55
SA38	2.80	2.89	2.93	2.98	2.80	2.90	2.94	2.99
SA37	2.49	2.68	2.77	2.94	2.49	2.69	2.77	2.92
SA36	2.62	3.28	3.44	3.65	1.80	2.08	2.51	2.95
SA27	3.44	3.96	4.00	4.02	1.18	1.44	1.55	1.83
SA30C	3.45	3.95	4.01	4.03	1.46	1.73	1.84	1.98
SA23	5.98	6.75	7.03	7.43	6.00	6.75	7.03	7.43
SA24	4.72	5.79	6.23	6.65	4.55	5.78	6.23	6.65
SA28X	3.44	3.96	4.00	4.02	0.80	1.38	1.55	1.83
SA39	2.20	2.52	2.69	2.94	2.22	2.51	2.67	2.93
SA39C	2.20	2.52	2.68	2.94	2.22	2.50	2.67	2.92

SA34	3.05	4.02	4.20	4.37	3.02	3.67	3.97	4.27
SA33	3.72	4.11	4.24	4.38	3.72	4.05	4.18	4.32
SA31C	3.44	3.96	4.01	4.05	1.89	2.50	2.78	3.05
SA29Y	3.32	4.26	4.52	4.82	2.75	3.81	4.25	4.68

Table 5: Year 2070 Intermediate SLR Comparison of stages: With-Project vs. Without-Project

By carefully reviewing the results in the tables, the largest reduction occurs when SLR is incorporated into the analysis. Without the levee for risk reduction during hurricane and tropical storm events, elevated lake levels infiltrate the unprotected area of St. Charles and St. John the Baptist Parishes and cause flooding.

Because of the lack of stream detail in the model, the areas away from the new pumping stations are unable to drain effectively to the stations and the analysis shows no elevated stages. A more detailed analysis that includes new channel sections and additional channel geometry from surveys, would likely show reduced stages for the with-project condition.

Exterior Storm Surge Modeling

This portion of the report documents some of the post-processing steps that were performed to determine stage-frequency and associated wave conditions from raw ADCIRC data. A brief summary of the different ADCIRC meshes used in the analysis is described first. Then, some of the surge results are examined in order to explain how the stage-frequency and associated wave values are determined from raw ADCIRC output. It should be noted that an ADCIRC modeling report was completed 22 April 2011. The ADCIRC model used was subjected to an Agency Technical Review (ATR), which was completed 28 September 2011.

Table 6 contains a summary of the ADCIRC simulations performed for the analysis. A total of 152 storms were included in the analysis for the 2011 base condition, on a version of the SL15 mesh that includes HSDRRS features such as the IHNC barrier and the Seabrook closure. After Hurricane Katrina, it was decided (in 2006) to pursue a common technical framework for use by all Federal Agencies that are involved with assessing hurricane-related threats to coastal communities; this includes storm selection and statistical performance. A detailed explanation of the selection of hypothetical storms, probabilities and statistical performance is in a document entitled “White Paper on Estimating Hurricane Inundation Probabilities for Storm Selection and Statistics Reference” (dated 10 June 2007). The mesh also includes added resolution in the project area. Figure 2 displays a version of SL15 which does not include the added resolution for the project area. Figure 3 displays the mesh with added resolution. The areas that appear black are areas that include high resolution. Future condition meshes were created for the “No Action” or “Without-Project Condition” and the “With-Project Condition”. The Year 2020 meshes include a modest SLR value of 0.3ft. In the Year 2020 mesh, the nodal attributes including bottom friction and canopy cover are not modified to reflect land loss that occurs with SLR. For Year 2020, it is assumed that the landscape will not change drastically enough to warrant modifying bottom friction or canopy coefficients. For the Year 2070 meshes, the nodal attributes are modified to reflect a future condition that includes loss of bottom friction and canopy. The Recommended Plan is Alignment C.

Table 6 Summary of ADCIRC Simulations

NO ACTION ADCIRC RUNS	SLR (ft)

No Action Base Condition 2011	0.00
No Action Future Condition 2020 Intermediate SLR	0.30
No Action Future Condition 2070 Low SLR	1.80
No Action Future Condition 2070 Intermediate SLR	2.30
No Action Future Condition 2070 High SLR	3.00
WITH-PROJECT ADCIRC RUNS	SLR (ft)
Alignment C Future Condition 2020 Intermediate SLR	0.30
Alignment C Future Condition 2070 Low SLR	1.80
Alignment C Future Condition 2070 Intermediate SLR	2.30
Alignment C Future Condition 2070 High SLR	3.00

There are differences between the SLR curves that were used for this project and the SLR curves on the USACE Sea-Level Calculator for Non-NOAA Long-Term Tide Gauges Web Page. For this project, the latest ER (ER 1100-2-8162, dated 31 December 2013) as well as local gages in the project area were used. Extensive time was spent in analyzing the gage data and subsidence, while maintaining as much accuracy as possible. SLR is the effect of eustatic sea level rise and subsidence. The rate of eustatic sea level rise may be the same, generally speaking, but the rate of subsidence in Louisiana varies from one place to another (and it is not a linear relationship). Thus, the SLR curves used for this report were appropriate for the project area.

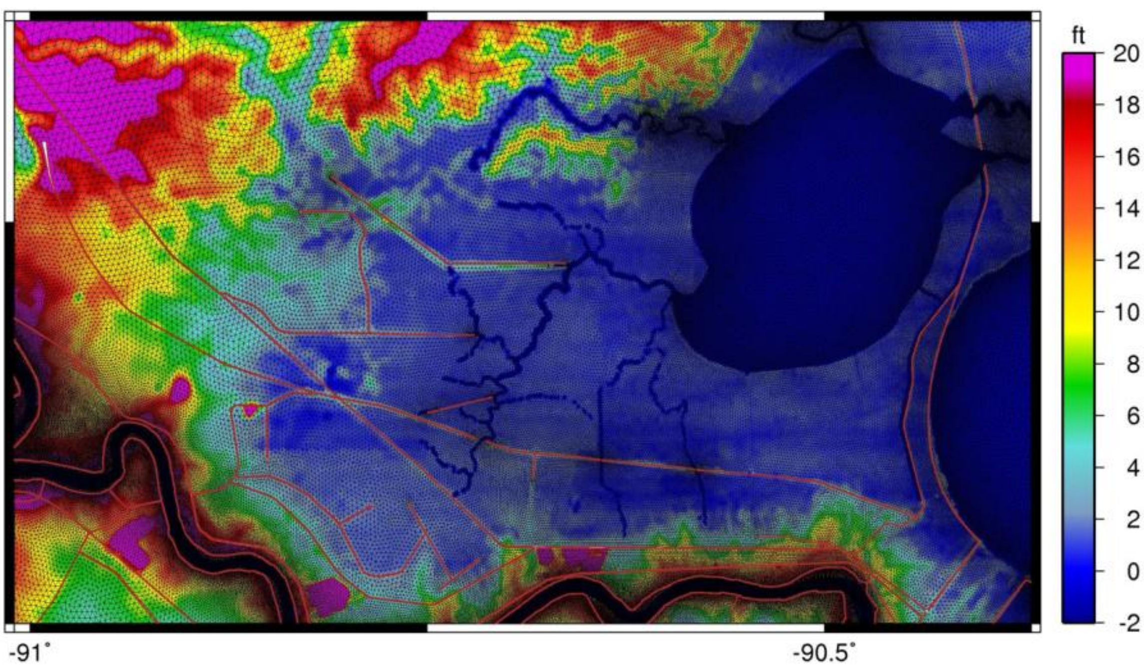


Figure 2 Mesh Elevations and Raised Feature Alignments in the IHNC Study ADCIRC Mesh. Contours are in feet relative to NAVD88 (2004.65 Epoch). Black lines represent element edges and display mesh resolution.

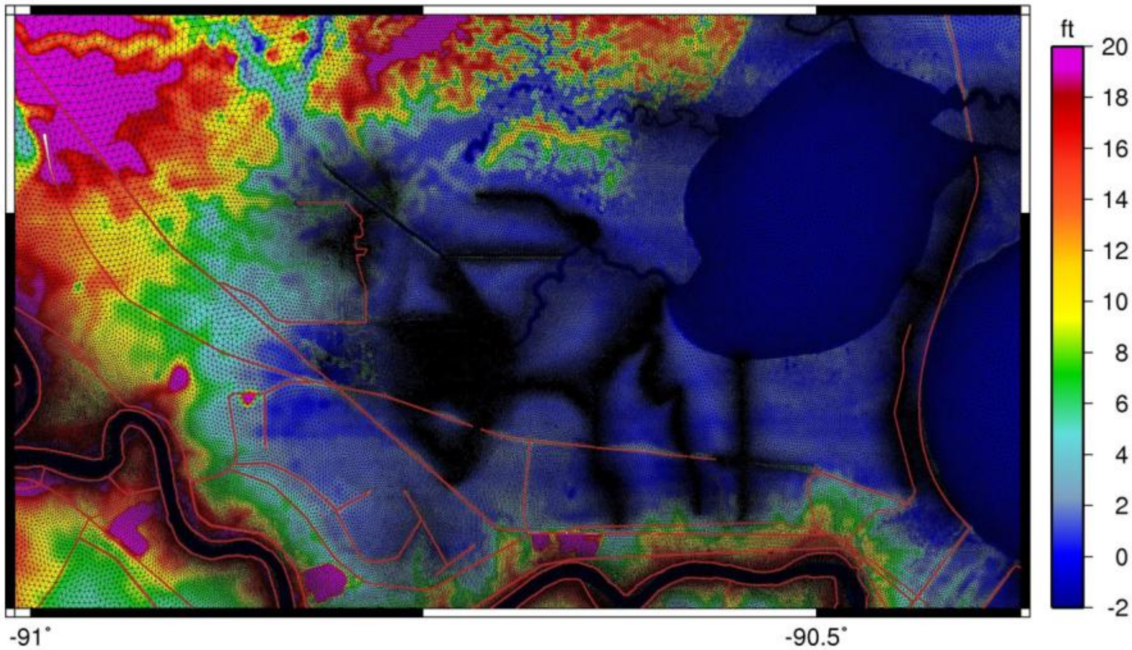
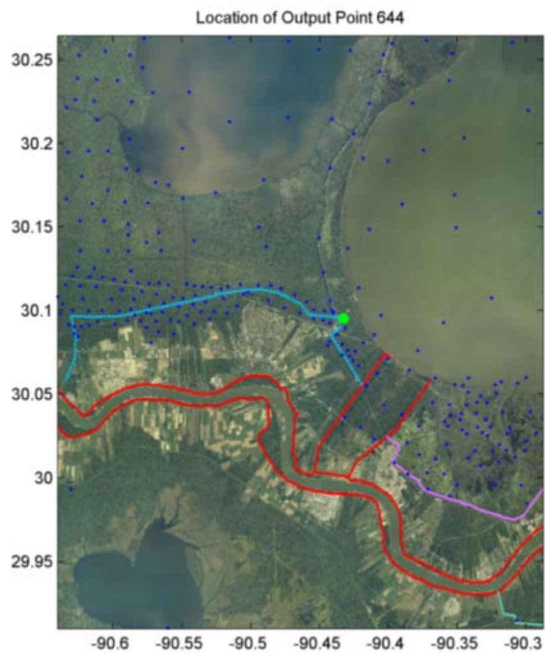


Figure 3 Mesh Elevations and Raised Feature Alignments in the ADCIRC Mesh. Contours are in feet relative to NAVD88 (2004.65 Epoch). Black lines represent element edges and display mesh resolution.

Figure 4 compares peak surge results from the 2011 base condition to the "IHNC 152" base condition. The "IHNC 152" was a suite of 152 simulations used to determine design conditions for the IHNC surge barrier. The IHNC grid, as pictured in Figure 2, does not contain high resolution in the project area. If the peak surge results are compared storm by storm from this project and the IHNC simulation, it can be determined what the effect of the added resolution on stage-frequency is in the project area. The left portion of Figure 4 displays the location of the output point represented as a green dot. This location was selected at the St. Charles Parish portion on HSDRRS, which is represented by a light purple line. The MRL is represented by the red line and Alignment C is represented as a blue line. The right portion of the figure is a regression analysis between the IHNC 152 and the project Base Condition. At the St. Charles Parish location, the surge results are nearly equal for both sets of simulations. Both suites model the same 152 storms, which allows processing in the JPM-OS statistical code. The 50yr, 100yr, 200yr and 500yr returns are plotted for both analyses in blue. For example, the 100yr surge for the IHNC set is 11.8 ft. NAVD88, while the 100yr surge for the project set is 12.0 ft. NAVD88. In summary, at this location, which is located away from the added resolution, the effect of resolution on statistical output results in a 0.1 ft. increase at the 50yr level, a 0.2 ft. increase at the 100yr level, a 0.2 ft. increase at the 200yr level and a 0.3 ft. increase at the 500yr level. It is important to note that the IHNC 152 surge analysis results in the St. Charles Parish area were modified prior to final design. Therefore, the IHNC 152 stage-frequency data presented in Figure 3 is different than what was actually used in HSDRRS design.

Figure 5 compares peak surge results from the 2011 base condition to the "IHNC 152" base condition at Reach 5 of the Alignment C levee. At this output point, the effect of resolution on statistical output results in a 0.3 ft. increase at the 50yr level, a 0.2 ft. increase at the 100yr level, a 0.3 ft. increase at the 200yr level and a 0.4 ft. increase at the 500yr level. Stage-frequency information was developed for the 2011 base condition using the same JPM-OS code as used for the HSDRRS design analysis.

With the 2011 base condition stage-frequency established, the stage-frequency for the 4 SLR conditions, including with- and without-project are processed using a regression analysis. Figure 6 displays a regression analysis between peak surge values of the 2011 base condition and the peak surge values from the Year 2020 Alignment C surge values. In this case, 52 storms are available for the regression. The trend line, as plotted in green, is used to estimate 50yr, 100yr, 200yr and 500yr surge values for the Year 2020 Alignment C condition. At this location, the 100yr for the Year 2020 Alignment C condition is estimated to be 12.2 ft. NAVD88. Figure 7 displays a regression analysis between the Year 2020 Alignment C condition, and the Year 2020 base condition. The trend line in Figure 7 is used to estimate the stage-frequency for the Year 2020 base condition. At this location, the Year 2020 base condition 100yr surge is estimated to be 10.6 ft. NAVD88.



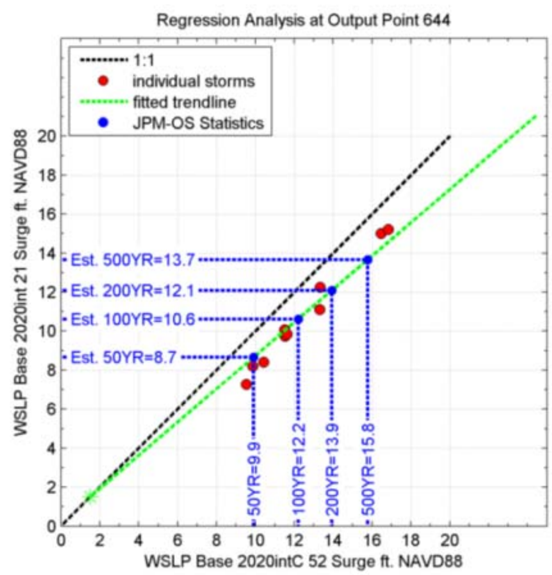


Figure 4 Comparison of IHNC 152 peak storm surge values and the Project 152 Base 2011 peak storm surge values at the St. Charles Parish HSDRRS Levee

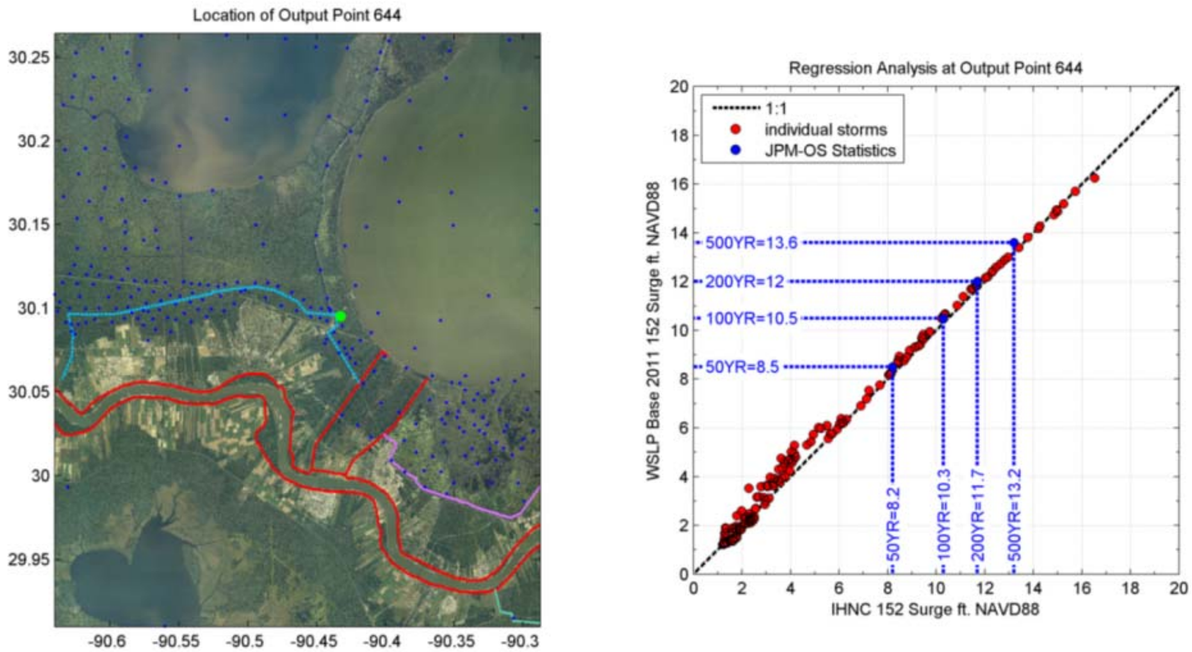


Figure 5 Comparison of IHNC 152 peak storm surge values and Project 152 Base 2011 peak storm surge values at Reach 5 of the project levee

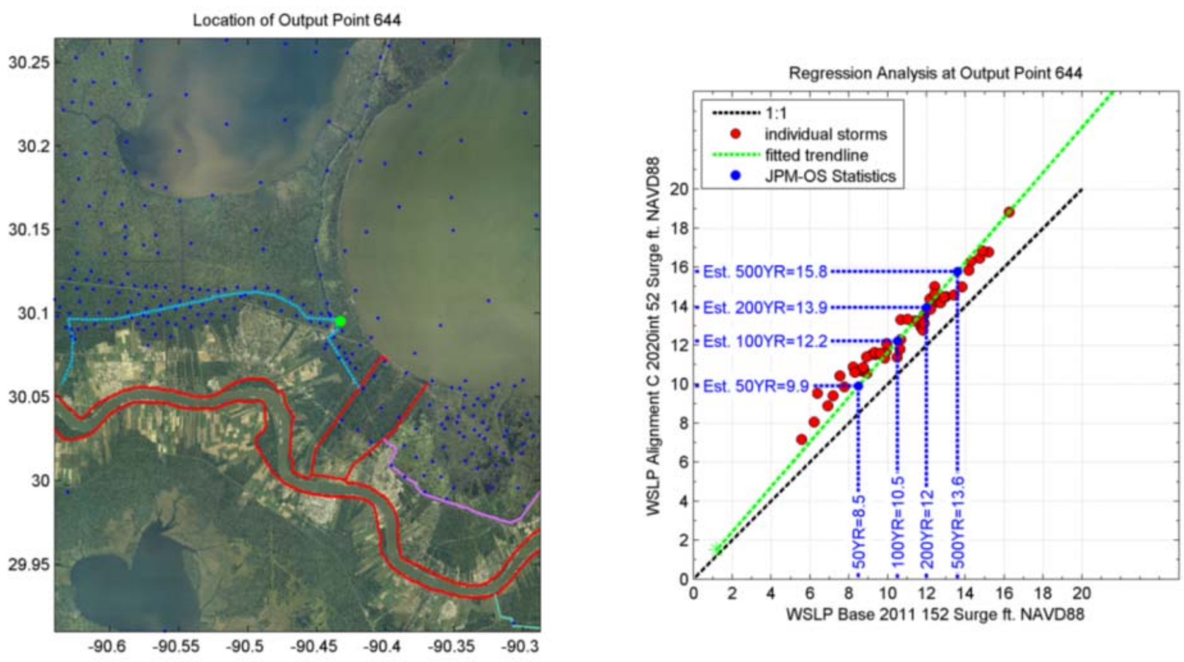


Figure 6 Comparison of Project 152 Base 2011 storm surge values and Project 52 Alignment C 2020 peak storm surge values at Reach 5 of the project levee

