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LOUISIANA COASTAL PROTECTION AND RESTORATION (LACPR) REPORT

COMMUNICATION

FROM

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TRANSMITTING

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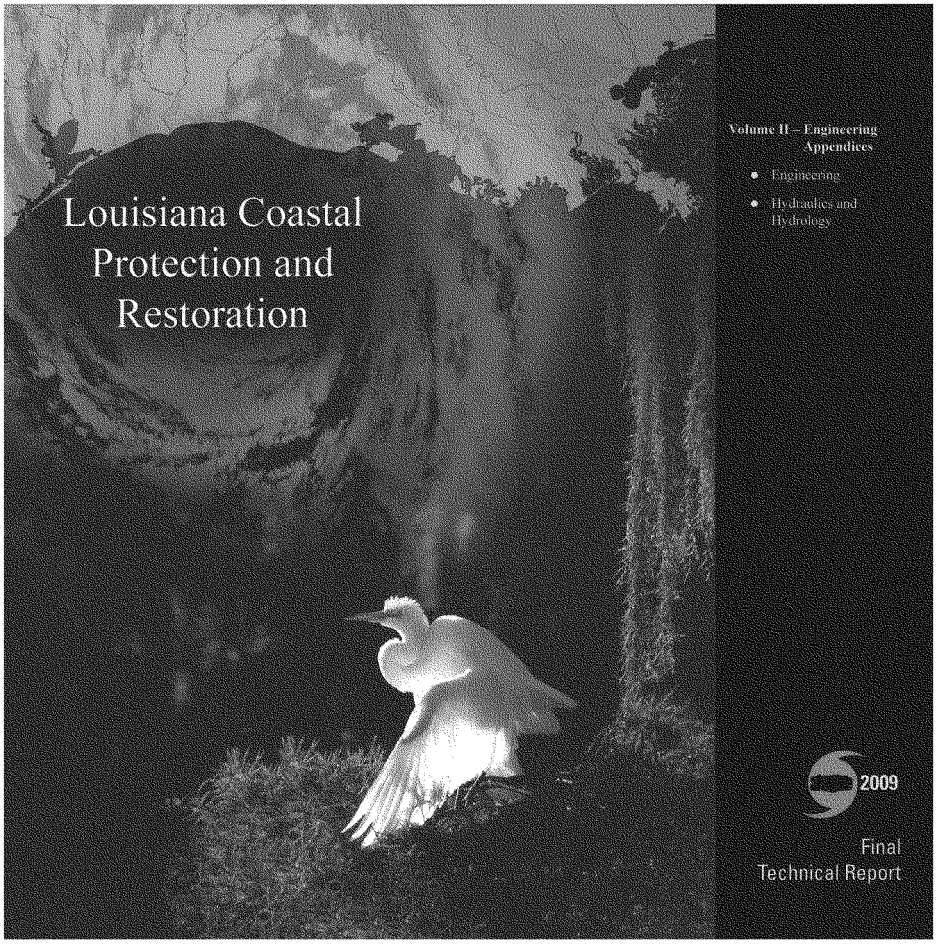
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FINAL TECHNICAL REPORT

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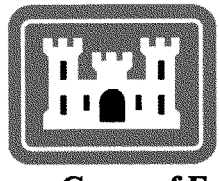

U. S. Army Corps of Engineers
New Orleans District
Mississippi Valley Division

LOUISIANA COASTAL PROTECTION AND RESTORATION ENGINEERING APPENDIX

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LOUISIANA COASTAL PROTECTION AND RESTORATION ENGINEERING APPENDIX

PURPOSE

The Louisiana Coastal Protection and Restoration (LACPR) Technical Report has been developed by the United States Army Corps of Engineers (USACE) in response to Public Laws 109-103 and 109-148. Under these laws, Congress and the President directed the Secretary of the Army, acting through the Chief of Engineers, to:

- Conduct a comprehensive hurricane protection analysis and design in close coordination with the State of Louisiana and its appropriate agencies;
- Develop and present a full range of flood control, coastal restoration, and hurricane protection measures exclusive of normal policy considerations for South Louisiana;
- Consider providing protection for a storm surge equivalent to a Category 5 hurricane; and
- Submit preliminary and final technical reports.

This Engineering Appendix outlines the engineering and design work done to support the preparation of the Louisiana Coastal Protection and Restoration (LACPR) Technical Report. It includes geotechnical investigations, levee design, structural design, coastal restoration features and cost estimates. Figure 1 shows the general planning area.

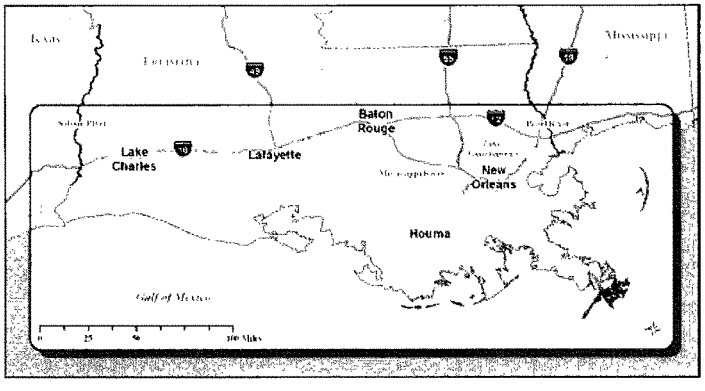


Figure 1 - LACPR Planning Area

STRUCTURAL ALIGNMENTS

Alternative alignments were developed in conjunction with other federal, state and local representatives and through extensive public involvement. Details on how these alignments were developed can be found in the Main Report and the Structural Plan Component Appendix.