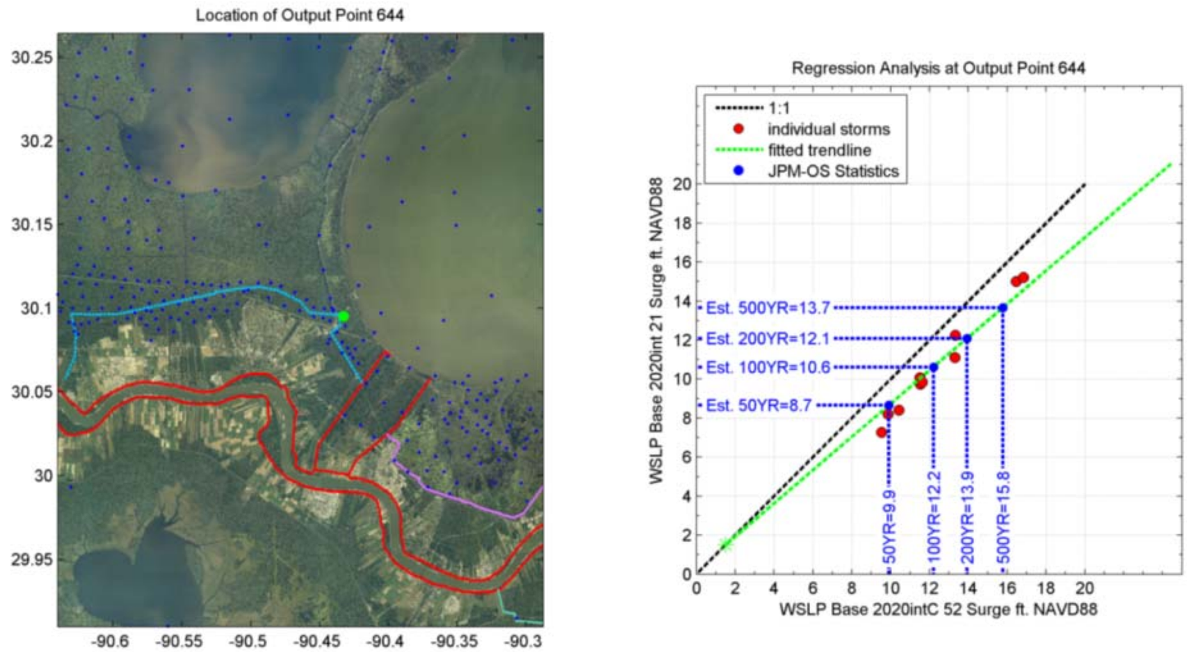


Figure 7 Comparison of Project 52 Alignment C 2020 storm surge values and Project 21 Base 2020 peak storm surge values at Reach 5 of the project levee

Figure 8 displays a regression between peak surge values from the 2011 Base Condition and the Year 2070 low SLR Alignment C condition. The trend line allows estimation of the stage-frequency data for the Year 2070 low SLR Alignment C condition. For example, the 100yr elevation is estimated to be 13.8 ft. NAVD88, based on the trend line. Figure 9 displays a regression plot between peak surge from the Year 2070 low SLR Alignment C condition and the Year 2070 low SLR project base condition. This regression trend line allows estimation of stage-frequency for the Year 2070 low SLR project base condition. For example, the 100yr elevation is estimated to be 12.7 ft. NAVD88.

The same regression analysis is applied for the Year 2070 intermediate SLR condition and the Year 2070 high SLR condition. Figure 10 and Figure 11 display the regression analysis for the Year 2070 intermediate SLR condition. Figure 12 and Figure 13 display the regression analysis for the Year 2070 high SLR condition.

Table 7 displays the final developed stage-frequency data for the project analysis for Years 2020 and 2070 conditions. The table contains stage-frequency data for all 7 design reaches.

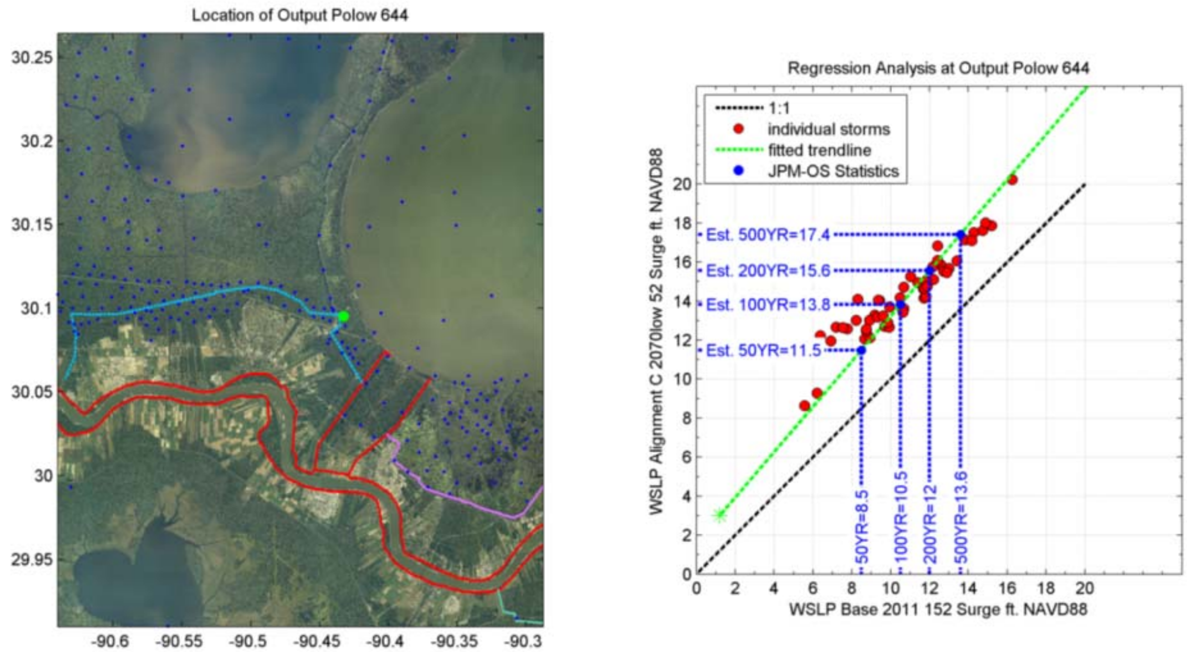


Figure 8 Comparison of Project 152 Base 2011 storm surge values and 52 Alignment C 2070 Low SLR peak storm surge values at Reach 5 of the project levee

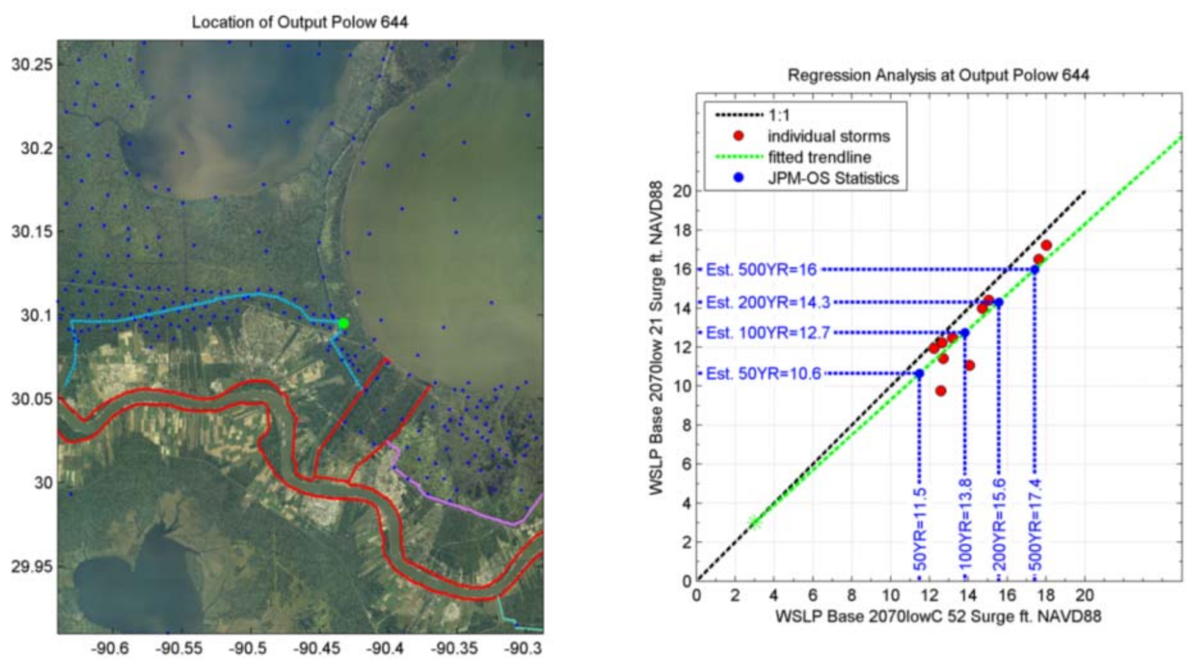


Figure 9 Comparison of 52 Alignment C 2070 Low SLR storm surge values and Project 21 Base 2070 Low SLR peak storm surge values at Reach 5 of the project levee

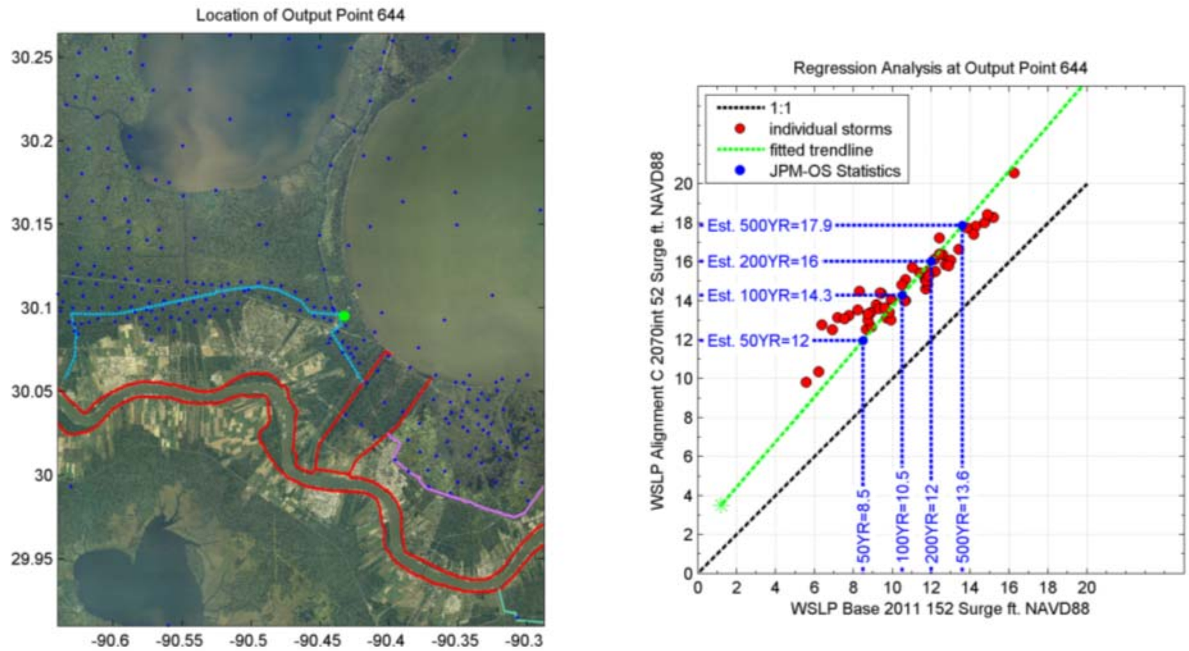


Figure 10 Comparison of 152 Base 2011 storm surge values and 52 Alignment C 2070 Intermediate SLR peak storm surge values at Reach 5 of the project levee

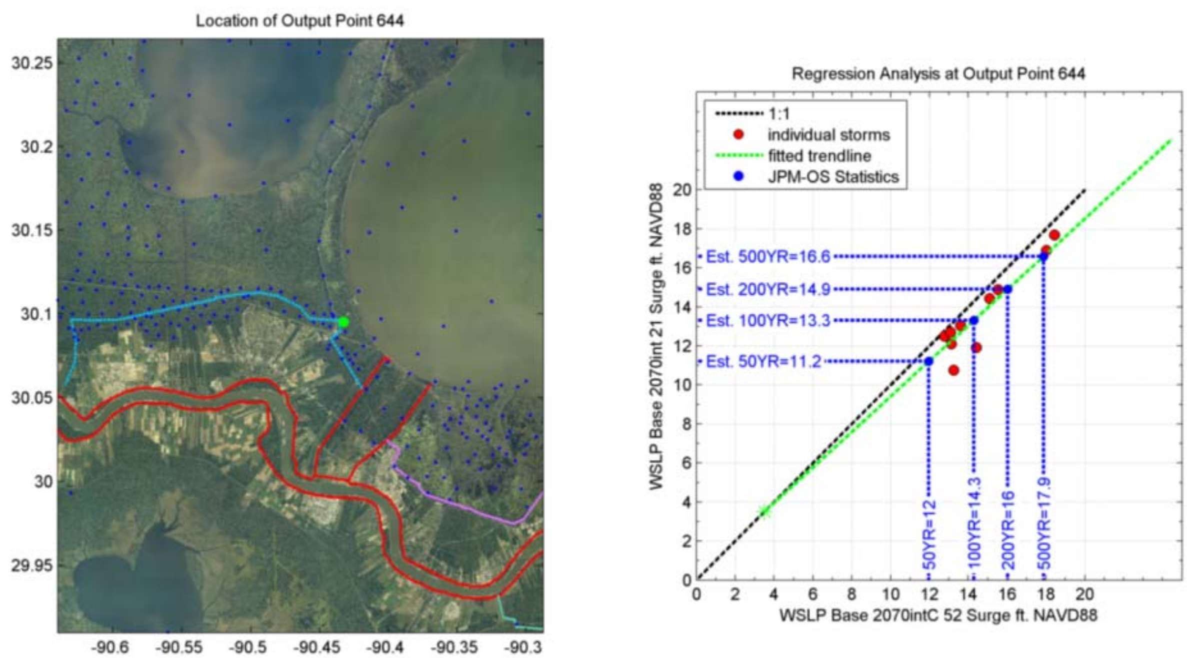


Figure 11 Comparison of 52 Alignment C 2070 Intermediate SLR storm surge values and 21 Base 2070 Intermediate SLR peak storm surge values at Reach 5 of the project levee

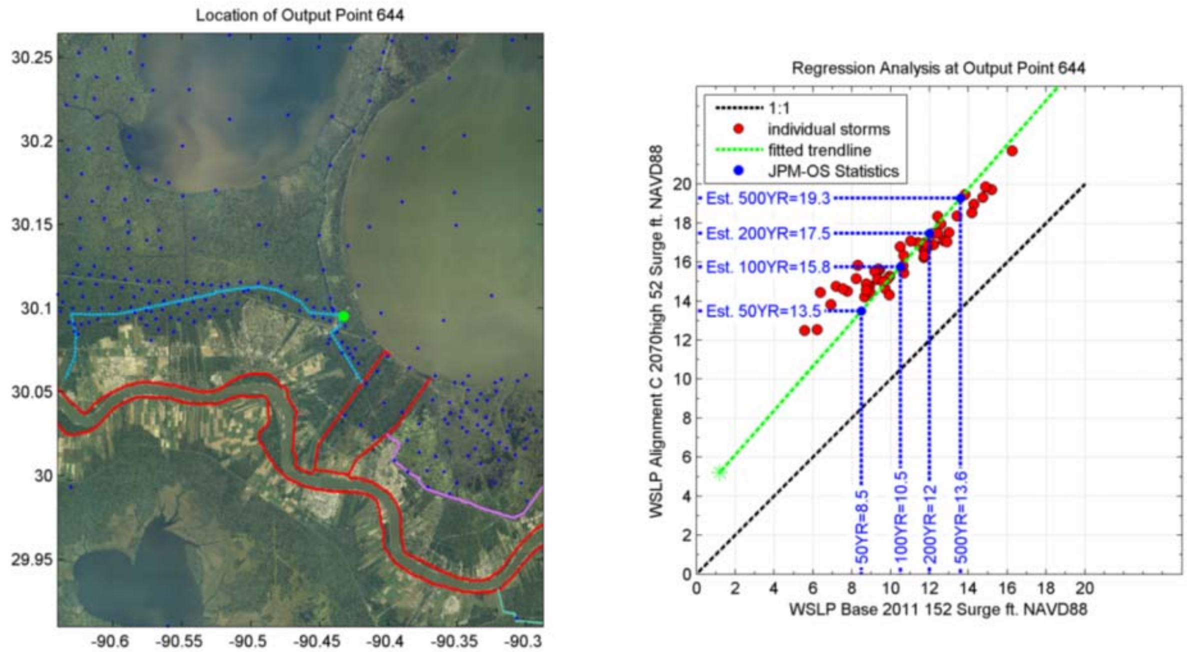


Figure 12 Comparison of 152 Base 2011 storm surge values and 52 Alignment C 2070 High SLR peak storm surge values at Reach 5 of the project levee

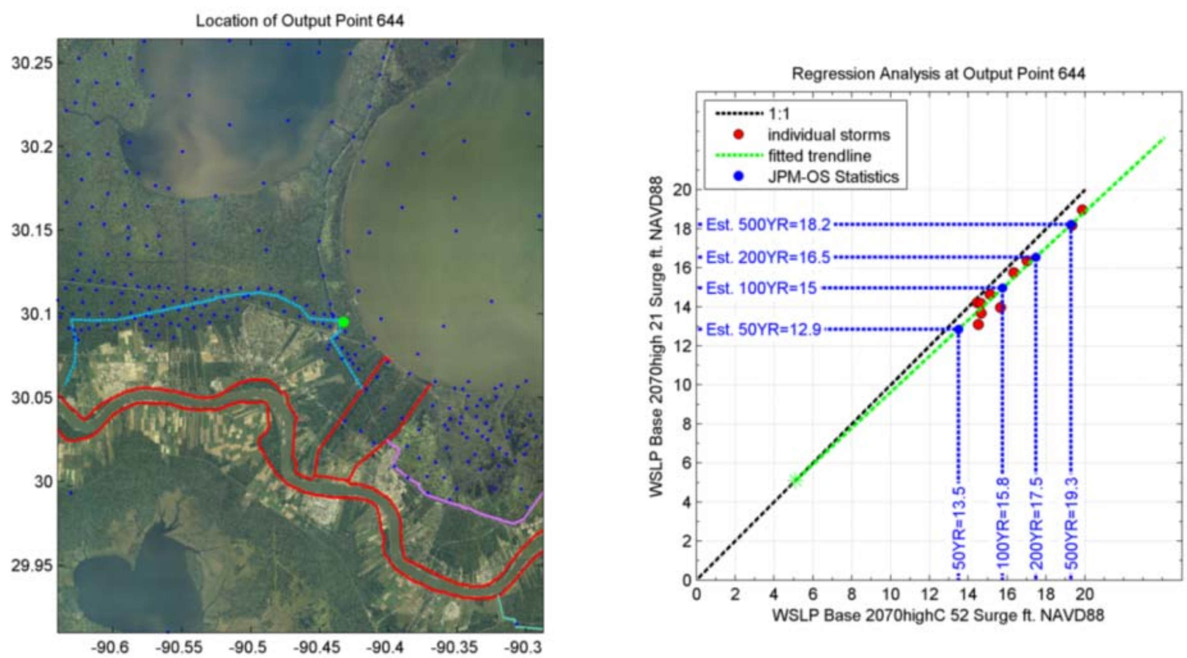


Figure 13 Comparison of 52 Alignment C 2070 High SLR storm surge values and 21 Base 2070 High SLR peak storm surge values at Reach 5 of the project levee