The alignments were used to develop cost estimates used to compare the structural alternatives through the Risk-Informed Decision Framework (RIDF)/Multi-Criteria Decision Analysis (MCDA) process. More information on this process can be found in the RIDF Appendix. These proposed alignments do not represent exact construction alignments. It is expected that a more thorough analysis will be conducted during the detailed design phase to determine modification of alignments that could possibly represent significant improvements to the schedule, budget, and overall effectiveness of any future projects.

Confirmation of or amendments to the final construction alignment will be made with due deliberation and consideration of many significant factors, including the costs and benefits to the level of protection, the environment, constructability, schedule, hydraulics, economic and cultural resources, and other general and local issues. Innovative engineering will also be a key component in the alternative alignment analysis.

REACHES

Planning Units

The planning area was divided into different Planning Units (PU) based on hydrologic boundaries (see figure 2). These Planning Units extend as follows:

1. Lake Pontchartrain Basin, or the area east of the Mississippi River (PU1)
2. Barataria Basin, or the area from the Mississippi River west to Bayou Lafourche (PU2)

3a. Eastern Terrebonne Basin, or the area west of Bayou Lafourche to Bayou de West (PU3a)

3b. Atchafalalya Influence Area, or the area west of Bayou de West to Freshwater Bayou (PU3b)

4. Chenier Plain, or the area west of Freshwater Bayou to the Sabine River (PU4)

Figure 2 - Planning Units

Reaches and Sections

For design purposes the planning area was broken down into four reaches based on areas that have similar geotechnical conditions (areas where the stratigraphy remains relatively consistent). Alternative alignments in each reach were further broken down into sections for ease in developing cost estimates. Plates 1 thru 4 show the locations of the geotechnical reaches and alignment sections as well as the lengths of those sections in feet and miles.

HYDRODYNAMIC MODELING AND DESIGN

USACE/FEMA JOINT SURGE STUDY

The Federal Emergency Management Agency (FEMA) and the U.S. Army Corps of Engineers (USACE) collaborated on the USACE/FEMA Joint Surge Study (JSS) for Southeastern Louisiana The project modeled hurricane storm surge throughout coastal Southeastern Louisiana to provide output for various planning and design uses within the two federal agencies. FEMA intends to use the output of this modeling effort as input for the development of new Digital Flood Insurance Rate Maps (DFIRMS) and other uses, while USACE is using the output of this study in the evaluation and design of hurricane protection systems including the LACPR effort. Details of the methodology and results of this study are included in the Hydraulics And Hydrology (H&H) Appendix.

ANALYSIS AND DESIGN

In the framework of LACPR the hydraulic analysis plays a key role in the cost-benefit analysis of the alternatives in the various planning units. Each levee alternative affects the propagation of the surge and the waves during a storm in a different way. The differences in storm surge and wave characteristics result in varying levee designs, overtopping volumes and interior stage frequency curves.

The hydraulic analysis of each alternative in LACPR consisted of the following consecutive steps:

1. Numerical computations of surge levels and wave characteristics
2. Frequency analysis
3. Determination of the levee heights and overtopping volumes
4. Determination of the interior stages including rainfall

To provide a range of alternatives for evaluation and to enable the economic evaluation it was decided to evaluate each levee alternative for different design levels and event frequencies. A levee design was made for three different levels of risk reduction (100-year, 400-year, 1000-year). A more detailed description of the design methodology and results of this effort are included in the H&H Appendix.

Planning Units 1 and 2

Levee design height results were provided for base conditions and east model grids A and B for different hydraulic reaches for two different modeling conditions with friction and without friction[1]. These model grids are described below and in more detail in the H&H Appendix. Details of the with and without friction conditions can also be found in the H&H Appendix along with maps showing the location of the hydraulic reaches.

Model Grid A: This grid includes non-overtopping barriers across the eastern edge of Lake Pontchartrain and along the GIWW in Barataria Basin (PU2).

Model Grid B: This grid includes overtopping barriers across the eastern edge of Lake Pontchartrain and along the GIWW in Barataria Basin.

A determination was made as to which of the previously described levee sections fell into each hydraulic reach. Based on preliminary analysis it was determined that non-overtopping barrier alternatives would be removed from further consideration because of their higher cost in relation to benefits over those provided by the overtopping barrier alternatives. Details of this preliminary analysis can be found in the Structural Plan Component Appendix. Since only barrier alternatives with overtopping barriers were considered in detail these are the only design results shown. Tables 1 to 4 below show the design levee heights for the base condition grid and grid B for with and without friction cases.