Table 7 Final stage frequency estimates for the WSLP levee design

			Without Project Surge Elevation (ft. NAVD88)				With Project / Alignment C Surge Elevation (ft. NAVD88)				Difference (ft. / %)							
Condition	**Reach ID**	**ADCIRC Output Point**	50YR	**100YR**	200YR	500YR	50YR	**100YR**	200YR	500YR	50YR		**100YR**		200YR		500YR	
2020 int	1	534	4.5	**5.7**	6.6	7.8	4.5	**5.7**	6.6	7.9	0.0	1%	**0.0**	**1%**	0.0	1%	0.1	1%
2020 int	2	439	5.5	**6.7**	7.8	9.1	5.9	**7.2**	8.4	9.9	0.4	7%	**0.5**	**7%**	0.6	11%	0.8	9%
2020 int	3	337	6.8	**8.1**	9.1	10.1	7.1	**8.5**	9.5	10.6	0.3	5%	**0.4**	**4%**	0.5	7%	0.5	5%
2020 int	4	365	8.0	**9.6**	10.8	12.1	9.1	**10.9**	12.3	13.8	1.1	13%	**1.3**	**12%**	1.5	17%	1.7	14%
2020 int	5	644	8.7	**10.6**	12.1	13.7	9.9	**12.2**	13.9	15.8	1.2	14%	**1.6**	**13%**	1.8	19%	2.1	15%
2020 int	6	117	10.4	**12.1**	13.4	14.6	10.5	**12.2**	13.5	14.7	0.1	1%	**0.1**	**1%**	0.1	1%	0.1	1%
2020 int	7	132	10.2	**11.9**	13.2	14.5	10.3	**12.0**	13.3	14.6	0.1	1%	**0.1**	**1%**	0.1	1%	0.1	1%
2070 low	1	534	8.3	**10.3**	11.9	14.1	8.4	**10.4**	12.1	14.3	0.1	1%	**0.1**	**1%**	0.2	2%	0.2	2%
2070 low	2	439	9.3	**11.2**	12.9	14.9	9.7	**11.7**	13.5	15.7	0.4	4%	**0.5**	**4%**	0.6	7%	0.8	6%
2070 low	3	337	10.0	**11.8**	13.1	14.5	10.4	**12.2**	13.6	15.0	0.3	3%	**0.4**	**3%**	0.5	5%	0.5	4%
2070 low	4	365	9.3	**10.8**	12.0	13.2	11.0	**12.9**	14.4	16.0	1.7	18%	**2.1**	**18%**	2.4	22%	2.8	19%
2070 low	5	644	10.7	**12.7**	14.3	16.0	11.5	**13.8**	15.6	17.4	0.8	8%	**1.1**	**8%**	1.3	11%	1.4	9%
2070 low	6	117	12.6	**14.4**	15.8	17.1	12.6	**14.4**	15.8	17.1	0.0	0%	**0.0**	**0%**	0.0	0%	0.0	0%
2070 low	7	132	13.2	**15.2**	16.7	18.2	13.2	**15.2**	16.7	18.2	0.0	0%	**0.0**	**0%**	0.0	0%	0.0	0%
2070 int	1	534	9.0	**11.1**	12.9	15.1	9.1	**11.2**	12.9	15.2	0.0	0%	**0.0**	**0%**	0.1	1%	0.1	1%
2070 int	2	439	10.0	**12.0**	13.8	15.9	10.4	**12.5**	14.4	16.6	0.4	3%	**0.5**	**3%**	0.5	5%	0.7	5%
2070 int	3	337	10.6	**12.4**	13.8	15.2	11.0	**12.9**	14.3	15.8	0.4	3%	**0.5**	**3%**	0.5	5%	0.6	4%
2070 int	4	365	11.1	**13.0**	14.4	15.9	11.5	**13.4**	14.9	16.5	0.4	3%	**0.4**	**3%**	0.5	4%	0.6	4%
2070 int	5	644	11.2	**13.3**	14.9	16.6	12.0	**14.3**	16.0	17.9	0.8	7%	**1.0**	**7%**	1.1	9%	1.3	8%
2070 int	6	117	12.9	**14.7**	16.1	17.4	13.0	**14.9**	16.2	17.5	0.1	1%	**0.1**	**1%**	0.1	1%	0.2	1%
2070 int	7	132	13.6	**15.5**	17.0	18.5	13.6	**15.6**	17.1	18.6	0.1	1%	**0.1**	**1%**	0.1	1%	0.1	1%
2070 high	1	534	11.3	**13.6**	15.5	18.0	11.2	**13.5**	15.4	17.9	-0.1	-1%	**-0.1**	**-1%**	-0.1	-1%	-0.1	-1%
2070 high	2	439	12.2	**14.4**	16.3	18.6	12.5	**14.7**	16.6	19.0	0.2	2%	**0.3**	**2%**	0.4	3%	0.4	3%
2070 high	3	337	12.7	**14.6**	16.0	17.5	13.0	**14.9**	16.4	17.9	0.3	2%	**0.4**	**2%**	0.4	3%	0.5	3%
2070 high	4	365	12.8	**14.7**	16.1	17.6	13.0	**15.0**	16.4	18.0	0.2	2%	**0.3**	**2%**	0.3	2%	0.4	2%
2070 high	5	644	12.9	**15.0**	16.5	18.2	13.5	**15.8**	17.5	19.3	0.6	5%	**0.8**	**5%**	0.9	7%	1.1	6%
2070 high	6	117	14.3	**16.1**	17.4	18.6	14.5	**16.3**	17.6	18.9	0.2	1%	**0.2**	**1%**	0.2	1%	0.2	1%
2070 high	7	132	14.9	**16.8**	18.2	19.7	15.0	**16.9**	18.4	19.9	0.1	1%	**0.1**	**1%**	0.2	1%	0.2	1%

Wave Conditions

Figure 14 displays peak significant wave heights from the 2011 Base Condition and the Year 2020 Alignment C condition. The raw STWAVE significant wave heights at point 644 (also known as Reach 5) are unrealistically high given the conditions surrounding the project area. STWAVE does not incorporate the effects of this vegetation. In the model, it was apparent that larger waves that form in Lake Pontchartrain are allowed to propagate to the levee. In reality, this propagation will not occur because the vegetation is simply too thick and too tall to allow it.

Figure 15 displays an aerial image of the project area. Currently, approximately one mile of dense canopy exists between Lake Pontchartrain and the most exposed portion of the project levee. This canopy is not accounted for in the Steady State Spectral Wave (STWAVE) model, allowing unrealistic larger waves to occur in the modeling.

Based on engineering judgment, the significant wave height and peak wave period used for the levee design are the minimum recommended wave height/wave period for coastal structure. For existing conditions, the significant wave heights are set to 1.5 ft. and the peak wave periods are set to 2.5 sec. For future conditions, the significant wave heights are set to 2.5 ft. and the peak wave periods are set to 3.0 sec.

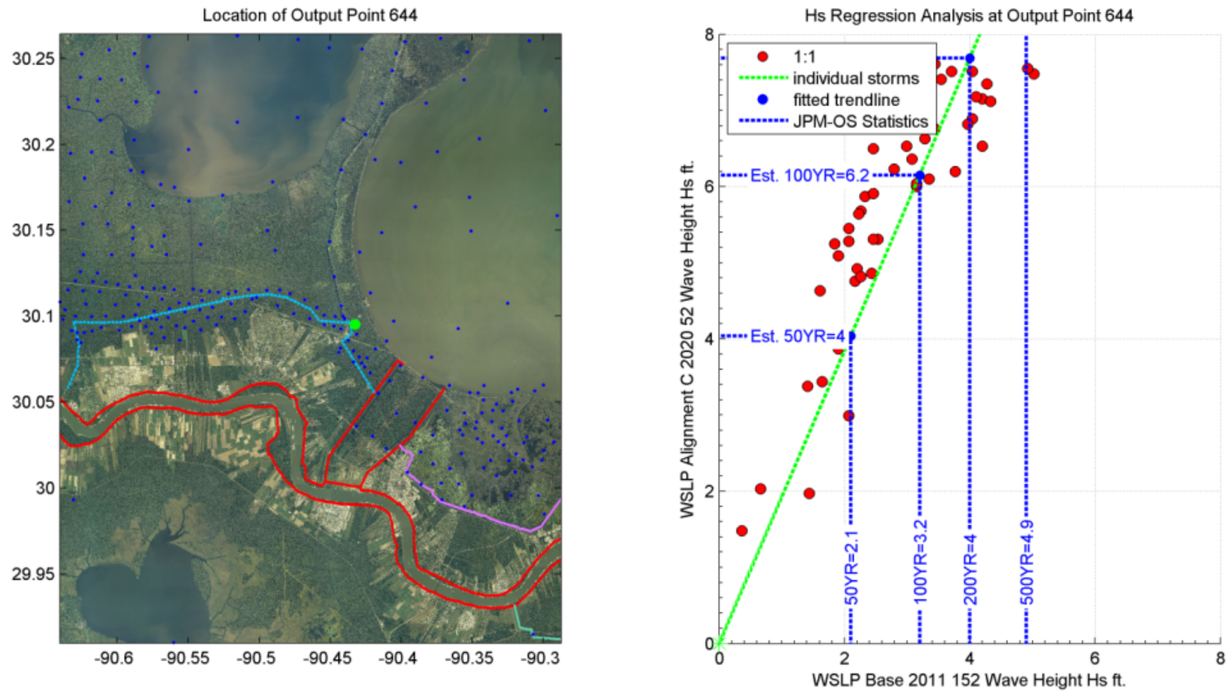


Figure 14 Wave Heights for 2011 Base Condition and Year 2020 Alignment C Condition

Figure 15 Recommended Alignment with Dense Canopy

Levee Design

The following section describes how the final levee design elevations were determined for the project for Year 2020 and Year 2070 conditions for the 50yr, 100yr and 200yr hydraulic boundary conditions.

The hydraulic and geometric parameters in the levee design approach are uncertain. For instance, there are errors in the computed surge elevation near the levees / floodwalls by the ADCIRC / STWAVE models. The coefficients of the empirical overtopping equations are calibrated against laboratory and field experiments and are inherently uncertain. It is believed that the uncertainty in these parameters should be taken into account in the design process to come up with a robust design. This section describes the method used which accounts for uncertainties in water elevations and waves, and computes the overtopping rate with state-of-the-art formulations. The objective of this method is to ensure that overtopping criteria can be met with a certain level of confidence due to the uncertainties.

A common way of dealing with uncertainties is the application of a Monte Carlo analysis. In the Monte Carlo analysis, the overtopping algorithm is repeated to compute the overtopping rate many times. Based on these outputs, a statistical distribution can be derived from the resulting overtopping rates. The parameters that are included in the Monte Carlo analysis are the 1% surge elevation, wave height and wave period. Uncertainties in the geometric parameters are not included; it is assumed that the proposed heights and slopes in the final design document are minimum values that will be constructed.

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 (Hs) and the peak wave period (Tp).

Van der Meer overtopping formulations
The overtopping formulation from Van der Meer reads (TAW, 2002):

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b \xi_0 \exp\left(-4.75\frac{R_c}{H_{m0}}\frac{1}{\xi_0 \gamma_b \gamma_f \gamma_\beta \gamma_v}\right)$$

$$with \quad \max imum: \quad \frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f \gamma_\beta}\right) \qquad (1)$$

With:
q : average overtopping rate [cfs/ft]
g : gravitational acceleration [ft/s2]
Hm0 : wave height at toe of the structure [ft]
ξ0: surf similarity parameter [-]
α : slope [-]
Rc : 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:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.92}\exp\left(-\frac{R_c}{\gamma_f \gamma_\beta H_{m0}(0.33+0.022\xi_0)}\right) \qquad (2)$$

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 16 graphically shows the overtopping for a levee and floodwall situation including the most relevant parameters.

In the design process, the best estimate 1% values is used for these parameters from the JPM-OS method (White Paper, 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.

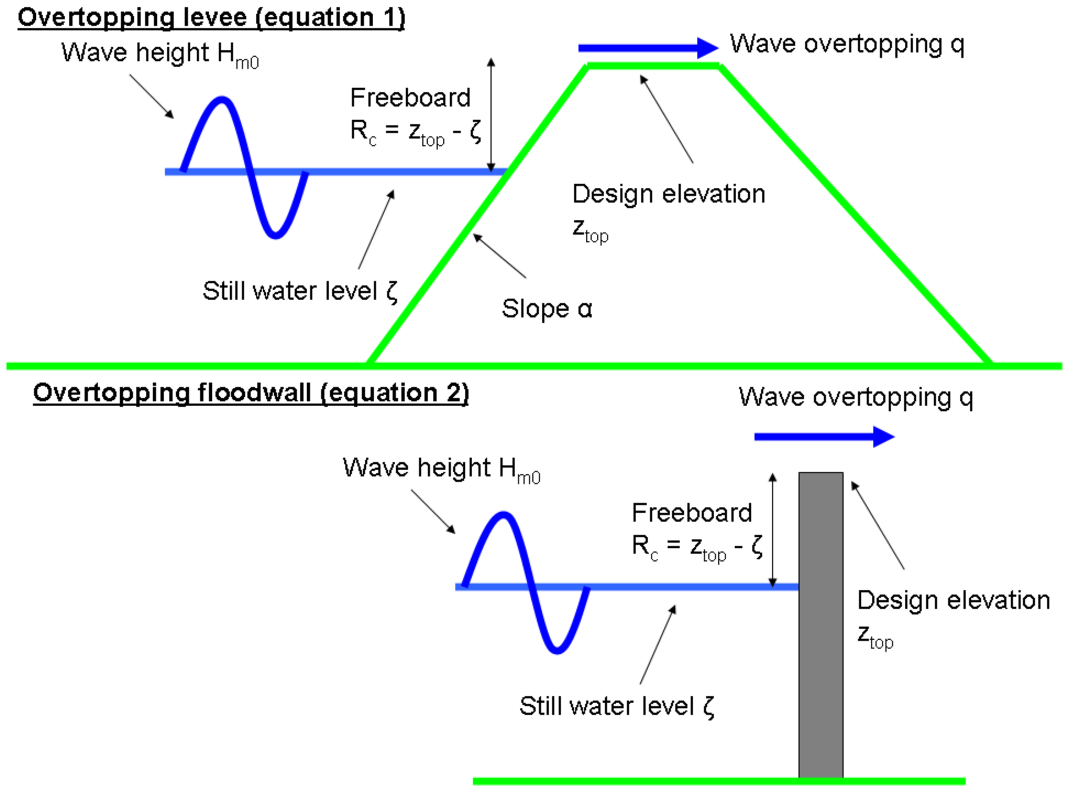


Figure 16 Definitions for Overtopping for Levee and Floodwall

The Monte Carlo Analysis is executed as follows:

1. Draw a random number between 0 and 1 to set the 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 steps 3. and 4. above for the three overtopping coefficients independently.
6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in steps 2., 4. and 5. above 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 Steps 1. through 5. above 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.

Results

Figure 17 displays an example of MATLAB Monte Carlo-based output for the 50yr design of segment 1 for Year 2020 conditions with a 1:4 levee slope. The final 50yr design elevation at 7.0 ft. NAVD88 was selected to limit the overtopping rates below 0.01 cfs/ft. with 50% assurance, and limit the overtopping rate below 0.10 cfs/ft. with 90% assurance. For a robust design, the Monte Carlo-based design methodology accounts for the uncertainty of the hydraulic boundary conditions, and the uncertainty in the Van der Meer overtopping equations. Table 8 contains the final design elevations for the 50yr, 100yr and 500yr conditions for all 4 SLR scenarios. Design elevations are determined for both 1:3 and 1:4 levee slopes. Figure 18 displays the 7 design reaches for the project.

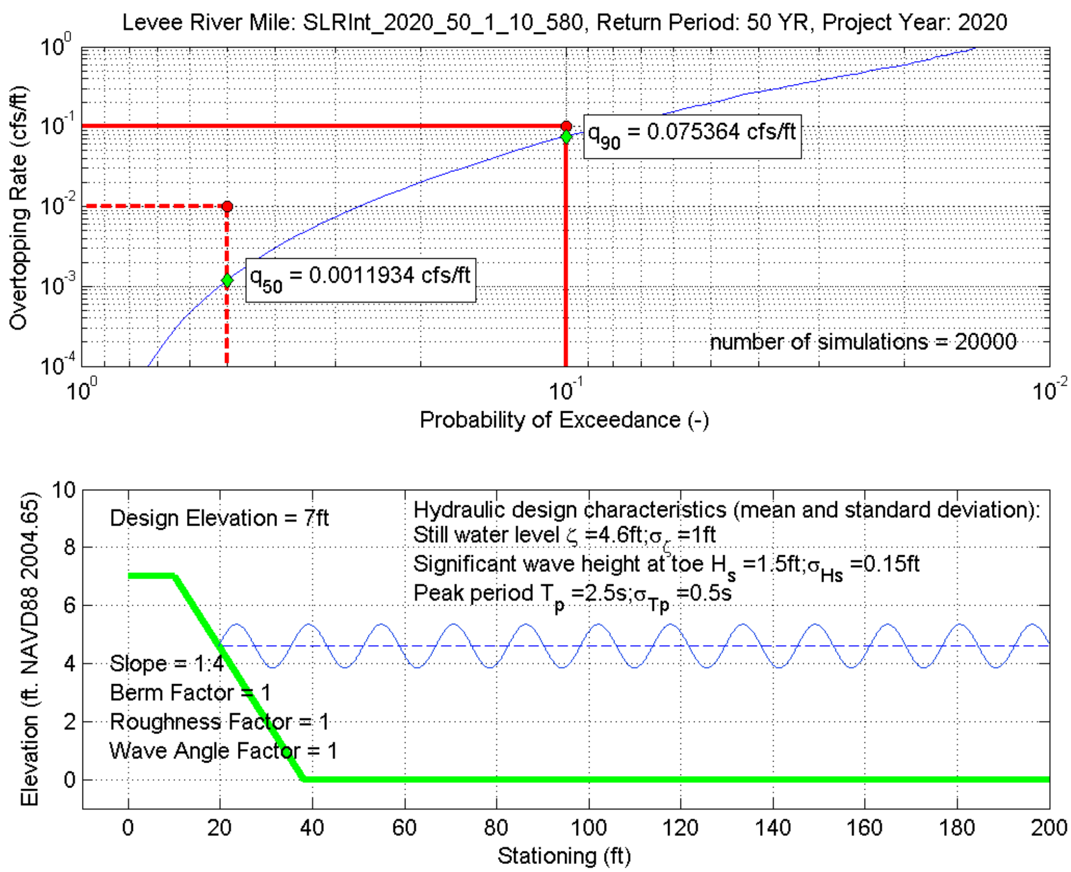


Figure 17 Monte Carlo-Based Hydraulic Design Output for Reach 1, 50yr, Year 2020

Table 8 Final Design Elevations (All elevations are in ft. NAVD88)