With and without friction cases differ in that STWAVE, a near shore wave generation and transformation model which was used as part of the hydraulic analysis done for this project, has an option to include calculation of wave dissipation due to bottom roughness or vegetation. STWAVE simulations were executed both with and without activating the frictional dissipation in the model. Simulations with friction will have lower wave heights in shallow, submerged wetland areas than the without friction simulations. Although, in areas strongly forced by the winds, frictional dissipation can be balanced by wind input.

[1] All elevations in this report are to NAVD88(2004.65).

The issue of the "with friction" or "without friction" STWAVE results was resolved for the LACPR project when the ongoing effort for designing the 100-year protection system settled on the "without friction case" as being the one used to design the reconstructed levee system. The team doing this work employed an ITR and EPR review process. The judgment of the EPR Team was that the with friction STWAVE results appeared to be unrealistic and not conservative enough, given the potential consequences of overtopping and the lack of wave data to support the friction factors used in the STWAVE simulations. It was decided that the LACPR methodology employ the same methodology used in the restoration effort since that effort when completed would be the base condition 2010 case for LACPR.

Without friction case elevations were used to develop costs used for running the MCDA. Estimates developed using the with friction case elevations were used in performing a sensitivity analysis to determine if using the with friction vs. without friction cases would affect project ranking (see Structural Plan Component Appendix). This analysis determined that the choice of with and without friction would not on balance affect the ranking of any plan.

Table 1
Design Elevations For Base Conditions (without friction) Planning Unit 1 and Planning Unit 2

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plate 1)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
BS-0005	1	6	26.5	32.5	35.5
BS-0007	1	7	26	34	38
BS-0009	1	Plaq EB (8)	29	38.5	42.5
BS-0026	1	1005c	18.5	20	23.5
BS-0032	1	1005a	16	21	23.5
BS-0037	2	403b,403c	11	14	18
BS-0040	1	1005f	18	23.5	26
BS-0048	1	5b,1004a,1004b	28.5	35	38
BS-0058	1	1005d	15	18	21
BS-0074	2	14b,1008	18	25	28.5
BS-0075	2	14a	17	21.5	23.5
BS-0092	1	1005b	16.5	19.5	21.5
BS-0093	1	1005e	18	19	21.5
BS-0094	1	1002b(1)	16.5	23	26
BS-0101	2	403d,403c	11	14	16
BS-0110	2	409	7.5	10.5	12.5
BS-0111	2	404c(1)	8	11.5	13
BS-0112	2	403f	8.5	12	13.5
BS-0130	2	1016,403a	11	17	20
BS-0136	1	1000c	16	20	22
BS-0137	1	1000b	15	19	21.5
BS-0138	1	1000a	15	20	22.5
BS-0139	1	1001a,1001b,1001c,1002a	16	21.5	24
BS-0157	1	1018	10	14	16.5
BS-0158	1	1015,1017	10	15	17.5

Table 1 Design Elevations For Base Conditions (without friction) Planning Unit 1 and Planning Unit 2					
Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plate 1)	Design Level		
BS-0175	1	1003,1002b(2)	22.5	30.5	34.5
BS-0176	1	1006	13.5	18.5	21
BS-0177	1	1a(1),1b	20	28	32
BS-0205	2	13,109	12	14	16.5

Table 2 Design Elevations For Grid B (without friction) Planning Unit 1 and Planning Unit 2					
Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plate 1)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
EB-0007	1	7	28	37.5	40.5
EB-0009	1	Plaq EB (8)	33	42.5	45.5
EB-0026	1	1005c	18.5	18.5	18.5
EB-0032	1	1005a,16	13	18.5	21.5
EB-0058	1	1005d	15	15	15
EB-0074	2	109,14b, 1008	18.5	26.5	30.5
EB-0075	2	13, 14a	17.5	23	26
EB-0092	1	1005b	16.5	16.5	18.5
EB-0093	1	1005e	18	18	18
EB-0094	1	1002b,1003	13	21	24.5
EB-0101	2	403a,403b,403c,403d, 403e, 1016b	11	11	11
EB-0110	2	409	6	8.5	10
EB-0112	2	403f,404c(1)	6.5	9	10.5
EB-0129	1	6,1004a, 1004b,5b	26.5	33.5	37
EB-0130	2	1016 (a)	16	22	25
EB-0136	1	1000c	11	15.5	17.5
EB-0137	1	1000b	9.5	13.5	16
EB-0138	1	1000a	9.5	14.5	17.5
EB-0139	1	1001a,1001b,1001c, 1002a	11	17.5	21
EB-0147	1	1005f	15	16.5	20.5
EB-0175	1	2,3	24.5	32	36
EB-0176	1	1006	9	14.5	18
EB-0177	1	1b,1a(1),1a(2)	26	35	39

Table 3
Design Elevations For Base Conditions (with friction) Planning Unit 1 and Planning Unit 2

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plate 1)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
BS-0005	1	6	24	29.5	32
BS-0007	1	7	21.5	28	31.5
BS-0009	1	Plaq EB (8)	24	31	35
BS-0026	1	1005c	18.5	20	23.5
BS-0032	1	1005a	14.5	19.5	21.5
BS-0037	2	403b,403c	11	12.5	16
BS-0040	1	1005f	17.5	23	26
BS-0048	1	5b,1004a,1004b	24	30	32
BS-0058	1	1005d	15	18	21
BS-0074	2	14b,1008	15.5	21.5	25
BS-0075	2	14a	16	20.5	22.5
BS-0092	1	1005b	16.5	19.5	21.5
BS-0093	1	1005e	18	19	21.5
BS-0094	1	1002b(1)	16.5	23	26
BS-0101	2	403d,403e	11	14	16
BS-0110	2	409	7	10	12
BS-0111	2	404c(1)	8	11.5	13
BS-0112	2	403f	8	11	13
BS-0130	2	1016,403a	11	15	17
BS-0136	1	1000c	16	20	22
BS-0137	1	1000b	15	19	21.5
BS-0138	1	1000a	15	20	22.5
BS-0139	1	1001a,1001b,1001c, 1002a	16	21.5	24
BS-0157	2	1018	10	14	16
BS-0158	1	1015,1017	10	15	17.5
BS-0175	1	1003,1002b(2)	19	25.5	29
BS-0176	1	1006	12.5	17	19
BS-0177	1	1a(1),1b	17.5	23.5	26.5
BS-0205	2	13,109	11	14	16.5

Table 4
Design Elevations For Grid B (with friction) Planning Unit 1 and Planning Unit 2

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plate 1)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
EB-0007	1	7	23	30.5	35
EB-0009	1	Plaq EB (8)	25.5	34.5	40
EB-0026	1	1005c	18.5	18.5	18.5
EB-0032	1	1005a,16	13	17	19.5
EB-0058	1	1005d	15	15	15
EB-0074	2	109,14b,1008	15.5	22	25.5
EB-0075	2	13,14a	16.5	22	24.5
EB-0092	1	1005b	16.5	16.5	18.5
EB-0093	1	1005e	18	18	18
EB-0094	1	1002b,1003	12.5	20.5	24.5
EB-0101	2	403a,403b,403c,403d, 403e, 1016b	11	11	11
EB-0110	2	409	5.5	8	9.5
EB-0112	2	403f,404c(1)	6	9	10
EB-0129	1	6,1004a,1004b,5b	26.5	33.5	37
EB-0130	2	1016 (a)	14	19	22
EB-0136	1	1000c	11	15.5	17.5
EB-0137	1	1000b	9.5	13.5	16
EB-0138	1	1000a	9.5	14.5	17.5
EB-0139	1	1001a,1001b,1001c, 1002a	11	17.5	21
EB-0147	1	1005f	15	16.5	20
EB-0175	1	2,3	21.5	27.5	30.5
EB-0176	1	1006	8	13	16
EB-0177	1	1b,1a(1),1a(2)	20.5	27	30

New North Shore Alignments

Levee alignments along the north shore of Lake Pontchartrain were reanalyzed in an effort to provide a better line of protection for existing development. These new alignments are shown on Plate 5. New design elevations were developed for these levees and are shown in Tables 5 and 6. Design elevations and costs were developed only for the without friction case for the new north shore alignments.

Table 5
Design Elevations For Base Conditions (without friction) New North Shore Alignments

PU	Levee Section (see plate 5)	Design Level		
		100-YR (ft)	400-YR (ft)	1000-YR (ft)
1	NNS0	16	20	22
1	NNS1	15	19	22
1	NNS2	15	19	22
1	NNS3	15	20	23
1	NNS4	15	20	23
2	NNS5	20	28	32
2	NNS6a	20	28	32
1	NNS6b	23	31	35
1	NNS6c	17	23	26
1	NNS6d	16	22	24
2	NNS7	16	20	22
2	NNS8	16	20	22
2	NNS9	16	20	22

Table 6
Design Elevations For Grid B (without friction) New North Shore Alignments

PU	Levee Section (see plate 5)	Design Level		
		100-YR (ft)	400-YR (ft)	1000-YR (ft)
1	NNS0	11	16	18
1	NNS1	10	14	16
1	NNS2	10	14	16
1	NNS3	10	15	18
1	NNS4	10	15	18
2	NNS5	26	35	39
2	NNS6a	26	35	39
1	NNS6b	13	21	25
1	NNS6c	13	21	25
1	NNS6d	11	18	21
2	NNS7	11	16	18
2	NNS8	11	16	18
2	NNS9	11	16	18

Planning Units 3a, 3b and 4

Three hyrdo modeling grids were run to develop the levee design heights for alternatives for PU3a, PU3b and PU4 for two different modeling conditions with friction and without friction. These model grids are described below and in more detail in the H&H Appendix.

West Model Grid A: This grid includes the Morganza to the Gulf alignment and then follows the GIWW to just east of the state line.

West Model Grid B: This grid includes the Morganza to the Gulf alignment and includes a back ring levee around the Houma area, ring levees for Morgan City and Patterson, a levee along the wetland interface from Patterson to Abbeville and ring levees around Kaplan, Gueydan and a larger ring levee around Lake Charles.

West Model Grid C: The grid incorporates features of both levee grids.

A determination was made as to which of the previously described levee sections fell into each hydraulic reach. Tables 7 to 8 below show the design levee heights for with and without friction cases. Without friction case elevations were used to develop costs used for running the MCDA. Estimates were also developed using the with friction case elevations for use in performing a sensitivity analysis.