Levee Section	Reach	ID	Design Year	50YR Levee Elevation with 1:3 slope	100YR Levee Elevation with 1:3 slope	200YR Levee Elevation with 1:3 slope	50YR Levee Elevation with 1:4 slope	100YR Levee Elevation with 1:4 slope	200YR Levee Elevation with 1:4 slope
2020int_50_1_534	1	534	2020 int	7	8.5	10	6.5	8	9.5
2020int_50_2_439	2	439	2020 int	8.5	10	11.5	8	9.5	11
2020int_50_3_337	3	337	2020 int	9.5	11.5	12.5	9.5	11	12.5
2020int_50_4_365	4	365	2020 int	11.5	14	15.5	11.5	13.5	15
2020int_50_5_644	5	644	2020 int	12.5	15	17	12	14.5	16.5
2020int_50_6_117	6	117	2020 int	13	15	16.5	12.5	14.5	16.5
2020int_50_7_132	7	132	2020 int	13	15	16.5	12.5	14.5	16
2070low_50_1_534	1	534	2070 low	12.5	15	17	12	14	16
2070low_50_2_439	2	439	2070 low	14	16	18	13	15.5	17.5
2070low_50_3_337	3	337	2070 low	14.5	16.5	18.5	14	16	17.5
2070low_50_4_365	4	365	2070 low	15.5	17.5	19	14.5	16.5	18.5
2070low_50_5_644	5	644	2070 low	16	18.5	20.5	15	17.5	19.5
2070low_50_6_117	6	117	2070 low	17	19	20.5	16	18	19.5
2070low_50_7_132	7	132	2070 low	17.5	19.5	21.5	16.5	19	20.5
2070int_50_1_534	1	534	2070 int	13.5	16	17.5	12.5	15	17
2070int_50_2_439	2	439	2070 int	14.5	17	19	14	16	18.5
2070int_50_3_337	3	337	2070 int	15.5	17.5	19	14.5	16.5	18.5
2070int_50_4_365	4	365	2070 int	16	18	19.5	15	17	19
2070int_50_5_644	5	644	2070 int	16.5	19	21	15.5	18	20
2070int_50_6_117	6	117	2070 int	17.5	19.5	21	16.5	18.5	20
2070int_50_7_132	7	132	2070 int	18	20	22	17	19.5	21
2070high_50_1_534	1	534	2070 high	15	18	20.5	14	17	19.5
2070high_50_2_439	2	439	2070 high	16	19	22	15.5	18.5	21
2070high_50_3_337	3	337	2070 high	17	19.5	21.5	16	18.5	21
2070high_50_4_365	4	365	2070 high	17	19.5	21.5	16	18.5	20.5
2070high_50_5_644	5	644	2070 high	17.5	20.5	22.5	16.5	19.5	21.5
2070high_50_6_117	6	117	2070 high	18.5	21	22.5	17.5	20	22
2070high_50_7_132	7	132	2070 high	19	21.5	23.5	18	20.5	22.5

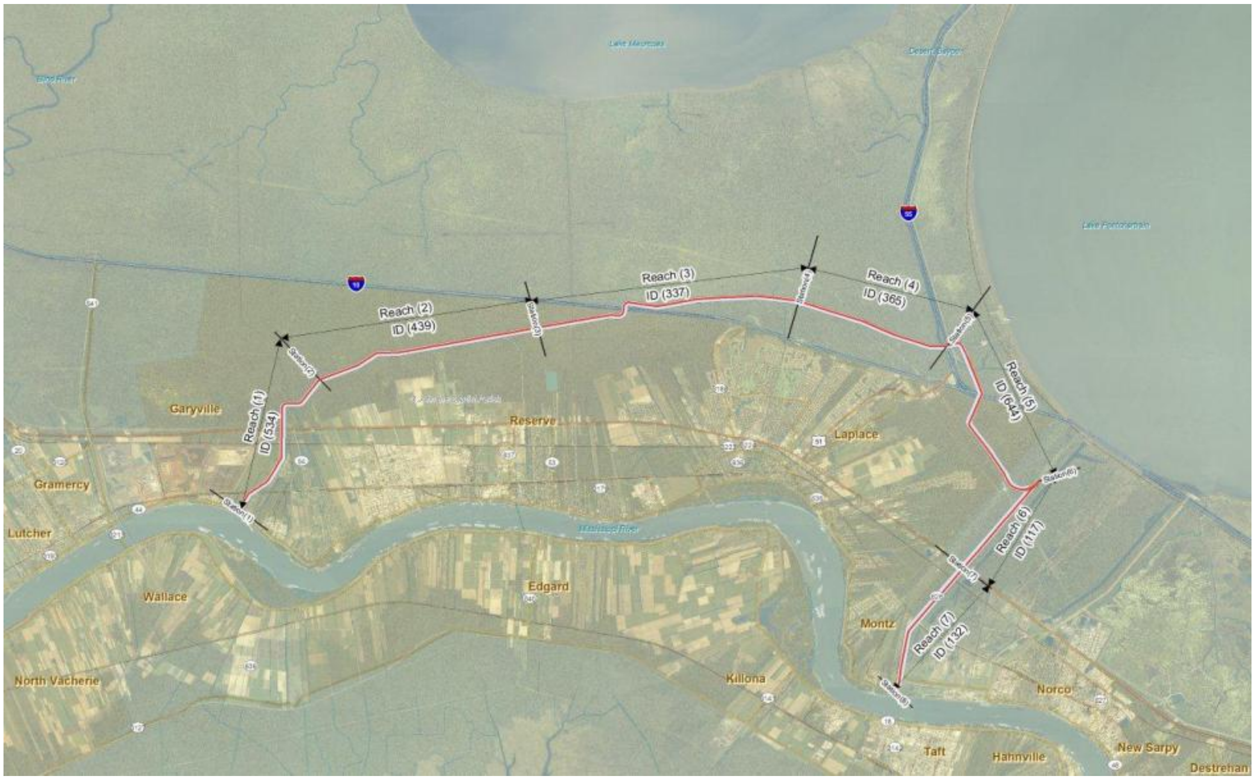


Figure 18 Project Levee Reaches

Sub-planning Stage Frequency

In the coastal area, the risk of flooding is dominated by storm surge. Inland areas might be more prone to flooding by heavy rainfall. In the analysis, both hazards have been evaluated. In order to conduct the economic analysis, the 2-, 5-, 10-, 25-, 50-,100-, 200- and 500-year stages have been developed for each of the sub-planning units in the study area.

The storm surge modeling does not include the effects of rainfall. The storm surge modeling is not capable of producing stages for higher frequency events such as the 2-, 5-, 10- and 25-year events. The suite of storms selected for the modeling is selected to produce stage frequencies for 50-year events and above. Therefore, for higher frequency events, it is preferable to use gage data for developing the stage-frequency. However, no long term gage data is available for the project area.

For the project area sub-planning units, the stage-frequency data developed through the hydrologic modeling were combined with the stage-frequency data developed through the surge modeling so as to develop complete stage-frequency data for the economic analysis.

Tables that contain the combined stage-frequency curves for the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year events for each sub-planning unit (for with- and without-project) were provided for economic analysis.

Induced Flooding

Stage-frequency data were developed for each of the sub-basins in the economic analysis. Figure 19 displays the 100-year stillwater elevations for the Base Year 2020 condition. The values at these locations include the effects of rainfall and surge as discussed in the previous section. Figure 20 displays the 100-year stillwater elevations for the with-project Year 2020 condition. Figure 21 displays the difference in the 100-year stillwater elevations between the with- and without-project condition for Year 2020. A positive number represents an increase due to the Alignment C condition. Figure 22 displays the difference in the 100-year stillwater elevations between the with- and without-project condition for Year 2070 with intermediate SLR.

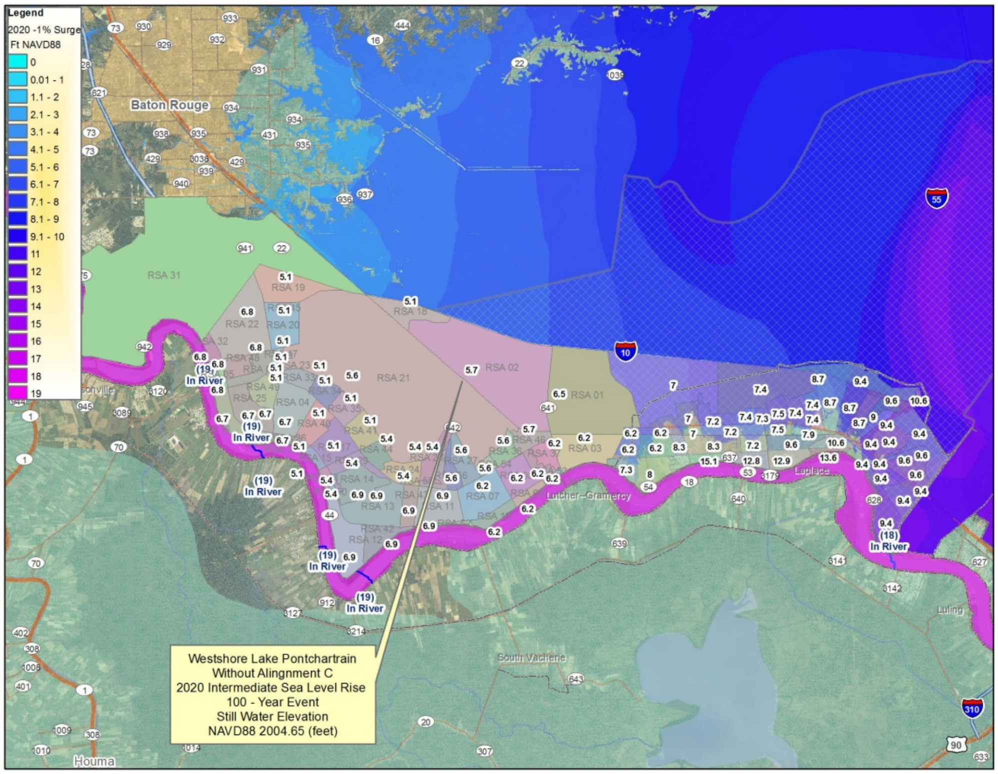


Figure 19 100-Year Stillwater Elevations for Year 2020-Intermediate SLR Condition – Without-Project

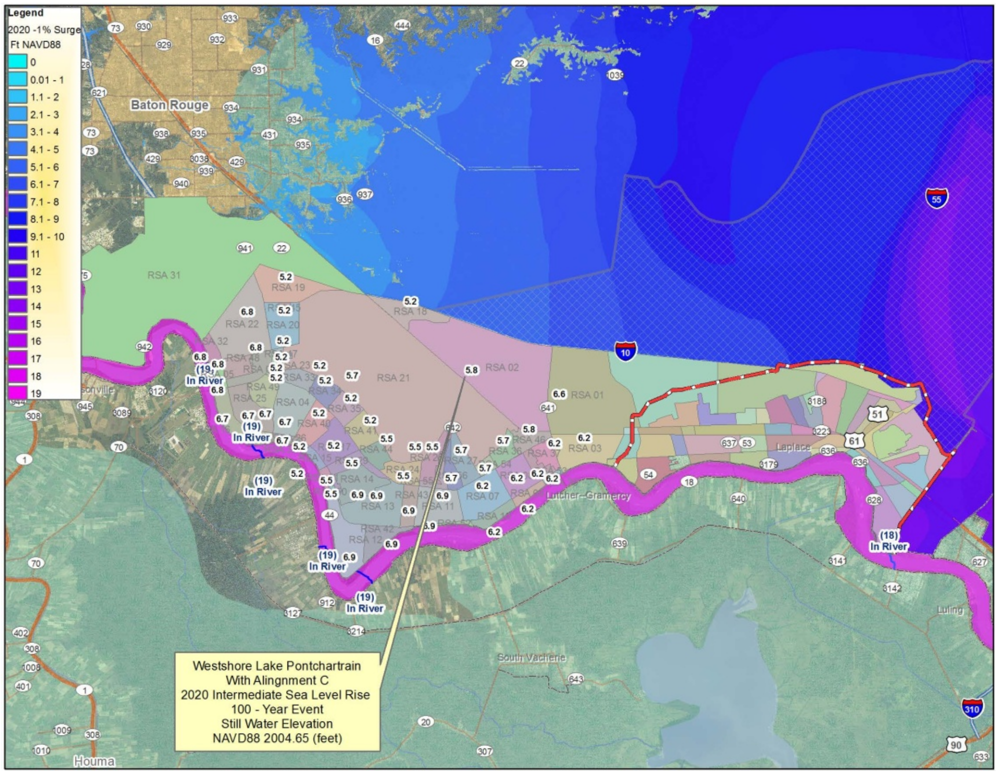


Figure 20 100-Year Stillwater Elevations for Year 2020-Intermediate SLR Condition – With Alignment C

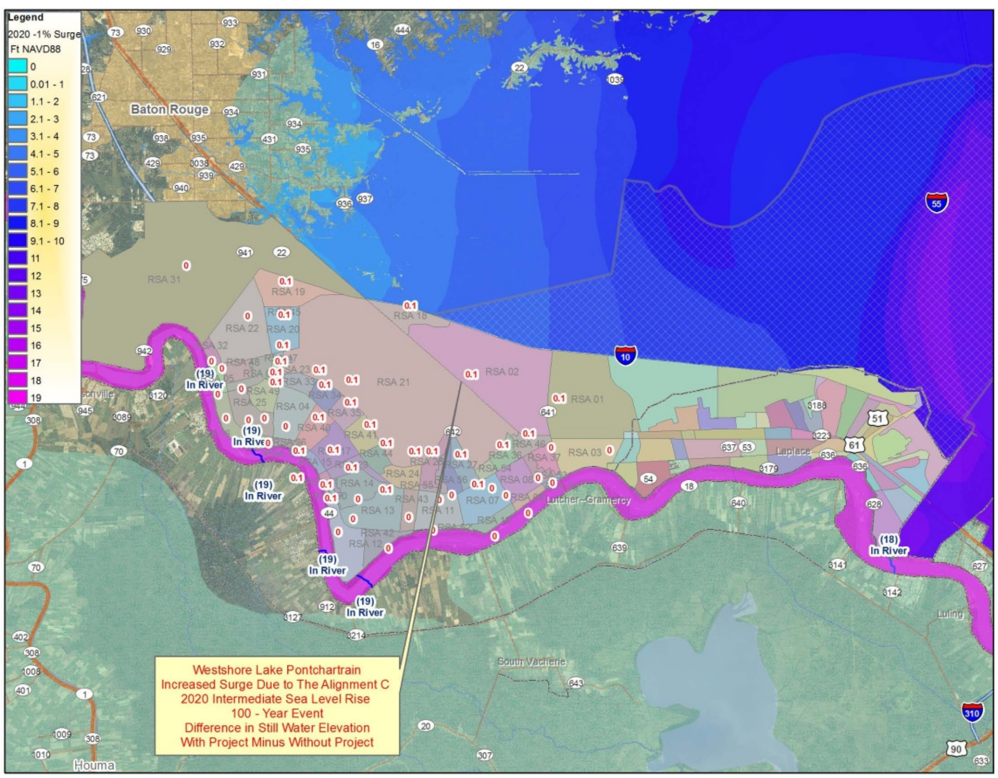


Figure 21 Difference Between With- and Without-Project for Year 2020-Intermediate SLR conditions

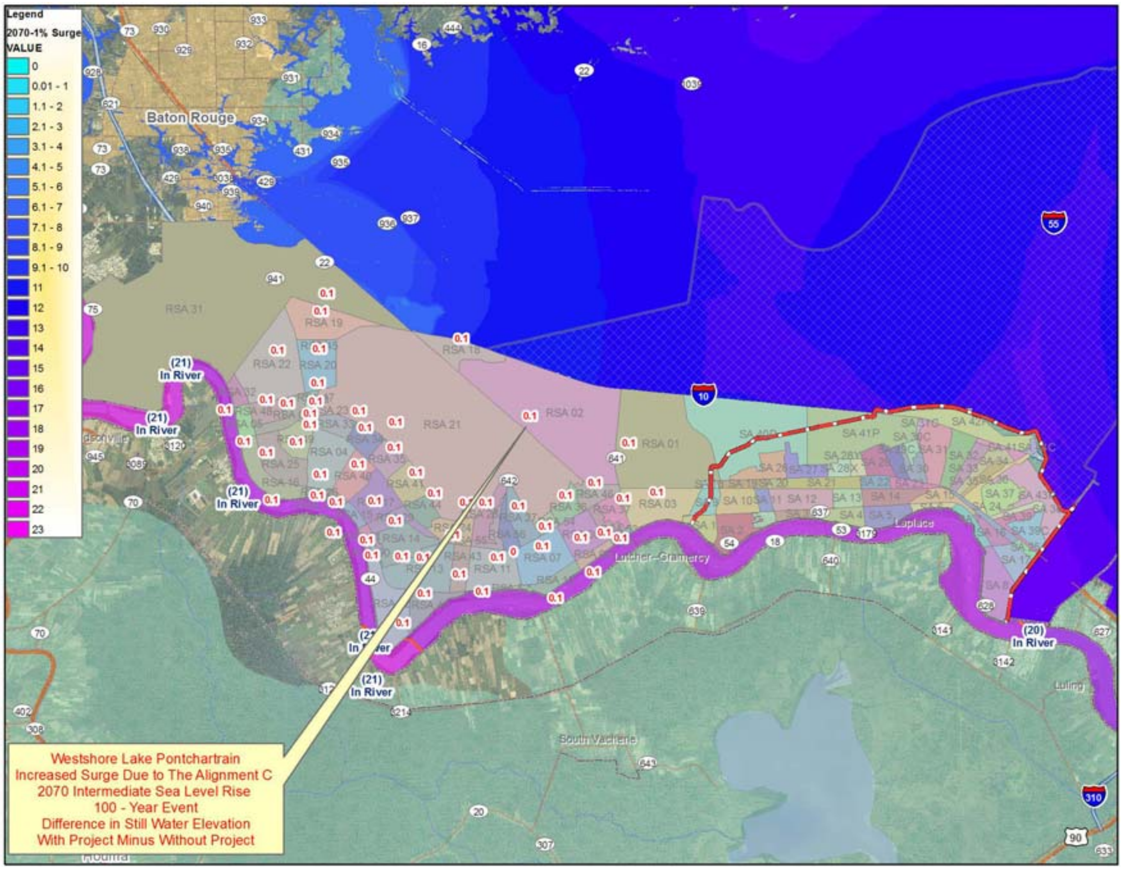


Figure 22 Difference Between With- and Without-Project for Year 2070-Intermediate SLR conditions

Geotechnical

Background and Furnished Information. As described at the beginning of this Appendix, the three levee alignments (A, C and D) were evaluated as part of the screening level effort. Levee cross section templates were developed based upon the proposed levee elevations and the geologic soil reaches. The analyses were based on Proposed Levee Elevations applicable to multiple alignments and for 11 soil reaches. Levee settlement estimates, recommended levee overbuild elevations and the number of projected levee lifts for Proposed Levee Elevations at Years 2020 and 2070 were also developed.

With Alignment C designated as the Recommended Plan, revised hydraulic design criteria were developed to meet increased intermediate sea level rise (SLR) elevations which resulted in increasing the Year 2020 and Year 2070 Proposed Levee Elevations to elevations greater than those used for screening analyses. A summary of the revised hydraulic design criteria for applicable soil reaches in Alignment C is shown in Table 9. These levels consider the previous-analyzed levee considering 1V:3H side slopes and no wave berms.

TABLE 9: REVISED ALIGNMENT C HYDRAULIC DESIGN CRITERIA

SOIL DESIGN REACH	ELEVATION (NAVD88)	
	REVISED Proposed Levee Elevation (2020)	REVISED Proposed Levee Elevation (2070)
1	8.5 / 10	16 / 17
2	10	17
4	11.5	17.5
7	11.5	17.5
8	14 / 15	18 / 19

Application of Soil Design Reaches. As noted in Table 9 above, five of the 11 soil design reaches have been identified as being applicable to Alignment C and the current hydraulic design levels. Selection of the application of these reaches were based on the original soil reach locations, the levee sections developed for the screening study, levee lift construction recommendations, original design elevations and current design elevations. These soil design reaches were then correlated to the 22 design sections previously developed (based upon the original 8 proposed levee elevation reaches, the 7 stillwater elevation reaches and the 10 geologic reaches) for this levee alignment and designated as C-1 through C-22.