Table 7
Design Elevations For Planning Unit 3a, Planning Unit 3b and Planning Unit 4 (without friction)

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
WA-W0017	3a	14b	21	29.5	34.5
WA-W0025	3a	15, 16	22	30.5	35.5
WA-W0026	3a	17, 18, 19, 20a	28	36.5	41
WA-W0035	3a	20b	23	31	35.5
WA-W0045	3a	21	15	24	29
WA-W0050	3a	1022, 22, 23b(1), 5000	12	19.5	23.5
WA-W0056	3a, 3b	23b(2), 23a(1), 24a	16.5	23	27
WA-W0063	3b	2007b	20	27.5	32.5
WA-W0073	4	807b	16	24	28.5
WA-W0079	3b	801b, 2007a, 2802	20.5	28	32.5
WA-W0082	4	807a	14	20.5	24.5
WA-W00103	4	803, 804, 805a	21.5	28	31.5
WA-W00119	4	807c, 807d	13.5	21.5	26
WA-W00124	4	806	12	19.5	24
WB-W0056		23a	18	25	28.5
WB-W0060	3b	301a, 801b	16.5	23.5	27.5
WB-W0075	3b	2302a	20	27	31
WB-W0085	3b	302b, 2302b	17	26	31.5

Table 7
Design Elevations For Planning Unit 3a, Planning Unit 3b and Planning Unit 4 (without friction)

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
WB-W0089	3b	301c	19	26.5	30.5
WB-W0090	3b	2006	18	24.5	28.5
WB-W0094	3b	301b, 301b(2), 3001	16.5	25	30
WC-W0169	3a	110,111,112		9.5	24
WT-W0104	3b	3002	14.5	22	26.5
WT-W0110	3b	3005	12.5	20.5	25
WT-W0111	3b	3003, 3004	17.5	25	29
WT-W0127	4	3009, 3010	11	15.5	17.5
WT-W0135	4	3006	9	15.5	18.5
WT-W0153	4	3007	9	15.5	18.5
WT-WPATT	3a	23a(2), 1023a, 24b, 801a	13.5	20	24
WT-W0999	4	3008	13.5	17	19.5

Table 8
Design Elevations For Planning Unit 3a, Planning Unit 3b and Planning Unit 4 (with friction)

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
WA-W0017	3a	14b	17.5	24	28
WA-W0025	3a	15, 16	18.5	25.5	29.5
WA-W0026	3a	17, 18, 19, 20a	21	29.5	34
WA-W0035	3a	20b	19	25	28.5
WA-W0045	3a	21	13.5	19.5	23
WA-W0050	3a	1022, 22, 23b(1), 5000	10.5	15.5	18
WA-W0056	3a, 3b	23b(2), 23a(1), 24a	12.5	17.5	20.5
WA-W0063	3b	2007b	18.5	25	29
WA-W0073	4	807b	13.5	19	22.5
WA-W0079	3b	801b (GIWW), 2007a, 2802	19.5	26.5	31
WA-W0082	4	807a	12.5	18.5	21.5
WA-W00103	4	803, 804, 805a	15.5	20.5	23.5
WA-W00119	4	807c, 807d	12.5	19	22.5
WA-W00124	4	806	11	17	20.5
WB-W0060	3b	301a, 801b	12	17.5	20
WB-W0075	3b	2302a	16	21.5	25
WB-W0085	3b	302b, 2302b	15.5	22	26
WB-W0089	3b	301c	14.5	21	25
WB-W0090	3b	2006	13.5	18.5	21
WB-W0094	3b	301b, 301b(2), 3001	12.5	18.5	22
WC-W0169	3a	110,111,112			

Table 8 Design Elevations For Planning Unit 3a, Planning Unit 3b and Planning Unit 4 (with friction)					
Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
WT-W0104	3b	3002	12	17	20.5
WT-W0110	3b	3005	12	17.5	20.5
WT-W0111	3b	3003, 3004	14	19	22.5
WT-W0127	4	3009, 3010	11	15.5	17.5
WT-W0135	4	3006	6.5	11.5	15
WT-W0153	4	3007	8.5	14	17
WT-WPATT	3a	23a(2), 1023a, 24b, 801a	9.5	14.5	17.5
WT-WLAKC	4	3008	10.5	13	15

FUTURE CONDITIONS

Degraded Coastline

An additional modeling grid was developed to determine the effects of a degraded coastline on surge and wave heights. Details on how this grid was developed (how the degraded coastline was estimated) can be found in the H&H Appendix. New levee elevations necessary to provide the desired protection under these conditions were developed. These levee elevations (for the without friction case) are shown in Tables 9 thru 10 for high level and barrier plans. High level plans are those alternatives which consist of raising existing levees and constructing new levees. Barrier plans are those alternatives which include building a barrier across Lake Pontchartrain in PU1 and along the GIWW in PU2 along with raising the existing levees and constructing new levees.