Revised Analyses and Modified Recommendations. The revised hydraulic design criteria increased the Proposed Levee Elevations in Year 2020 and Year 2070; prior analyses were reviewed and additional analyses performed to assess where modifications to previous recommendations were necessary.

Considering limited changes in Proposed Levee Elevations previously analyzed and original computed factors of safety, it was recommended that the templates developed for soil reaches 1, 2, 4 and 7 be utilized to evaluate the new design grades. Soil Reach 8 for the Year 2020 Proposed Levee Elevation of El. +14 was also still applicable. The stability berm geometries and geosynthetic fabric lengths shown in previous analyses do not require further modifications and the recommended overbuild remains as 1.5 feet for these Year 2020 sections.

In Soil Reach 8 where the Year 2020 Proposed Levee Elevation is at El. +15, additional analyses were performed to evaluate the higher proposed grades. Increased stability berm dimensions and increased geosynthetic reinforcement fabric length were recommended in order to achieve the minimum required factors of safety. The revised dimensions and geometry should be applied to Alignment C for stations previously identified as C-1 through C-5.

As noted previously, stability analyses were not conducted for the Year 2070 Proposed Levee Elevations. Rather, sufficient gain-in-strength was assumed to occur over the life of the levee and as subsequent lifts are placed. In general, the change in grade between the new Year 2020 Proposed Levee Elevations and new Year 2070 Proposed Levee Elevations varies between 5 and 8 feet. The original change in grade averaged about 5 feet for the previous analyses. It should be noted that the 8-ft. grade change occurs where the highest factors of safety are computed for the Year 2020 Proposed Levee Elevations.

In general, the lift schedule described in the screening analysis should be followed, but the thickness of the lifts at Years 2030, 2045 and 2060 will be increased where the net grade change is increased.

Additional Considerations. As previously noted, the limited geotechnical data for this screening study required the development of assumed time-rate of settlement parameters for estimates of lift thickness and lift construction recommendations. However, even these assumptions would not address the stress history and time-rate away from the boring locations. Because Alignment C is located within a previously undeveloped area, additional lifts or increased lift thickness may be required.

Datum and Topography

As discussed in the **Datum and Topography** section of the **Screening Phase (Background) Information** section of this Appendix, all elevations used in the design were NAVD88-2004.65 datum. Any elevation data not in the NAVD88-2004.65 datum was adjusted prior to use.

Civil / Structural Design

The same set of standard details developed during the screening analysis to provide a schematic elevation view of the typical pump station T-Wall, Interstate T-Wall, Roadway/Railroad Floodgate T-Wall and Pipeline T-Wall were utilized for the Recommended Plan.

The revised design levee elevations for Alignment C were reduced from eight during the screening analysis to five in the recommended plan analysis. It was decided to maintain the same 22 levee design section limits in order to correlate and compare the screening and the recommended plan design sections. The design sections were adjusted based upon the revised geotechnical levee template, the proposed connector canal, and the proposed frontal ditch. A frontal ditch was added to the levee footprint to minimize wetland impacts. Typical Section drawings of the alignment can be found in Annex 3 of this Appendix.

As was done during the screening analysis, special attention was made to locate the right-of-way limits for the proposed levee sections to coincide with the existing rights-of-way from highways, pipelines, etc. to avoid remainder parcels that were nonfunctional to the original owner. This was accomplished since the growth of the levee template and frontal ditch was toward the unprotected side of the project and the highway and pipeline rights-of-way are on the protected side of the project.

Access Routes and Staging Areas. Potential access routes and staging areas have been identified during the feasibility-level design of the recommended plan alignment. Potential access roadways were identified by using aerial imagery to identify existing features along Alignment C. The aerial imagery utilized for the evaluation consisted of Google Earth imagery dated 05 March 2013 and 2004 Digital Orthophoto Quarter Quadrant (DOQQ) imagery available from the Louisiana State University Atlas Database website (http://atlas.lsu.edu). The Google Earth imagery was used to identify potential access points and the 2004 DOQQ was used to document the potential access points on the drawings.

Alignment C is primarily through wetland areas and adjacent to a major pipeline corridor. While the existing pipeline corridor has already been clear cut and mitigated, the corridor was excluded based upon construction loading being detrimental to existing pipelines.

The next approach was to identify potential access via direct access points, existing access points and new access points in that order of selection hierarchy. Since the proposed alignment crosses existing public roadways such as U.S. Highways 51 and 61, Louisiana Highway 44, Frenier Road and Oak Park Boulevard, direct access to the levee right-of-way could be obtained for construction. Existing access consisting of aggregate and dirt roads were identified along the alignment. There were twelve aggregate and dirt roads identified along the alignment with one being located within Louisiana State lands and the remainder being within what is assumed to be private lands. Actual ownership was not determined but assessed and evaluated from existing large tract ownership maps available to the project design team. Potential new access points consisted of potential new roadways through the wetlands to the levee right-of-way. Typically, these were potential new road extensions of existing aggregate or dirt roadways. There were three potential new access road (extensions) identified from the aerial imagery. This process identified twenty potential access roads for construction of the project.

The twenty potential access roads identified are in excess of what is needed for construction of an 18-mile levee. Some of the potential access roads were selected to form a recommended list of potential access roads based upon the selection criteria described above and the potential haul distances between the access points. The recommended list includes three direct accesses, six existing accesses and three new accesses for a total of twelve access points. A one-acre staging area was allocated for all twelve access points for haul ticket collection and truck wash-down. During the P.E.D. phase of the project, these routes and staging areas will be finalized.

Borrow Sources. Borrow material for this project would come from the Bonnet Carré Spillway. The project design team has reviewed the potential for obtaining all of the required borrow for this project from the spillway. They believe the spillway has adequate clay material available for this project. An alternative borrow pit investigation has not been conducted at this time.

Quantities. Quantities were computed for clearing and grubbing, geotextile, earthwork, aggregate roadway, turf establishment, T-Walls, drainage gates, roadway gates, railroad gates, pump stations and pipeline relocations in the same manner as during the screening evaluation. The quantities for clearing and grubbing, geotextile, earthwork and turf establishment increased based upon the revised levee elevation and template changes. New quantities for access roads and staging areas were computed including clearing, grubbing and aggregate roadway. The revised and new quantities have been included in the MII Cost Estimate.

Relocations

The assumption for Alignment C was that a pipeline floodwall would be required wherever a pipeline crossed the levee footprint. The pipeline would cross through a cutoff wall under the pipeline floodwall. It was decided that the existing carrier line would remain in operation while a bypass line would be constructed through a sleeve in the T-wall cutoff piles. When the bypass would be completed and in place, the switch over-tie in with the existing line then would follow along with the removal of the abandoned pipeline. These assumptions are consistent with the screening level assumptions. For the recommended plan, it was assumed the pipeline would be relocated for the full right-of-way width of the proposed levee to accommodate the proposed protected side canal and the unprotected side ditch. A pipeline relocation length of 600 feet was used versus the widest right-of-way of 541 feet. The costs for relocations have been included in the MII Cost Estimate.

Cost Estimates

The project cost estimate was developed in the MCACES 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 increases above costs. This philosophy was used 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 estimate is structured to reflect the project construction tasks performed. The estimate has been subdivided by USACE feature codes and by the 22 levee design reaches (levee and floodwalls), 36 pipeline relocations and 4 pump stations. The cost estimate included consideration of labor rates, materials, equipment, fuel, crew production, relocation, mob/demobilization, field and office overhead, taxes, bonds, engineering, contingencies and escalation. A construction schedule was developed to provide 100-year protection from project design year of 2020 through the project life span of 50 years to Year 2070. Annex 1 to this Appendix contains the Cost Engineering Report, the MII Cost Estimate, the Project Construction Schedule and the Summary Cost Schedule Risk Analysis (CSRA). Annex 2 to this Appendix contains the Detailed CSRA.

References

ER 1110-2-1150, Engineering and Design for Civil Works Projects, 31 August 1999.

ER 1110-1-8159, Engineering and Design, DrChecks, 10 May 2001.

ER 1110-1-12, Engineering and Design, Quality Management, 21 July 2006.

EC 1165-2-209, Civil Works Review Policy, 31 January 2010.

EC 1165-2-214, Civil Works Review Policy, 15 December 2012.

ER 1100-2-8162, Incorporating Sea Level Change in Civil Works Programs, 31 December 2013.

Hurricane and Storm Damage Reduction System (HSDDRS), Design Guidelines, USACE, New Orleans District, June 2008.

CECW-CE, Engineering and Construction Bulletin, No. 2004-13, Issued 30 Aug 2004.

EM 1110-2-6056, Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums, 31 December 2010.

USACE New Orleans District Guide for Minimum Survey Standards for Performing Topographic, Hydrographic, and Static GPS Control Surveys (Edition 2.1).

Louisiana State Plane Coordinate System South Zone (1702) using North American Datum of 1983 (NAD83) for horizontal datum.

North American Vertical Datum of 1988 Epoch 2004.65 (NAVD88-2004.65) for vertical datum.

ER 1110-1-12, Engineering and Design, Quality Management, dated September 30, 2005.

JPM-OS method (White Paper on Estimating Hurricane Inundation Probabilities for Storm Selection and Statistics Reference (dated 10 June 2007)

Resio (2007)

Smith, 2006, pers. comm.

SCREENING PHASE (BACKGROUND) INFORMATION

The following information below was used in the plan formulation process to identify the tentatively selected plan (TSP) described in the Draft Report. The Draft Report was presented to the public on August 23, 2013. The information below is the same information presented in the Draft Report and does not reflect changes to the TSP recommendation that occurred after the publication of the Draft Report. The information is included to inform the reader of the planning process as it had been conducted up to publication of the Draft Report. After the release of the Draft Report, the team refined the design of the TSP with additional engineering and environmental investigations. This information is presented in the sections above. Based on feasibility level of design and based on comments received following publication of the Draft Report, portions of the TSP was modified. For the full details of the additional planning efforts a brief description of those modifications please see section 3.9 and section of the main report.

Figure 23 displays the 3 alternative alignments that were presented to the public in the August 2013 Draft Report

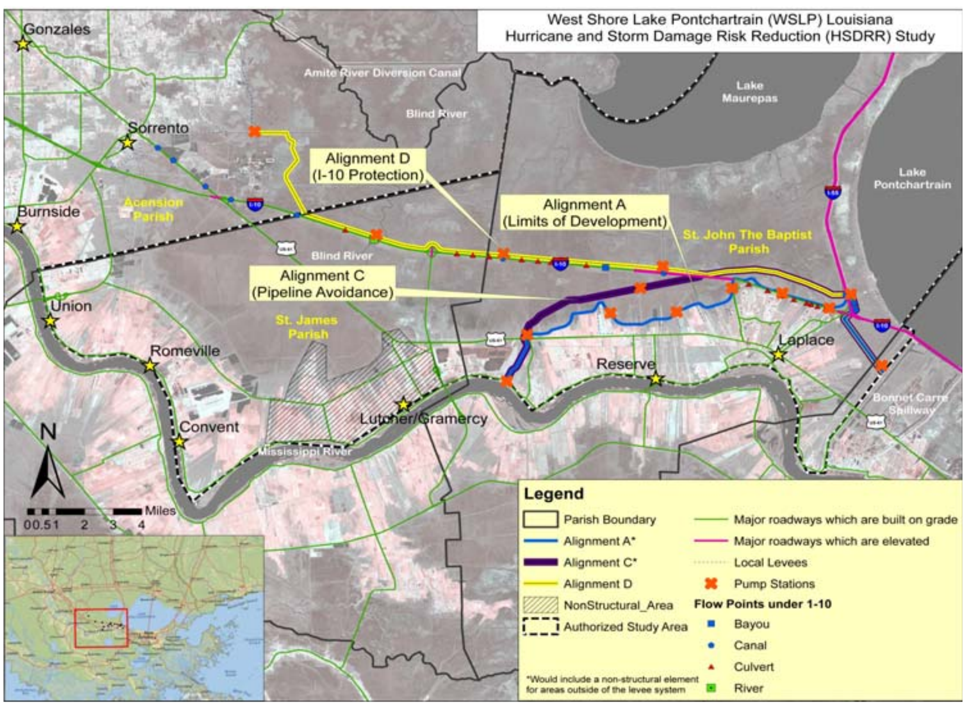


Figure 23: The Three Alternative Alignments

Alternative A

Alternative A starts at the Upper Guide Levee of the Bonnet Carre Spillway in St. Charles Parish, LA (north of the transmission and pipeline corridors), extends west around the I-10/I-55 interstate interchange and ends at the Mississippi River Levee just west of the Hope Canal in St. John the Baptist Parish, LA, a distance of 20.41 miles. The earthen levee generally follows the wet/dry interface. The following information is based on modeling for a 100-year level of risk reduction in the Baseline Year of 2020 for a period of evaluation of 50 years.

The top of levee elevation (net elevation) for this alignment ranges from El. 13.5 NAVD88 on the eastern reaches of the levee near the Bonnet Carre Spillway and gradually tapering to El. 7.0 NAVD88 as the levee moves west across the project .

Floodwalls

Ten Floodwalls (T-type walls), comprising a total of 4,774 linear feet, range from 10 ft. to 19 ft. in height; the top of wall design elevation is El. 17.0 NAVD88. The floodwalls, for the most part, are located where the alignment runs under I-10 and the I-10/I-55 interchange.

Floodgates

Nine Floodgates, comprising a total of 1,218 linear feet, range from 10 ft. to 19 ft. in height; the top of gate design elevation is El. 17.0 NAVD88. The floodgates, for the most part, are located along the alignment, usually where canals and roads are. Additionally, two 25-ft. wide railroad swing gates (each 11 ft. high) are included for those instances where the levee crosses the railroad.

Drainage Structures

Gravity Drainage Structures (with sluice gates), comprising a total of 240 linear feet, range from 20 ft. to 29 ft. in width. These are located near proposed pumping stations.

Pumping Stations

There are 8 pumping stations located along the alignment. The different sizes (which assumes there is no storage capacity available) are as follows:

2 at 240 cfs each

1 at 328 cfs

1 at 400 cfs

2 at 460 cfs each

1 at 656 cfs

1 at 787 cfs

Pumping stations are located at the various canals that cross the alignment, such as the Hope, Mississippi Bayou, Reserve Relief, Ridgefield, Vicknair and Montz Canals. It is generally expected that the gates would be closed, and the pumps would be operated during tropical/hurricane storm surge events. Pumping would continue until the water level returns to existing natural water level conditions (currently estimated to be El. 2.0 NGVD), at which time the operation of the pumps would be discontinued and the gates would be opened.

Pipeline Relocations

There are numerous pipeline relocations involved in this alignment. The diameters of the various pipelines are as follows:

6 in. and less	18 pipelines
12 in. and less (but greater than 6 in.)	40 pipelines
24 in. and less (but greater than 18 in.)	11 pipelines
Greater than 24 in.	1 pipeline

Alternative D

Alternative D starts at the Upper Guide Levee of the Bonnet Carre Spillway in St. Charles Parish, LA (north of the transmission and pipeline corridors), extends west around the I-10/I-55 interstate interchange, continues west along I-10 and ends at the Marvin Braud Pumping Station, in the vicinity of Sorrento (within the McElroy Swamp) in Ascension Parish, LA, a distance of 28.28 miles. The following information is based on modeling for a 100-year level of risk reduction in the Baseline Year of 2020 for a period of evaluation of 50 years and is subject to change based on further evaluation in future phases of the project. The top of levee elevation (net elevation) for this alignment ranges from El. 13.5 NAVD88 on the eastern reaches of the levee near the Bonnet Carre Spillway and gradually tapering to El. 8.0 NAVD88 as the levee moves west across the project area.

Floodwalls

Six Floodwalls (T-type walls), comprising a total of 4,011 linear feet, range from 15 ft. to 19 ft. in height; the top of wall design elevation is El. 17.0 NAVD88. The floodwalls, for the most part, are located where the alignment runs under I-10 and the I-10/I-55 interchange.

Floodgates

Three Floodgates, comprising a total of 306 linear feet, range from 15 ft. to 19 ft. in height; the top of gate design elevation is El. 17.0 NAVD88. The floodgates, for the most part, are located along the alignment, usually where canals and roads are.

Drainage Structures

Gravity Drainage Structures (with sluice gates), comprising a total of 396 linear feet, range from 20 ft. to 29 ft. in width. These are located near proposed pumping stations. For the Bayou Conway area, the required channel size is 24 ft. wide x 12 ft. deep (to convey 1,100 cfs of flow). For the Blind River area, the required channel size is 40 ft. wide x 20 ft. deep (to convey 4,500 cfs of flow).

Pumping Stations

There are 6 pumping stations located along the alignment. The different sizes (which assume there is no storage capacity available) are as follows:

1 at 200 cfs

1 at 400 cfs

1 at 450 cfs

2 at 1,100 cfs each (this includes the Bayou Conway area)

1 at 4,500 cfs (this is for the Blind River area)

Pumping stations are located at the various canals that cross the alignment, such as the Montz, Reserve Relief and Ridgefield Canals, as well as a local canal near approx. Baseline Station 951+00 and the Bayou Conway and Blind River areas. It is generally expected that the gates would be closed, and the pumps would be operated during tropical/hurricane storm surge events. Pumping would continue until the water level returns to existing natural water level conditions (currently estimated to be El. 2.0), at which time the operation of the pumps would be discontinued and the gates would be opened.

Pipeline Relocations

There are numerous pipeline relocations involved in this alignment. The diameters of the various pipelines are as follows:

6 in. and less	7 pipelines
12 in. and less (but greater than 6 in.)	6 pipelines
24 in. and less (but greater than 18 in.)	1 pipeline

There are at least two instances where the pipeline would cross through the floodwall (at approx. Baseline Station 1382+00 and at approx. Baseline Station 1404+00).

Culverts

There are 6 culverts (in addition to the culverts that exist under I-10) that facilitate tidal exchange of water with the wetlands.

Hydraulics and Hydrology

Interior Drainage

The interior drainage analysis for the feasibility study was broken down into two stages:

1) Determine the rough-order-of-magnitude (ROM) capacities of gravity drainage structures and pumps recommended to prevent project induced flooding for each of the proposed alignments (A, C and D).

2) For the tentatively selected plan (TSP), determine the capacities of gravity drainage structures and pumps using a detailed rainfall-runoff analysis.

For the ROM phase of the analysis, pump and gravity drainage recommendations were determined using an XP-SWMM model completed during the reconnaissance phase of the study for Alignments A and C. Figure 24 depicts the storage basin layout for used in the model. These basins correspond to the sizes and capacities listed in Table 10. Alignment D covers the area of Alignment C in addition to the drainage basins of the Blind River and Bayou Conway. Structures and pumps were sized for Blind and Conway using the HEC-HMS and HEC-RAS modeling suite. The recommendations are also listed in Table 10. All design values are based on a 10-yr, 24-hr rainfall.