Table 9 Design Elevations For Degraded Coastline Condition High Level Plans - Without Friction Case Planning Unit 1 and Planning Unit 2					
Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
BS-0005	1	6	28.5	36.5	41
BS-0007	1	7	27	36.5	40
BS-0009	1	Plaq EB (8)	32.5	42.5	45
BS-0026	1	1005c	18.5	20	23.5
BS-0032	1	1005a	17	23	26
BS-0037	2	403b,403c	12	18	21.5
BS-0040	1	1005f	19	25	29
BS-0048	1	5b,1004a,1004b	32	40.5	43.5
BS-0058	1	1005d	15	19	21.5
BS-0074	2	14b,1008	18	25	28.5
BS-0075	2	14a	17	21.5	23.5

Table 9
Design Elevations For Degraded Coastline Condition High Level Plans - Without Friction Case
Planning Unit 1 and Planning Unit 2

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR (ft)	400-YR (ft)	1000-YR (ft)
BS-0092	1	1005b	16.5	19.5	21.5
BS-0093	1	1005e	18	20	23
BS-0094	1	1002b(1)	17.5	24	27.5
BS-0101	2	403d,403e	12	17.5	20
BS-0110	2	409	10	14	16.5
BS-0111	2	404c(1)	10.5	14.5	17
BS-0112	2	403f	11	15.5	18.5
BS-0130	2	1016,403a	14	19.5	23
BS-0136	1	1000c	16	20	22
BS-0137	1	1000b	15	19	21.5
BS-0138	1	1000a	15	20	23
BS-0139	1	1001a,1001b,1001c,1002a	16	22	25
BS-0157	2	1018	11.5	16	18.5
BS-0158	2	1015,1017	11.5	17	20
BS-0175	1	1003,1002b(2)	24	33	37.5
BS-0176	1	1006	14	19	22.5
BS-0177	1	1a(1),1b	23	32	36.5
BS-0205	2	13,109	14	18	19.5

Table 10
Design Elevations For Degraded Coastline Condition Barrier Plans - Without Friction Case
Planning Unit 1 and Planning Unit 2

Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR	400-YR	1000-YR
EB-0007	1	7	29	40	42.5
EB-0009	1	Plaq EB (8)	36.5	46.5	48
EB-0026	1	1005c	18.5	18.5	18.5
EB-0032	1	1005a,16	14	20.5	24
EB-0058	1	1005d	15	16	15.5
EB-0074	2	109,14b, 1008	18.5	26.5	30.5
EB-0075	2	13, 14a	17.5	23	26
EB-0092	1	1005b	16.5	16.5	18.5
EB-0093	1	1005e	18	19	19.5
EB-0094	1	1002b,1003	14	22	26
EB-0110	2	403a,403b,403c,403d, 403e, 1016b	12	14.5	15
EB-0111	2	409	8.5	12	14

Table 10 Design Elevations For Degraded Coastline Condition Barrier Plans - Without Friction Case Planning Unit 1 and Planning Unit 2					
Hydraulic Reach (see H&H Appendix)	PU	Levee Section (see plates 2-4)	Design Level		
			100-YR	400-YR	1000-YR
EB-0112	2	403f,404c(1)	9	12.5	15.5
EB-0129	1	6,1004a, 1004b,5b	28.5	37.5	42.5
EB-0130	2	1016 (a)	19	24.5	28
EB-0136	1	1000c	11	15.5	17.5
EB-0137	1	1000b	9.5	13.5	16
EB-0138	1	1000a	9.5	14.5	18
EB-0139	1	1001a,1001b,1001c, 1002a	11	18	22
EB-0147	1	1005f	16	18	23.5
EB-0175	1	2,3	26	34.5	39
EB-0176	1	1006	9.5	15	19.5
EB-0177	1	1b,1a(1),1a(2)	29	39	43.5

Relative Sea Level Rise

Two estimates of future relative sea level rise (RSLR) were developed. These estimates vary among the planning units. These estimates are shown below.

	Planning Unit 1	Planning Unit 2
RSLR Estimate 1	1.3 feet	1.9 feet
RSLR Estimate 2	2.6 feet	3.2 feet

	Planning Unit 3a, 3b	Planning Unit 4
RSLR Estimate 1	1.9 feet	1.3 feet
RSLR Estimate 2	3.2 feet	2.6 feet

Information on how these estimates were developed can be found in the H&H Appendix.

ANALYSIS OF LAKE PONTCHARTRAIN AND BARRIER STRUCTURE SIZING

The University of New Orleans performed a study to determine the required size of the structures at the Rigolets and Chef Passes to avoid adversely affecting the tidal prism. These would be gated structures allowing tidal exchange during non-storm conditions. This preliminary study determined that the structure at the Rigolets would need to be 1900 feet wide and the structure at the Chef would need to be 750 feet wide. Details of this study are included as an attachment to the H&H. Further study will be necessary during detailed design to determine the exact size and configuration of these structures.

INTERCEPTED DRAINAGE

Because results from the detailed interior drainage modeling work are not available at this time, estimates of additional pumping that would be required for new levee alignments were made using a simplified method. Details of this methodology can be found in the H&H Appendix. Pumping requirements for North Shore levees are shown in Figure 3.