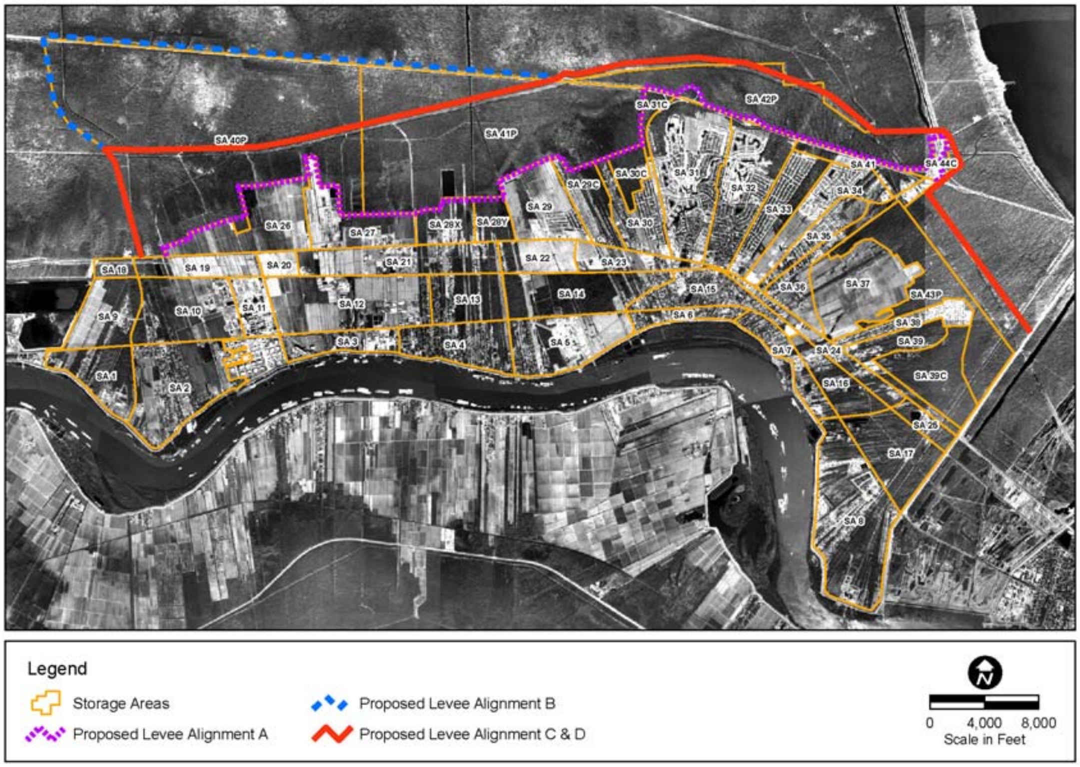


Figure 24: Storage Basin Layout

Table 10: ROM Determinations

Item / Location:	Alignment A	Alignment C and D	Blind River and Bayou Conway (Alignment D only)
Gravity Drain, SA-40P	1 RCBC*, 6' High by 20' Wide	1 RCBC, 6' High by 20' Wide	
Gravity Drain, SA-41P	2 RCBC's, 6' High by 20' Wide	2 RCBC's, 6' High by 20' Wide	
Gravity Drain, SA-42P	2 RCBC's, 6' High by 18' Wide	2 RCBC's, 6' High by 18' Wide	
Gravity Drain, SA-43P	2 RCBC's, 6' High by 18' Wide	2 RCBC's, 6' High by 18' Wide	
Pump Station, SA-40P	480 cfs	450 cfs	
Pump Station, SA-41P	1180 cfs	400 cfs	
Pump Station, SA-42P	920 cfs	200 cfs	
Pump Station, SA-43P	985 cfs	1100 cfs	
Gravity Drain, Blind River			40ft. wide, 20ft. deep rectangular cross section

Gravity Drain, Bayou Conway			24ft. wide, 12 ft. deep rectangular cross section is required
Pump Station, Blind River			1100 cfs
Pump Station, Bayou Conway			4500 cfs

*RCBC - Reinforced Concrete Box Culvert

Tropical/Hurricane Storm Surge Modeling

State-of-the-Art coastal ocean hydrodynamic analysis methods were used to determine the storm surge and wave results. The modeling system for this study was established by fine-tuning existing models used previously for the Joint Storm Surge (JSS) Analysis in Southern Louisiana for the Louisiana Coastal Protection and Restoration (LACPR) project, as well as the recent flood insurance rate map modernization study conducted by the Federal Emergency Management Agency (FEMA) (USACE 2008a; USACE 2007).

The data gathered from Advanced Circulation (ADCIRC) and the Steady State Spectral Wave (STWAVE) modeling were used to generate surge and wave return values ranging from the 50 year return to the 2000 year return in 50 year increments. A set of 152 hurricane condition storm events were used to develop an existing (2011) condition and future conditions for a 2020 intermediate relative sea level rise (SLR) and 2070 low, intermediate, and high SLR as well as alternative alignments intermediate SLR. The Joint Probability Method, with Optimum Sampling (JPM-OS) was applied for each data set to develop stage frequencies. The resulting levee design heights for the screening level effort for each alignment and for each condition (2011, 2020 and 2070) are shown on the following maps (Figures 25 through 33). It should be noted that, for Figures 28 through 33, the notation of "Considering Intermediate Sea Level Rise" on each of these maps refers to Intermediate Relative Sea Level Rise.

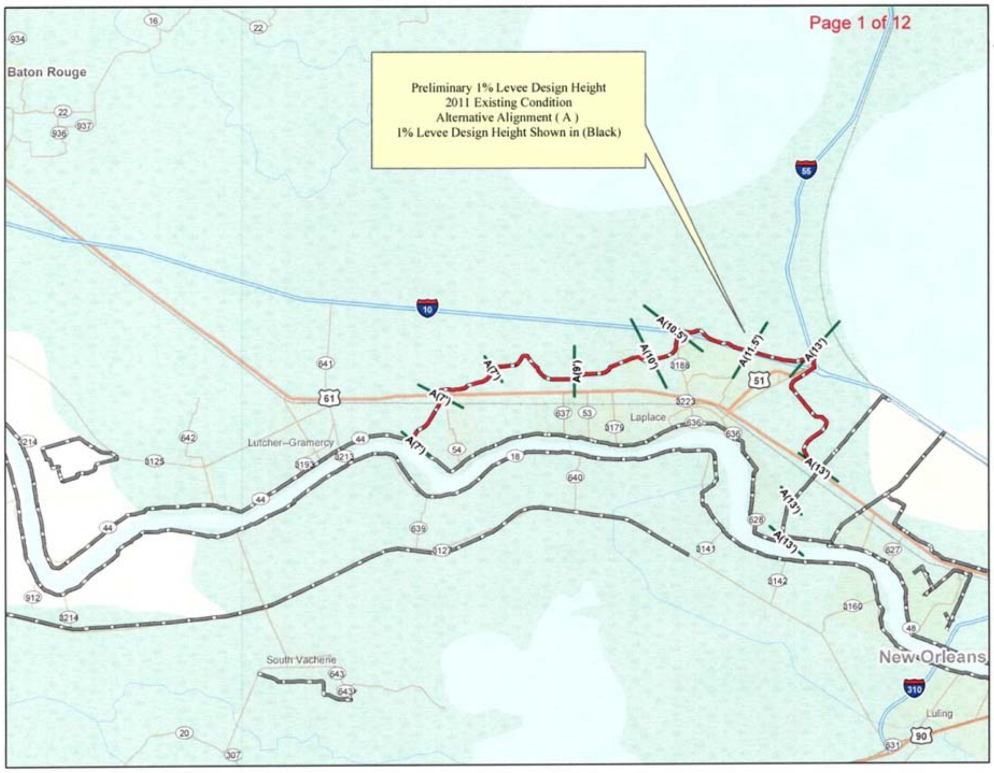


Figure 25: Levee Design Height Existing Conditions Alignment A

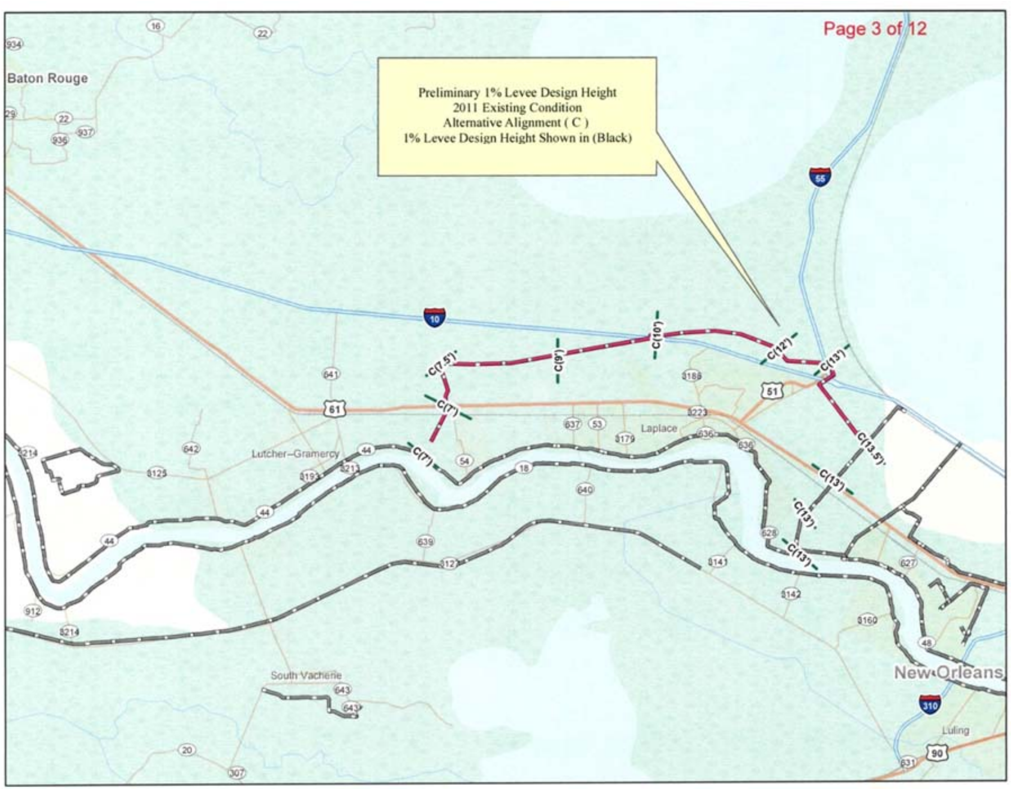


Figure 26: Levee Design Height Existing Conditions Alignment C

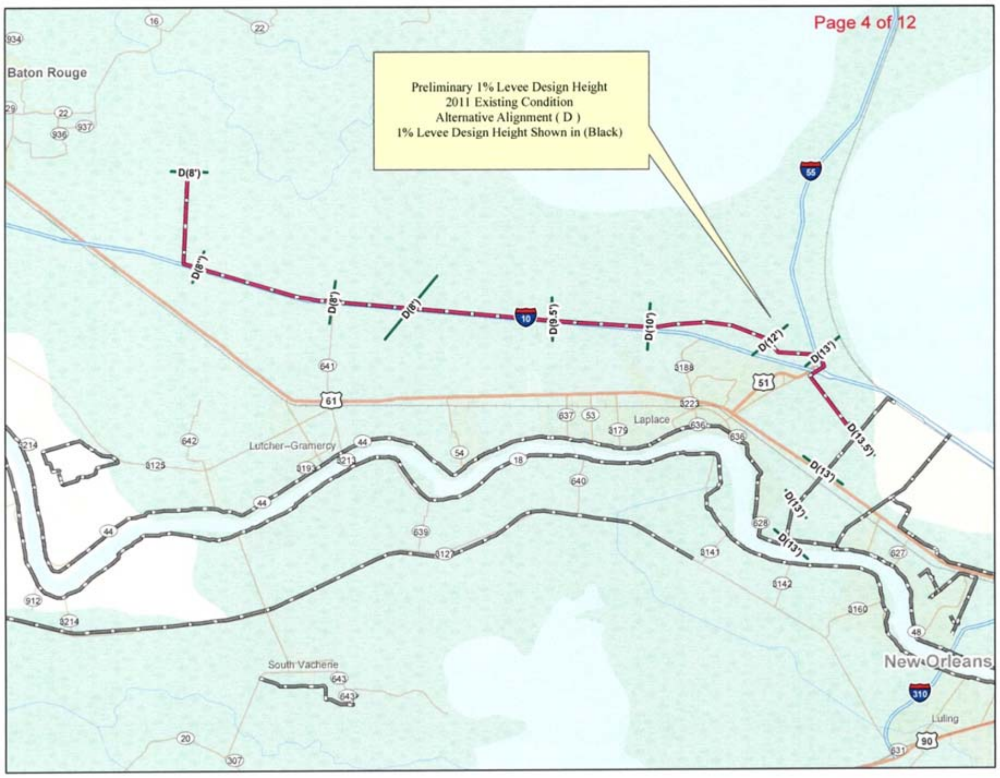


Figure 27: Levee Design Height Existing Conditions Alignment D

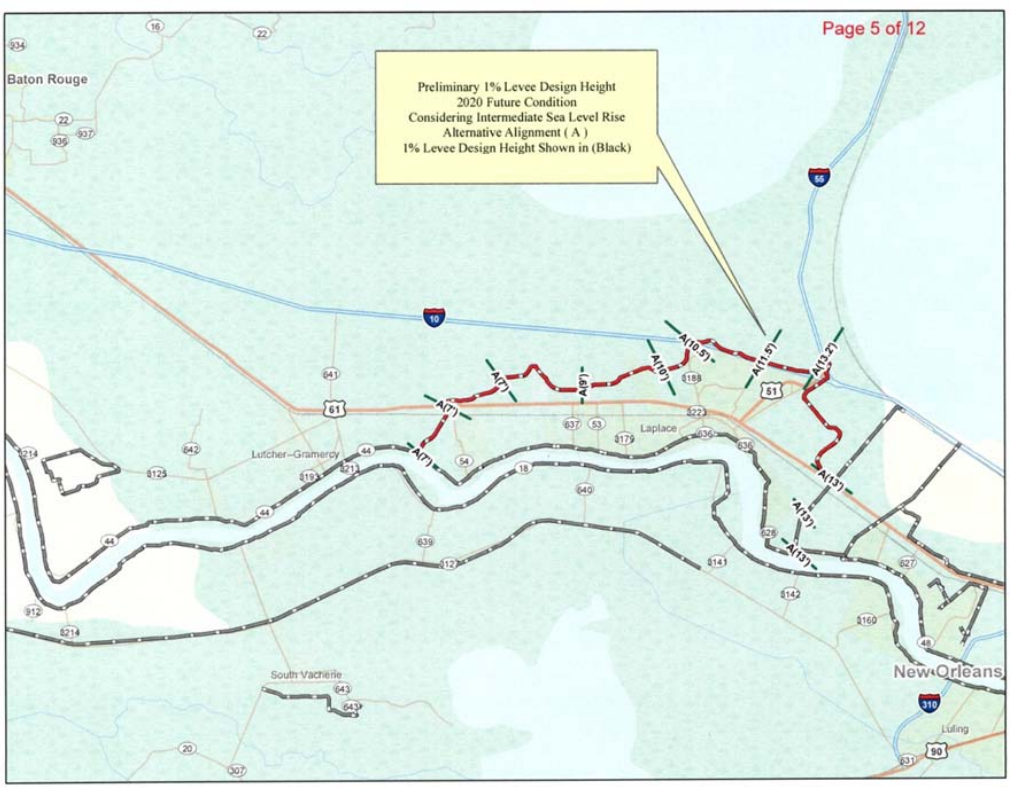


Figure 28: Levee Design Height 2020 Future Condition Alignment A

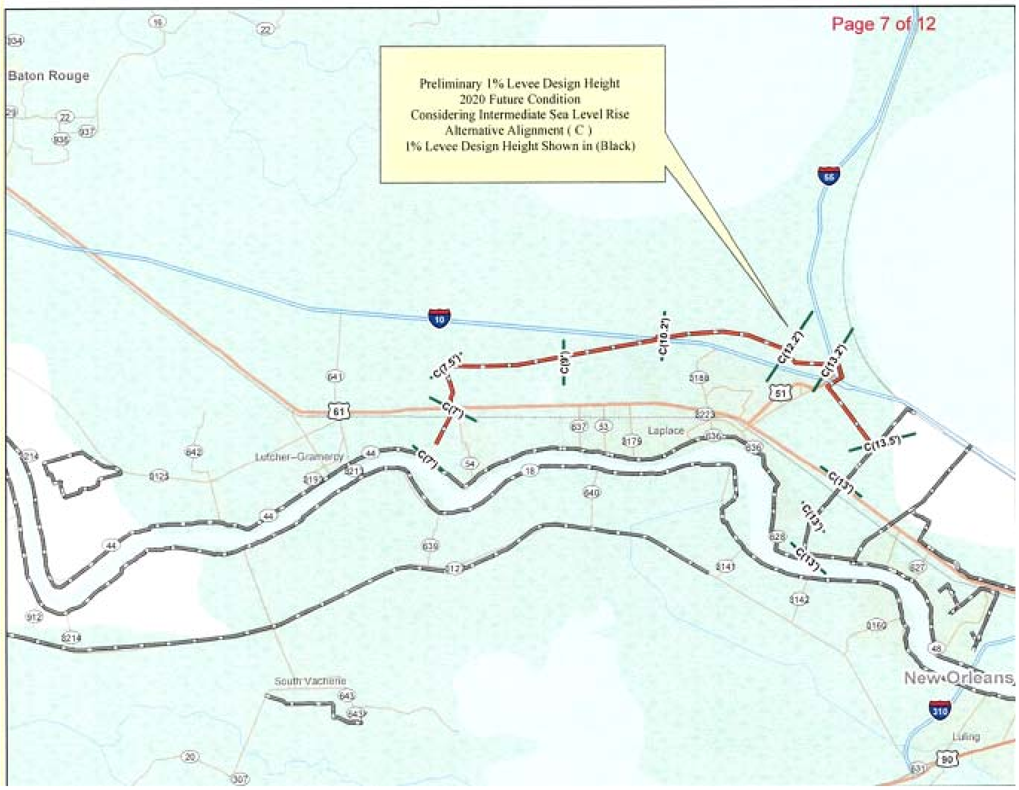


Figure 29: Levee Design Height 2020 Future Condition Alignment C

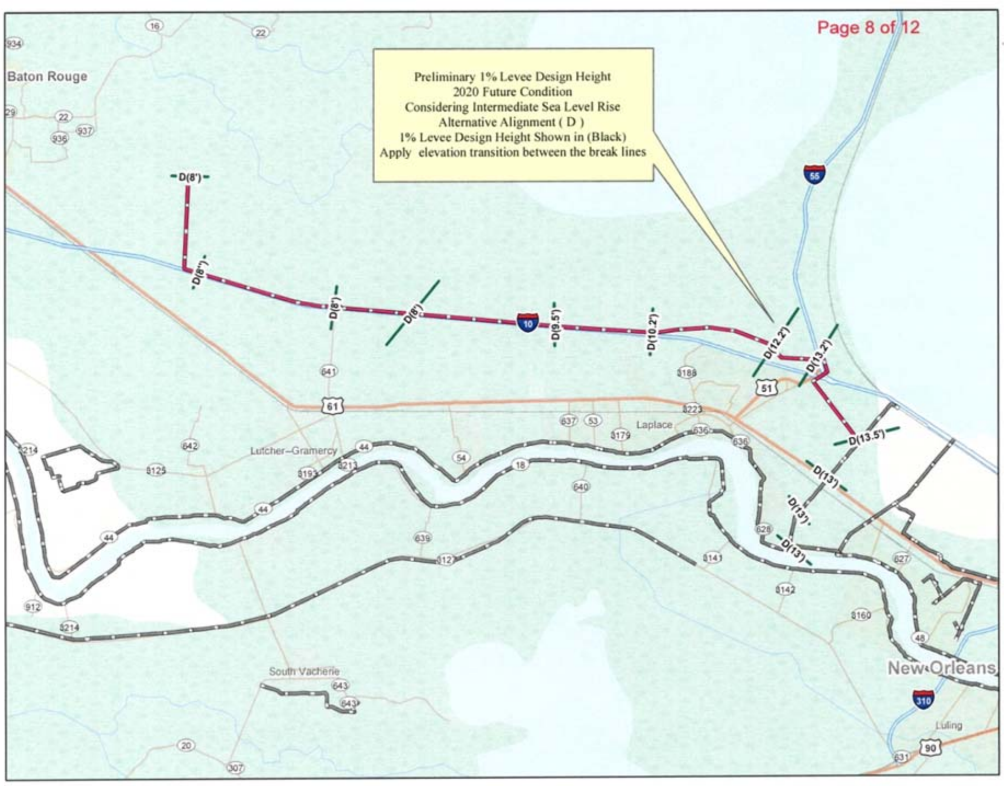


Figure 30: Levee Design Height 2020 Future Condition Alignment D

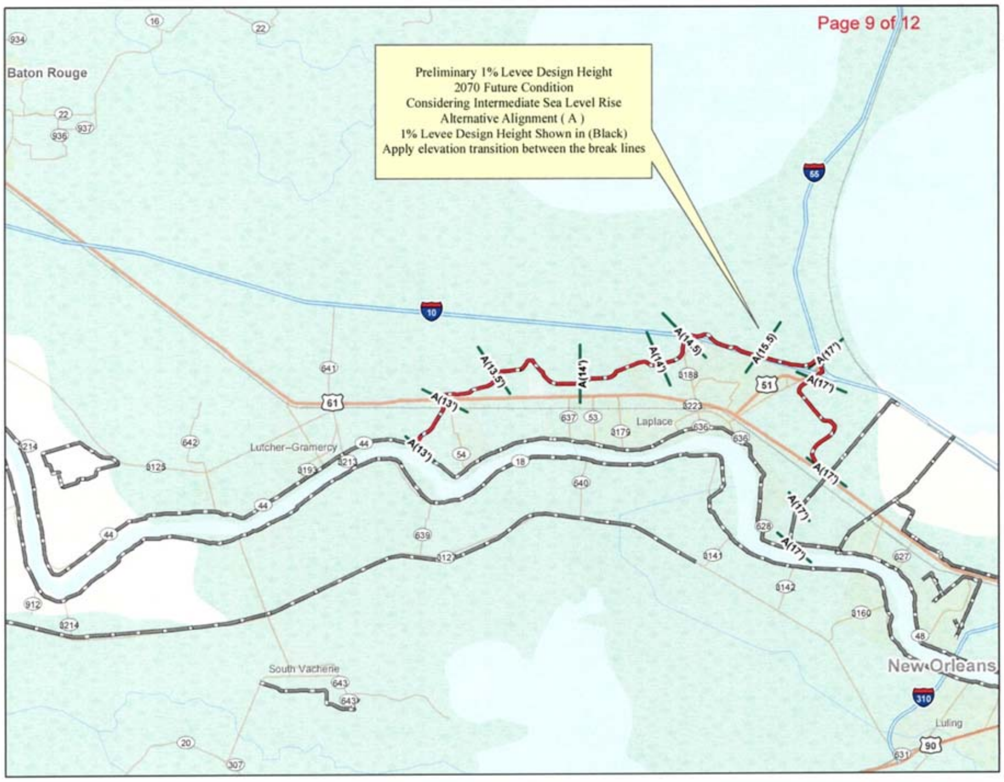


Figure 31: Levee Design Height 2070 Future Condition Alignment A

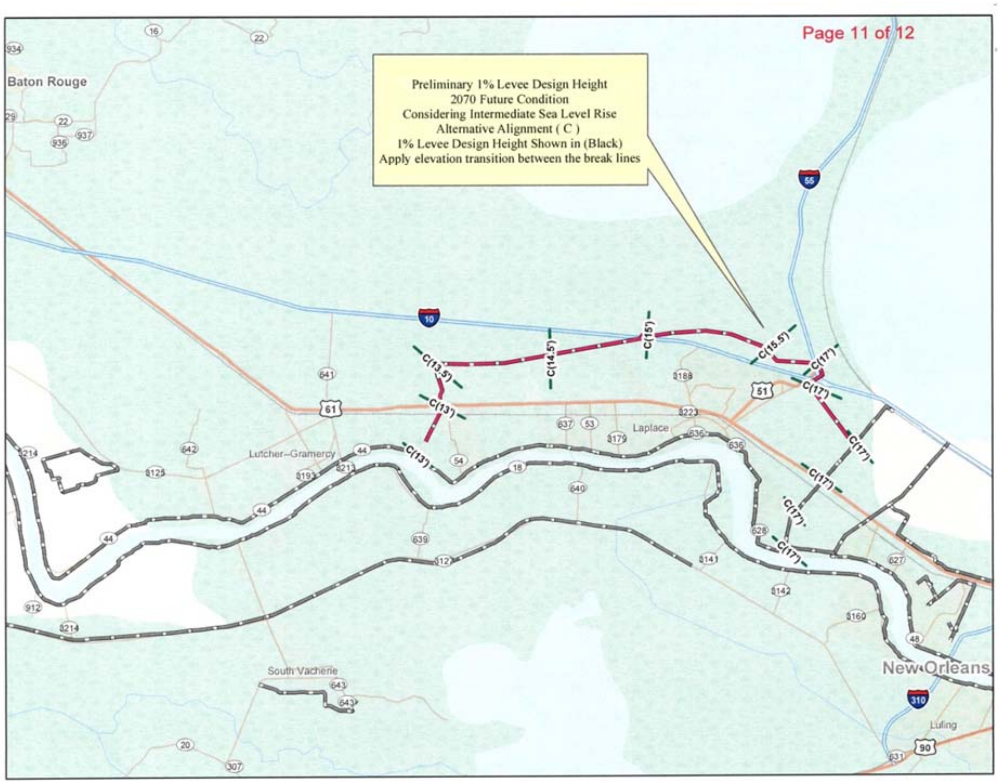


Figure 32: Levee Design Height 2070 Future Condition Alignment C

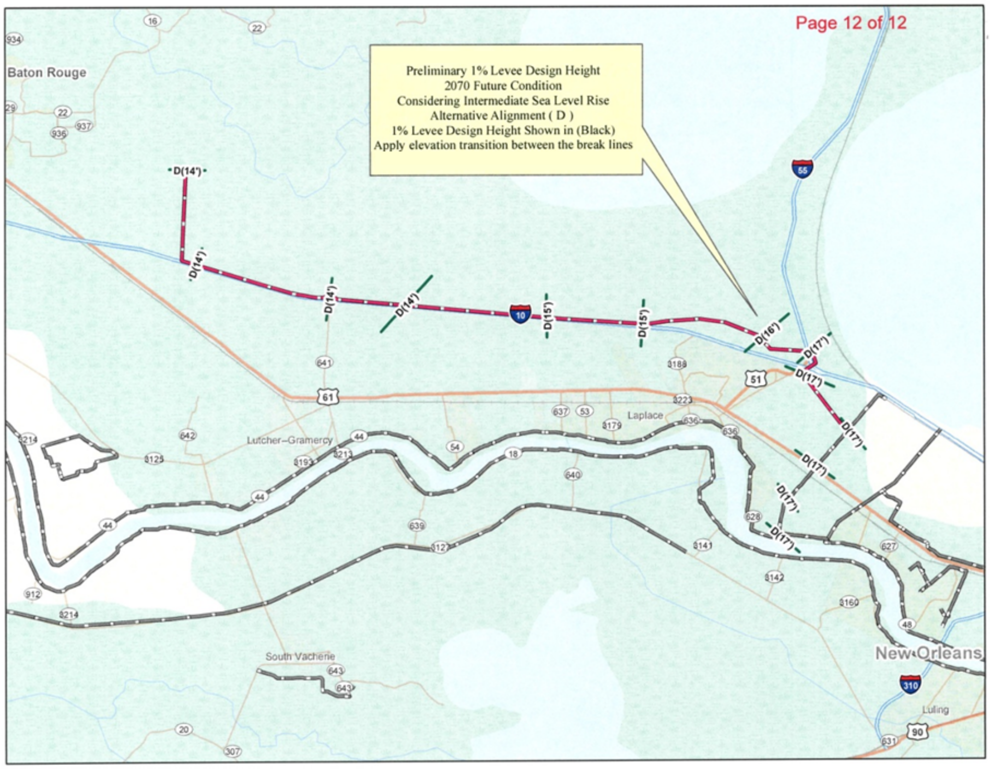


Figure 33: Levee Design Height 2070 Future Condition Alignment D

Potential Sea Level conditions are represented in the modeling system by application of a relative Sea Level Rise (SLR) that is consistent with USACE ER 1100-2-8162 (31 December 2013). Subsidence levels predicted in the study area were incorporated in the ADCIRC initial water level parameter to capture the combined effects of subsidence and local SLR into a single SLR value. For the Year 2020 and Year 2070 simulations, unique SLR values were added to the 2011 initial water surface elevations (WSE) to determine the initial WSE appropriate for each year and SLR rate. In addition to accounting for SLR of future conditions, the Year 2070 scenarios accounted for potential degradation of vegetation in landscapes. SLR changes (as well as salinity intrusion) can cause an associated vegetation degradation and / or loss (this was considered in the ADCIRC modeling). Since these are slow-moving processes, forecasts of 50 years in the future were used, with intermediate SLR conditions. See Figure 34 for SLR estimates for Years 2011 through 2080.

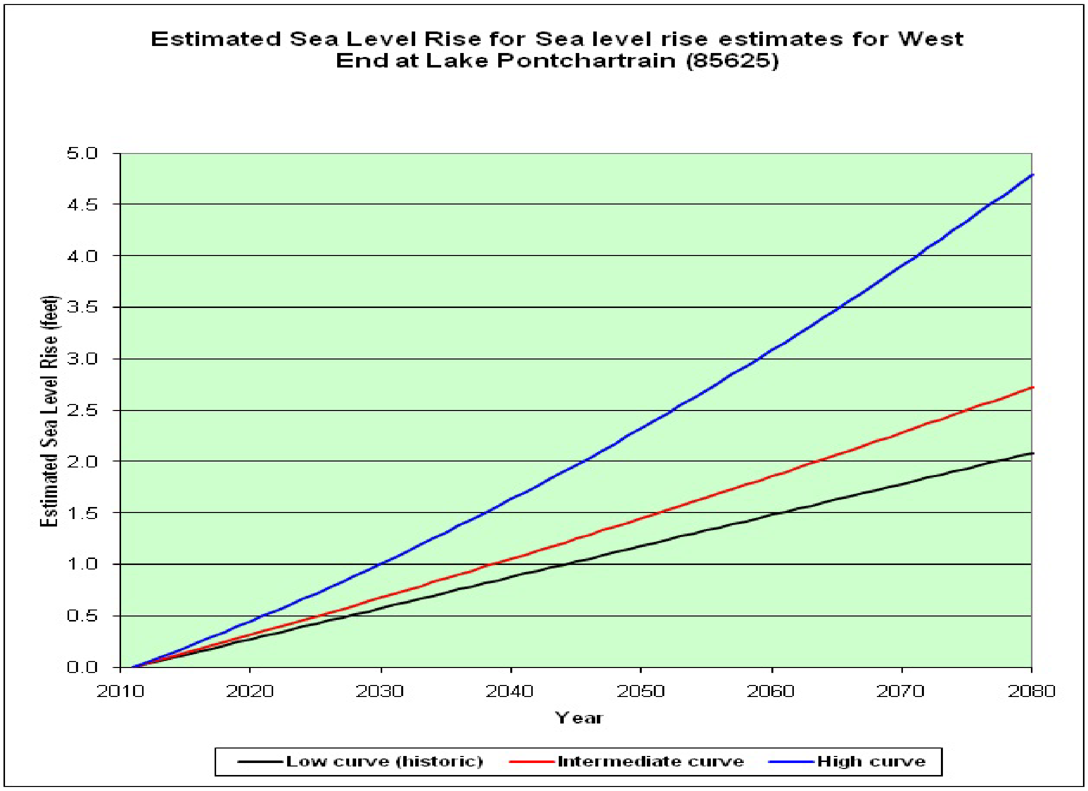


Figure 34: Estimated Sea Level Rise (SLR) for Years 2011 through 2070

Water Quality

This water resource is significant because of the Clean Water Act, as amended, the Pollution Prevention Act, the Safe Drinking Water Act, and the Water Resources Planning Act, regulations which provide for the protection of U.S. waters for the purposes of drinking, recreation, and wildlife. It also provides for the purposes of restoring and maintaining the chemical, physical, and biological integrity of the Nation's waters. Study area water quality is influenced by basin elevations, surface water budget, land cover and use, coastal and geological processes, and regional weather. The study area is in the southwestern portion of a basin consisting of uplands to the north and estuary to the south, with increasing estuary salinity eastward. The estuary has experienced hydromodification via the construction of canals and embankments. Historical study area water quality is depicted in several references which include the review of data from basin tributaries and estuary lakes and passes. Garrison (1999) provides a statistical summary of general parameters, major ions, nutrients, trace metals, and organic compounds for water quality data collected in Lake Maurepas between 1943 and 1995. Overall, the summary suggests the lake is freshwater, oligotrophic, and does not contain elevated contaminant levels. To determine the most prevalent water quality issues present in the study area, historical Section 305(b) lists were reviewed to determine the most significant causes and sources of subsegment impairment. The most current (2012) 303(d) list for the study area is depicted in Table 11. Ordered by decreasing frequency cited, suspected causes of impairment include non-native aquatic plants, low dissolved oxygen, mercury, elevated turbidity, and fecal coliform, while suspected sources of impairment include wetland habitat modification, introduction of non-native organisms, atmospheric deposition, unknown sources, on-site treatment systems, natural sources, and agriculture.

Table 11: Study Area 2013 303(d) List

Subsegment	Impaired Use for Suspected Cause	Suspected Cause of Impairment	Suspected Source of Impairment	IR Category	TMDL Priority
040401	FWP	Dissolved Oxygen	Wetland Habitat Modification	IRC 5	L
		Mercury	Atmospheric Deposition	IRC 4a	
			Source Unknown	IRC 4a	
		Non-Native Aquatic Plants	Introduction of Non-native Organisms	IRC 4b	
		Turbidity	Wetland Habitat Modification	IRC 4a	
	ONR	Turbidity	Wetland Habitat Modification	IRC 4a	
	PCR	Water Temperature	Natural Sources	IRC 5	L
			Wetland Habitat Modification	IRC 5	L
040403	FWP	Dissolved Oxygen	Agriculture	IRC 5	L
			Wetland Habitat Modification	IRC 5	L
		Mercury	Atmospheric Deposition	IRC 4a	
				IRC 5	L
			Source Unknown	IRC 4a	
				IRC 5	L
		Non-Native Aquatic Plants	Introduction of Non-native Organisms	IRC 4b	
040404	FWP	Dissolved Oxygen	On-site Treatment Systems	IRC 5	L
		Non-Native Aquatic Plants	Introduction of Non-native Organisms	IRC 4b	
	PCR	Fecal Coliform	On-site Treatment Systems	IRC 5	H
040602	FWP	Non-Native Aquatic Plants	Introduction of Non-native Organisms	IRC 4b	

Both historical 305(b) and current 303(d) lists suggest primary study area water quality problems relate to hypoxia. As a further to this suggestion, in 2011 a TMDL report was prepared for the lower Amite River watershed (located just north of subsegments partially included in the study area) to address organic enrichment and low dissolved oxygen. Long-term water quality monitoring in the study area was conducted by the Louisiana Department of Environmental Quality (LDEQ). Water quality trends in the study area based on this water quality assessment would be expected to continue. In particular, low dissolved oxygen conditions in the Maurepas Swamps and increasing marine influence in the northern study area are expected to persist, while the historically most common suspected causes of impairment within the study area would continue to generate water quality problems in competition with management efforts to eliminate impairments. With project water quality is addressed in the EIS.

Climatology

Temperature

Records of temperature are available from "Climatological Data" for Louisiana, published by the National Climatic Data Center. The study areas can be described by using the normal temperature data observed at the Hammond, and Donaldsonville stations. These stations are shown in Table one below with the monthly and annual mean normals which are based on the period of 1991-2011. The average annual mean normal temperature is 59.4^{o}F, with monthly mean temperature normal varying from 81.9^{o}F in July to 48.7^{o}F in December.

Precipitation

Records of precipitation are available from “Climatological Data” for Louisiana, published by the National Climatic Data Center. Two stations in the Louisiana study have been used to show the rainfall data for the areas of Donaldsonville and Ponchatoula/Hammond. Both stations have normal precipitation records which are based on the period of 1991-2011. The average annual normal rainfall of the two stations is 58.14 inches. The wettest normal month is June with a monthly average of 6.48 inches. October is the driest normal month averaging 4.11 inches and Donaldsonville has the greatest day with 24.49 inches of rain falling in June 2001.

Geotechnical

Engineering included the preparation of earthwork stability templates, settlement and lift schedule predictions, preparation of schematic alignment layouts, schematic pump station layouts, and scoping level project cost estimates for the elimination of alternatives to determine a tentatively selected plan. Schematic earthwork templates and settlement and lift schedule predictions were also performed.

The process to complete the scoping level engineering started with the geotechnical evaluation of the different alignments. The geotechnical evaluation consisted of reviewing existing soil boring data, preparation of earthwork stability templates, T-Wall analysis, settlement predictions, additional lifts, and secondary settlement predictions.

Geotechnical data was used to develop soil design parameters for the proposed alignments. At the time of the geotechnical report, four alternative alignments (reduced to three in August 2012) were being considered for the project. These alignments are denoted as Alignments A, C and D. Eighty three borings have been utilized for this screening study, with 23 geologic reaches and eleven soil reaches being developed. The alignments and reaches, as well as the developed soil design parameters, are shown in tabular and graphical form in the Draft Geotechnical Report Appendix I from March 2012.

Of the 83 borings furnished, 32 borings are located on Alignment A from its western limit at Hope Canal to its intersection with I-10 west of Highway 3188. These 32 borings comprise Soil Reaches 1 through 5. An additional 17 borings are located on the portion of Alignment A which coincides with I-10 from Highway 3188 to just west of the intersection with I-55 and comprise Soil Reaches 6 and 7. Thus, over half of the available data and selected reaches coincide with Alignment A.

The proposed alignments from the I-55 interchange to the St. Charles Parish line vary among the furnished drawings. For the purposes of this study, Alignment A is referenced as Alignment A in the geologic descriptions and reaches. Alignments C and D should be considered to coincide with Reach A in this area. Soil Reaches 8 through 10 were developed from the 27 borings in this area. However, as noted, these borings may not coincide with any or all of the current alignments.

Two of the available borings were utilized to define Soil Reach 11 at Mississippi Bayou. The remaining three borings were included with Soil Reach 1, but these borings coincide with Alignment C along the western side of the project.

Geotechnical data is not available for the portions of Alignments C and D which did not coincide with Alignment A at the time of this study. It has been projected that anticipated geologies at these locations are based on available data and information.

It should be noted that the geotechnical investigation was limited for this preliminary screening phase and did not include any exploration.

Methodology and Assumptions. The analyses consider the HSDRRS design guidelines dated 23 October 2007, with the geotechnical section as updated on 12 June 2008, although the scope does not include all cases required by this guideline. Required factors of safety and design cases are based on these guidelines. The HSDRRS design guidelines have been updated since issuance of the draft report. The scope of this study only includes an evaluation of Q-case parameters assuming eventual use of S-case parameters will be less restrictive.

Water Levels. Hydraulic design criteria were selected based on GFI in the form of preliminary hydrographic survey maps. The levees were evaluated using the water levels furnished for the future conditions anticipated for the year 2020. To include structural superiority, the floodwall analyses are based on water levels projected for the year 2070.

The scope of this alternative alignment screening level study included stability analyses by Spencer's Method for water at the project grade level (PGL), still water level (SWL) and low water level (LWL) at the levees. The scope did not include consideration of the Top of Levee (TOL), as this was not considered a critical design case for this alternative alignment screening level study. The scope for this study also did not include an evaluation of stability by the Method of Planes (MOP) analyses. Stability analyses for the structures only considered extreme water level (EWL) and SWL.

Stability Analyses. Stability of earthen levees for the 11 soil design reaches were evaluated. Five of these reaches were also evaluated with geotextile reinforcement to reduce the size of the berms. Nine structures (T-walls and gates) were also evaluated.