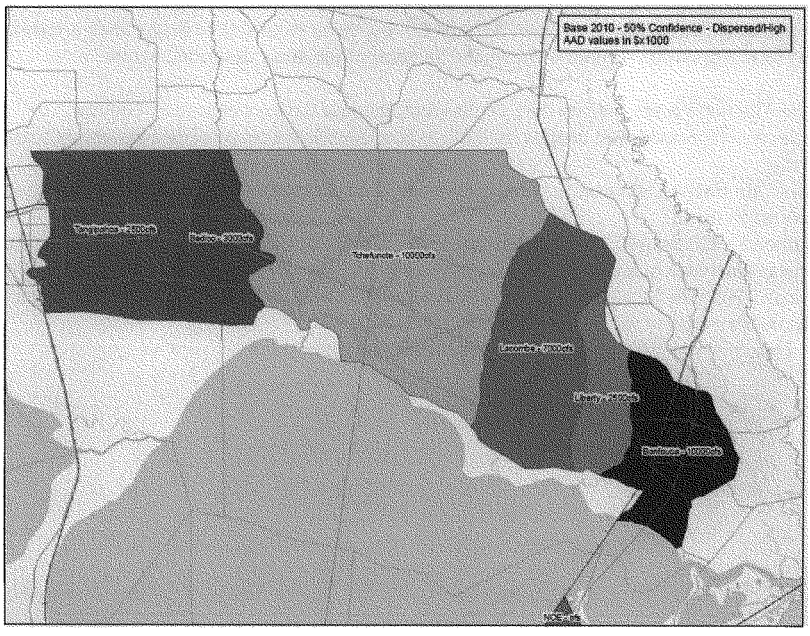


Figure 3 - Estimated Pumping Capacity For Northshore Leveed Areas

Estimates were also made for pumping requirements for leveed areas in PU2. New requirements are 15,000 cfs for the area on the east side of PU2 and 2500 cfs for the area on the west side of PU2. These estimates will be refined as more detailed modeling information becomes available.

Pumping requirements were also developed for the new north shore levees. These requirements are shown below.

Area	Required Pumping Capacity
Madisonville	5,000 cfs
South Covington	1,600 cfs
Madisonville to Madisonville	5,600 cfs
Mandeville	5,200 cfs

West Lacombe	2,400 cfs
East Lacombe	5,000 cfs
Slidell	54,000 cfs

INCREASED SURGE LEVEL ON PLAQUEMINES PARISH LEVEES

Estimates of the additional levee elevations needed to prevent induced damages to areas in Plaquemines Parish affected by construction of barriers in PU1 and PU2 were developed. These estimates are shown below. Details of how these values were determined are shown in the H&H Appendix. Figure 4 shows the location of the areas described below.

Ollie - increase levee heights by 2.6ft to accommodate the max change at 100year.
Bellevue - increase levee heights by 0.6ft to accommodate a reasonably constant increase in level.
Pointe ala Hache - increase levee heights by 0.3ft to accommodate a reasonably constant increase in level.
Diamond - Mississippi Levee - increase levee heights by 0.3ft to accommodate a reasonably constant increase in level.
Gainard Woods - Mississippi Levee - increase levee heights by 0.2ft to accommodate a reasonably constant increase in level.

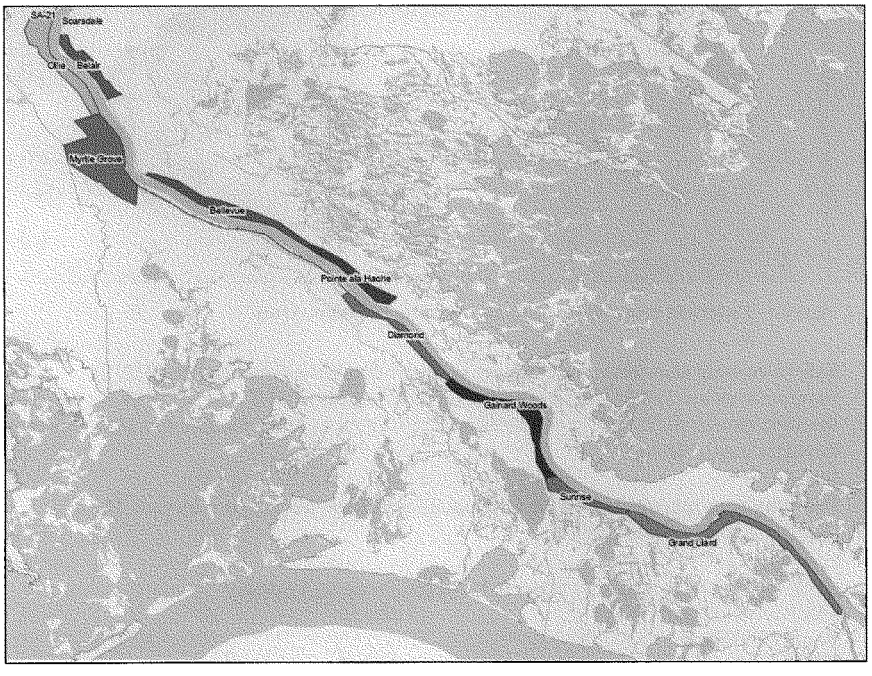


Figure 4 - Areas Subject to Additional Surge by Construction of Barrier Plans

GEOTECHNICAL

GEOLOGY

Geologic Formations

The history of the planning area is dominated by deltaic growth. The Mississippi River deltaic plain is composed of two active and several inactive deltaic complexes extending some 180 miles across southeast Louisiana. Several major deltaic complexes that have formed during the last 7,000 years have been identified in coastal Louisiana. These complexes reflect changes in the course of the Mississippi River. From oldest to youngest, the abandoned delta complexes are the Maringouin, Teche, St. Bernard, Lafourche, and Plaquemine. The active complexes include the Atchafalaya and Modern deltas. The relative ages of these complexes are well established, but the absolute ages are less accurate. Ages were derived from radiocarbon data published in previous studies (Frazier, 1967 and McFarlan, 1961) as well as archeological evidence (McIntire, 1958).