Levee Stability. The earthen levees generally consist of a 10-ft levee crown with 3 horizontal on 1 vertical (3H:1V) side slopes. Substantial stability berms on the flood side and protected side are required for Soil Reaches 6 through 10. For these reaches, the berms can be reduced with the addition of geotextile reinforcement. A tabular summary of the results along with a plate of the governing stability analysis results are provided in the Draft Geotechnical Report Appendix I from March 2012 (which is available upon request).

Structure Stability. The T-walls and gates are located within Soil Design Reaches 1, 8 and 11. The majority of the cases analyzed indicate the presence of an unbalanced load. A tabular summary of all the results along with a plate of the governing analyses are included in the Final Geotechnical Report Appendix I from February 2014 (which is available upon request). In addition to stability analyses, estimates of allowable pile load capacity were also computed for each soil reach where structures will be located.

Underseepage Analysis for Levees. With large stability berms required for several levees and considering a predominantly clay foundation, levee underseepage potential is not a significant design concern for most of the design soil reaches. However, Soil Reach 11 identified channel fill that will require either a cutoff, relief wells or seepage berms. Detailed underseepage analyses will be required during final design of theRecommended Plan to meet the HSDRRS design guidelines. The final field investigation should consider the estimated locations of abandoned distributaries and channel fill. Additional measures may be required to ensure adequate factors of safety are maintained.

Underseepage Analysis for Structures. Underseepage of pile-supported T-walls was evaluated using the Lane's Weighted Creep Ratio (LWCR) method to establish the tip elevations for the sheet pile cutoff wall. The flow path was assumed only to be the penetration of the sheet pile and horizontal contacts were not assumed. The sheet pile tip embedments are governed by seepage instead of the HSDRRS requirement of 5 feet of penetration below the critical failure plane (for unbalanced load cases).

Settlement Analyses. Settlement analyses were performed for Soil Reaches 1, 4, 6 and 10. An evaluation of the time-rate of consolidation settlement was not conducted; however, estimates for lift construction are available.

In general, settlement parameters for all reaches considered the surficial natural levee deposits and underlying Pleistocene deposits as precompressed. In addition, based on the available data, the swamp deposits were modeled to have an over consolidation ratio (OCR) between 3 and 10 in Soil Reaches 1 and 4 and between 1 and 2 in Soil Reaches 6 and 10. The interdistributary clays were typically modeled as normally consolidated. These values were based on the available boring data and correlations of moisture content to compression ratio

(CR) values developed in the region. The parameters generally only consider the stress history at the available boring locations. The stress history at alignments away from the boring data was not assumed.

The higher OCR values in the swamp deposits may only be applicable to previously developed areas in Alignment A. Thus, even in Soil Reach 1, additional lifts may be needed to maintain the levee height in previously undeveloped areas along Hope Canal and along Alignment C. Due to the shallow depth of the Pleistocene interface on the western side of the project, additional fill height would be anticipated to be low. However, moving eastward along the project as the Pleistocene interface increases in depth, the potential for lift construction would increase. Further, it appears current alignments diverge from developed areas east of the I-10/I-55 interchange, increasing this potential even further.

Based on the parameters developed for Soil Reaches 1 and 4, a minimum of 1.5-ft overbuild was assumed in all of the levee stability analyses. The overbuild height for Soil Reach 1 did not require consideration of submergence. Submergence was considered for Soil Reach 4. Settlements greater than 1.5 feet were computed for Soil Reaches 6 and 10 where larger berms and/or greater fill heights would be required. Thus, lift construction will be required for these reaches to maintain the design grade.

The greatest levee height and greatest settlement were computed for Soil Reach 10. This soil reach also has the deepest Pleistocene interface. For Soil Reach 10, an overbuild height of 2.6 feet was computed. It was estimated an additional 3 inches of settlement would occur for this overbuild once the initial levee is fully consolidated. This resulted in a total overbuild of approximately 3 feet. It was determined that only one additional lift thickness be assumed and this lift may be considered as 1.5 feet with an initial overbuild of 1.5 feet. It was also decided that this lift schedule be assumed for Soil Reaches 8, 9 and 10. Based on calculations for Soil Reach 6, it was estimated the overbuild would need to be increased from 1.5 feet to 2.5 feet. Thus, a 1-ft lift thickness beyond the initial 1.5-ft overbuild should be assumed. This lift thickness was applied to Soil Reaches 6 and 7. No lift schedule is deemed necessary for Soil Reaches 1 through 4 and 11 on Alignment A.

The furnished hydraulic data is based on a design year of 2020. The design levee heights were considered to occur from 2012 to 2020. This is a relatively short design period. Therefore, only one construction lift was assumed to be feasible. It was determined that this lift be estimated to occur halfway through the design period or four years into the eight-year design. Given the limited data for this screening study, only assumed time-rate of settlement parameters could be developed. However, even these assumptions would not address the stress history and time-rate away from the boring locations. For alignments within previously undeveloped areas, an additional lift or increased lift thickness may be required.

Datum and Topography

The furnished soil borings and the soil parameter plots are referenced to NGVD. These elevations were reduced by 1 foot for conversion to the NAVD88 datum. Water levels were provided in NAVD88. All the analyses for this feasibility report reflect the NAVD88 datum. Topographic survey data was not obtained for the alternative alignments. Review of available Lidar data indicated average grade at Elevation 1.0 NAVD88 should be used for the analyses of the levees. While the ground elevation varies along the length of each alignment, the assumed ground elevation of 1.0 NAVD88 was appropriate for the majority of the alignment and conservative for the areas of higher ground elevation. With the exception of furnished gate elevations, average grade at Elevation 1.0 was also used for the typical T-wall analyses.

Civil/Structural Design

Three alternatives were evaluated for scoping level engineering: Alignment A, Alignment C and Alignment D. Prior to the scoping level engineering, the alignments consisted of non-dimensional generalized locations on large scale mapping. The purpose of the scoping level engineering was to refine the generalized alignment locations into levee cross sections coordinated with existing topography features (streams, channels, wetlands, etc.) and existing infrastructure (highways, pipelines, utilities, etc.).

After the levee templates were completed, it was decided to apply the design templates to Alignments A, C and D.

A set of standard details was prepared to provide a schematic elevation view of the typical pump station T-Wall, Interstate T-Wall, Roadway/Railroad Floodgate T-Wall and Pipeline T-Wall. These typical elevations included clearance recommendations from the geotechnical engineers to ensure the new construction would not adversely impact existing infrastructure. Drawings showing the typical elevations are available upon request.

The pump station flow rates and gravity drainage gate sizes were computed. These pump station flow rates and gravity drainage gate sizes were based upon hydrologic units defined in the existing SWMM model. If multiple drainage outfalls existed in the hydrologic unit, the projected pump station flows and gravity drainage gate sizes were divided based upon the percentage of the outfall's contributory area in the delineated hydrologic unit. The pump stations were grouped into twelve types based upon the pump and gate sizes. Typical Floor Plans were developed for each pump station type. These typical floor plans and a typical elevation through the station are available upon request.

A "smoothed" version of **Alignment A** was used in order to minimize the encapsulation of wetlands in the protection system. Alignment A begins at the Upper Guide Levee of the Bonnet Carre' Spillway and travels westerly parallel to an existing pipeline corridor, around the Interstate 10/Interstate 55/US Highway 51 interchange, then follows Interstate 10 to the LA 3188 (Belle Terre Boulevard) interchange, then southerly and westerly paralleling the wetland wet/dry line to Mt. Airy where it terminates at the Mississippi River levee. The "smoothed" alignment was placed on the DOQQ base map and adjusted in a few minor locations. These locations included the Interstate 10 crossing east of the LaPlace interchange, the Interstate 55 crossing north of the US Highway 51 entrance ramp, the Interstate 10 crossing west of the Belle Terre interchange, and the existing water tower adjacent to the Belle Terre interchange. The modifications at the Interstate crossings were performed to cross the elevated structures with a ninety degree crossing that will ultimately be passed between existing bridge bents with a T-Wall. The Interstate 55 crossing was moved north to include the entrance/exit ramps from US Highway 55 and provide access for evacuation and recovery.

The top of levee elevation (net elevation) for this alignment is El. 13.5 NAVD88 (based on providing 100-Year protection in the Baseline Year of 2020), then decreases to El. 13.2 NAVD88 (at approx. Baseline Station 421+00), then decreases to El. 11.5 NAVD88 (at approx. Baseline Station 552+00), then decreases to El. 10.5 NAVD88 (at approx. Baseline Station 614+00), then decreases to El. 10.0 NAVD88 (at approx. Baseline Station 700+00), then decreases to El. 9.0 NAVD88 (at approx. Baseline Station 821+00) and finally decreases to El. 7.0 NAVD88 (at approx. Baseline Station 1013+00). The levee design, which involves the placement (in 2 lifts, 5 years apart) of approx. 3.1 million cubic yards of compacted and uncompacted clay fill, on top of 3.7 million square yards of geotextile fabric (with a 70-ft. width) along with a 100-ft. base width, 3:1 side slopes and 10-ft. crown width, creates a footprint of 411 acres. An aggregate limestone road (6 ft. wide x 8 in. thick) sits on top of the levee crown, a total of 29,615 cubic yards.

The design levee templates were placed along the proposed Alignment A at the defined soil and hydraulic reaches and based upon the recommended offsets for future maintenance activities, impacts to existing pile supported structures, offsets for stability from potential excavations (pipeline rights-of-way) and existing drainage features. Special attention was made to locate the right-of-way limits for the proposed levee sections to coincide with the existing rights-of-way from highways, pipelines etc. to avoid remainder parcels that were nonfunctional to the original owner. After the earthen embankments were placed on the base map and transitions performed from template section to template section, Alignment A was evaluated for specialty locations such as pump stations, T-Walls, gates, ramps, and pipeline crossings. The typical elevation details described above were utilized at appropriate locations and widths adjusted based upon the pump station size, Interstate crossing width, roadway/railway width, number of pipelines, etc. Alignment A was approximately 107,800 feet (20.41 miles) long and included 4,774 feet of T-Wall, 240 feet of drainage gates, 1,218 feet of roadway gates, two railway gates, seventy pipeline crossings, and eight pump stations. Schematic plans and typical levee sections (first and second lifts) were developed for Alignment A with levee template section, pump station, gate, T-Wall and pipeline crossings annotated. These schematic plans and typical levee sections are available upon request.

Alignment C begins at the Upper Guide Levee of the Bonnet Carre' Spillway and travels westerly parallel to an existing pipeline corridor, around the Interstate 10/Interstate 55/US Highway 51 interchange, then follows the existing pipeline corridor to Interstate 10/LA 3188 (Belle Terre Boulevard) interchange, then southerly and westerly paralleling the existing pipeline corridor to Mt. Airy where it terminates at the Mississippi River levee. Alignment C was developed to minimize the number of pipeline crossings.

The top of levee elevation (net elevation) for this alignment is El. 13.5 NAVD88 (based on providing 100-Year protection in the Baseline Year of 2020), then decreases to El. 13.2 NAVD88 (at approx. Baseline Station 304+00), then decreases to El. 12.2 NAVD88 (at approx. Baseline Station 354+00), then decreases to El. 10.2 NAVD88 (at approx. Baseline Station 612+00), then decreases to El. 9.0 NAVD88 (at approx. Baseline Station 722+00), then decreases to El. 7.5 NAVD88 (at approx. Baseline Station 905+00) and finally decreases to El. 7.0 NAVD88 (at approx. Baseline Station 968+00). The levee design, which involves the placement (in 2 lifts, 5 years apart) of approx. 3.1 million cubic yards of compacted and uncompacted clay fill, on top of 3.4 million square yards of geotextile fabric (with a 70-ft. width) along with a 100-ft. base width, 3:1 side slopes and 10-ft. crown width, creates a footprint of 856 acres. An aggregate limestone road (6 ft. wide x 8 in. thick) sits on top of the levee crown, a total of 26,124 cubic yards. A conveyance canal is situated along the entire levee (with a bottom depth elevation of El.-10 ft. NAVD88).

The design levee templates were placed along the proposed Alignment C at the defined soil and hydraulic reaches and based upon the recommended offsets for future maintenance activities, impacts to existing pile supported structures, offsets for stability from potential excavations (pipeline rights-of-way) and existing drainage features similar to Alignment A. There was a section of Alignment C from the Interstate 10/LA 3188 (Belle Terre Boulevard) interchange to the Mt. Airy community where there were no soil boring data and design levee templates were not developed. The other alignment's design levee templates that were in the closest proximity of the required hydraulic reach defined were used. Special attention was made to locate the right-of-way limits for the proposed levee sections to coincide with the existing rights-of-way from highways, pipelines etc. to avoid remainder parcels that were nonfunctional to the original owner. Once all the required design levee templates were selected for the hydraulic reaches, the levee sections were transitioned together similar to Alignment A. Alignment C was evaluated for specialty locations such as pump stations, T-Walls, gates, ramps and pipeline crossings.

Alignment C was approximately 96,500 feet (18.27 miles) long and included 5,304 feet of T-Wall, 2080 feet of drainage gates, 288 feet of roadway gates, two railway gates, thirty-six pipeline crossings, and four pump stations. Schematic plans and typical levee sections (first and second lifts) were developed for Alignment C with levee template section, pump station, gate, T-Wall and pipeline crossings annotated. These schematic plans and typical levee sections are available upon request.

Alignment D begins at the Upper Guide Levee of the Bonnet Carre' Spillway and travels westerly parallel to an existing pipeline corridor, around the Interstate 10/Interstate 55/US Highway 51 interchange, then follows the existing pipeline corridor to Interstate 10/LA 3188 (Belle Terre Boulevard) interchange, then westerly paralleling the Interstate 10 right-of-way approximately to the St James/Ascension Parish line, then turns northerly through the McElroy Swamp to the New River Canal, then westerly to the Marvin Braud Pump Station levee. Alignment D was developed to provide flood protection to the maximum number of communities in St Charles, St. John the Baptist, St. James, and Ascension Parishes and protect the Interstate 10 corridor. Alignment D also minimizes the number of pipeline crossings.

The top of levee elevation (net elevation) for this alignment is El. 13.5 NAVD88 (based on providing 100-Year protection in the Baseline Year of 2020), then decreases to El. 13.2 NAVD88 (at approx. Baseline Station 305+00), then decreases to El. 12.2 NAVD88 (at approx. Baseline Station 354+00), then decreases to El. 10.2 NAVD88 (at approx. Baseline Station 600+00), then decreases to El. 9.5 NAVD88 (at approx. Baseline Station 750+00) and finally decreases to El. 8.0 NAVD88 (at approx. Baseline Station 940+00). The levee design, which involves the placement (in 2 lifts, 5 years apart) of approx. 3.8 million cubic yards of compacted and uncompacted clay fill, on top of 3.1 million square yards of geotextile fabric (with a 70-ft. width) along with a 100-ft. base width, 3:1 side slopes and 10-ft. crown width, creates a footprint of 1,181 acres. An aggregate limestone road (6 ft. wide x 8 in. thick) sits on top of the levee crown, a total of 36,880 cubic yards. A conveyance canal is situated along the entire levee (with a bottom depth elevation of El.-10 ft. NAVD88).

The design levee templates were placed along the proposed Alignment D at the defined soil and hydraulic reaches and based upon the recommended offsets for future maintenance activities, impacts to existing pile supported structures, offsets for stability from potential excavations (pipeline rights-of-way) and existing drainage features similar to Alignments A and C. There was a section of Alignment D from the Interstate 10/Hope Canal crossing to the Marvin Braud levee where there were no soil boring data and design levee templates were not developed. The other alignment's design levee templates that were in the closest proximity of the required hydraulic reach defined were used. Special attention was made to locate the right-of-way limits for the proposed levee sections to coincide with the existing rights-of-way from highways, pipelines, etc. to avoid remainder parcels that were nonfunctional to the original owner. Once all of the required design levee templates were selected for the hydraulic reaches, the levee sections were transitioned together similar to Alignments A and C. Alignment D was evaluated for specialty locations such as pump stations, T-Walls, gates, ramps and pipeline crossings.

Alignment D was approximately 149,300 feet (28.28 miles) long and included 4,011 feet of T-Wall, 396 feet of drainage gates, 306 feet of roadway gates, no railway gates, fourteen pipeline crossings, and six pump stations. Schematic plans and typical levee sections (first and second lifts) were developed for Alignment D with levee template section, pump station, gate, T-Wall, and pipeline crossings annotated. These schematic plans and typical levee sections are available upon request.

Quantities. Quantities were computed for clearing and grubbing, geotextile, earthwork, aggregate roadway, turf establishment, T-Walls, drainage gates, roadway gates, railroad gates, pump stations and pipeline relocations.

Clearing and grubbing was based upon the proposed levee right-of-way limits denoted on the typical levee sections for the length of the reach and converted to acres. Geotextile was based upon the proposed width denoted on the typical levee sections for the length of the reach and converted to square yards. Earthwork was computed by end area denoted on the typical levee sections for the length of the reach. To determine the end area for each typical levee section, the average groundline elevation along the alignment centerline was computed. LIDAR data from the Louisiana State University Atlas Database was loaded into ArcGIS and the EZProfiler extension was used to obtain x, y, z, coordinates in Louisiana State Plane Coordinate System. The EZProfiler parameters were set to obtain coordinates and elevations every 45 feet along the alignment since the LIDAR data had 15 feet by 15 feet pixels. The EZProfiler dumped the coordinate and elevation data into an Excel spread, where the groundline elevation was averaged. The average groundline elevation was included in the levee typical section and the end areas were computed for each individual reach. After the end areas were computed, the length of the earthen levee segments were multiplied by the end area and ten by a 1.25 consolidation factor before converting into cubic yards. The 1.25 consolidation factor was used to account for consolidation and compaction of underlying existing soils as the new earthwork lifts are performed. Turf establishment quantities were set equal to the clearing and grubbing limits and converted to acres. Aggregate road surfacing was computed from the levee segment length and a section 6 feet wide and 8 inch deep then converted to cubic yards. T-Walls, Drainage Gates, and Roadway Gates were tabulated by length and incremental wall heights. An incremental wall height of 5 feet was set as the criteria. Railroad gates were measured per each. Pipeline relocations were measured per each and the incremental pipeline size. Incremental pipeline sizes were set at less than or equal to 6 inches, greater than 6 inches up to 12 inches, greater than 12 inches up to 18 inches, greater than 18 inches up to 24 inches and greater than 24 inches. All quantities for Alignments A, C and D were computed in the same manner.

Relocations

An ArcGIS State of La. Oil Spill Response Database was used to identify the pipeline locations for each alignment. This database contained not only the shapefiles of the pipelines but in most instances the owner, size, type and the carried material. This data was used for each of the three alignments. The assumption for each alignment was that a pipeline floodwall would be required wherever a pipeline crossed the levee footprint. The pipeline would cross through the pipeline floodwall. It was decided that the existing carrier line would remain in operation while a bypass line would be constructed through a sleeve in the T-wall cutoff piles. When the bypass would be complete and in place, the switch over-tie in with the existing line then would follow. A unit cost for the different pipe size ranges was used (unit costs were furnished by USACE). See below.

Pipeline Relocations

Description	Estimated Quantity (Q)	Units	Unit Cost (UC)
≤6" Diameter	14	Each	$515,000
>6" to ≤12" Diameter	16	Each	$700,000
>18" to ≤24" Diameter	5	Each	$1,550,000
> 24" Diameter	1	Each	$1,920,000

Cost Estimates

After each alignment's quantities were finalized, cost estimates were prepared for each alignment. For each item, the item description, item quantity, unit of measure, unit cost, item cost, contingency and total item cost was tabulated in an Excel spreadsheet; the same information was later prepared in MII MCACES format. Since the unit of measure for the pump stations was set by the cubic feet per second (cfs) flow rate of each type of pump station, separate quantities and costs were computed for each type of pump station. Separate tabs for each pump station were created in the Excel spreadsheet (and subsequently shown in the MII MCACES format for each alignment). The cost for each pump station was divided by the flow rate to determine the unit cost. All cost estimates for Alignments A, C and D were computed in the same manner.