Progradation of these deltas is responsible for the formation of two distinct geomorphic regions; the deltaic plain in the central and southeastern portions of coastal Louisiana and the chenier plain in the southwestern part of the state (see Figure 5). Progradation of the present and former Mississippi River courses and deltas are responsible for creating the recent alluvial valley and deltaic plain of southeastern Louisiana. Each time the Mississippi River has built a major delta lobe seaward, it has subsequently been abandoned in favor of a shorter, more direct route to the sea. These meander belt changes in the alluvial valley and accompanying shifts in centers of deposition have resulted in the distribution of deltaic sediments along the coast of central and southeast Louisiana. Soon after a delta lobe is abandoned, marine transgression caused by compaction and subsidence of deltaic sediments begins. The end result of this long period of deltaic sedimentation has been the formation of a vast expanse of marsh and swamp separated by abandoned courses and distributaries.

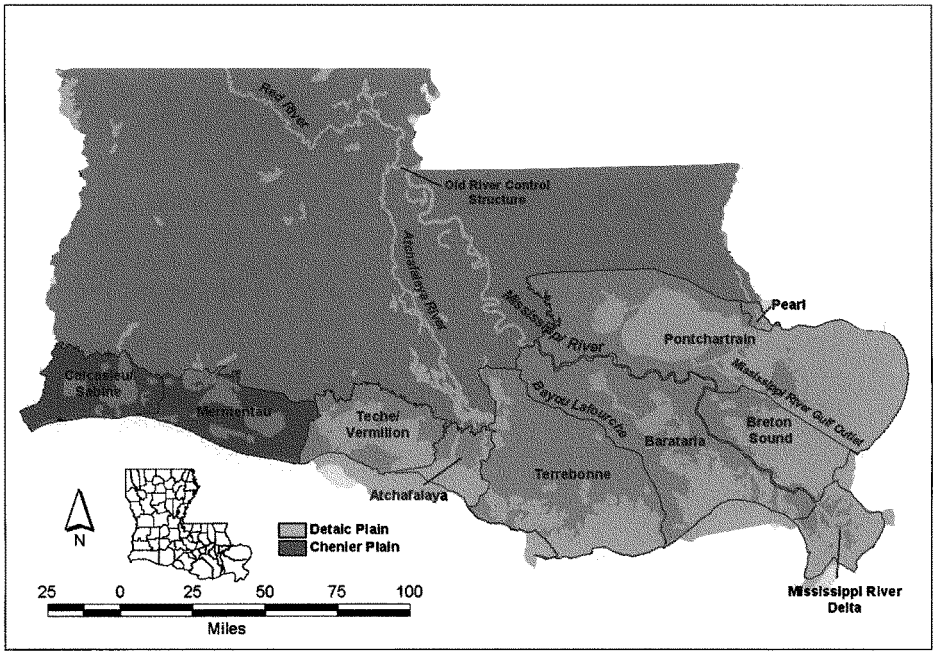


Figure 5 - Location of Chenier and Deltaic Plains

The deltaic plain is generally composed of a characteristic suite of depositional environments which include marsh, swamp, natural levee, interdistributary, prodelta, and Pleistocene. At the surface and shallow subsurface, marsh, swamp, and natural levee deposits make up the majority of the depositional environments. Marsh deposits range from 1 to 20 feet thick and are generally composed of very soft, organic clays and peat, with high water content and low strengths. Swamp deposits vary widely in thickness, water content, and strength, and are characterized by soft to stiff clays with some silt, wood, and organics. Natural levee deposits border the hundreds of abandoned distributaries and courses located throughout the deltaic plain. They reach a maximum of 30 feet in thickness and are generally composed of oxidized, medium to stiff, clays and silty clays having relatively low water content and high strength. Located beneath marsh, swamp, and natural levee deposits are interdistributary deposits. Interdistributary deposits are up to 200 feet thick and are characterized by very soft to soft clays with some silt and shells. Interdistributary deposits have relatively high water content and low strength. Prodelta deposits are generally found beneath interdistributary deposits. They are composed of medium clay with relatively low water content and moderate strengths. Prodelta deposits are up to 120 feet thick in the deltaic plain. Prodelta clays sit stratigraphically above Pleistocene deposits. Pleistocene deposits represent the best foundation material in coastal Louisiana. They typically consist of interbedded, highly oxidized, stiff to very stiff, clay, silt, silty sand, and sand. These deposits have low water content and high strengths. In the central portion of the deltaic plain, where the ancestral

Mississippi River entrenchment is located, substratum sands are located beneath prodelta deposits. A cross section showing an example of deltaic plain geology is shown in figure 6.

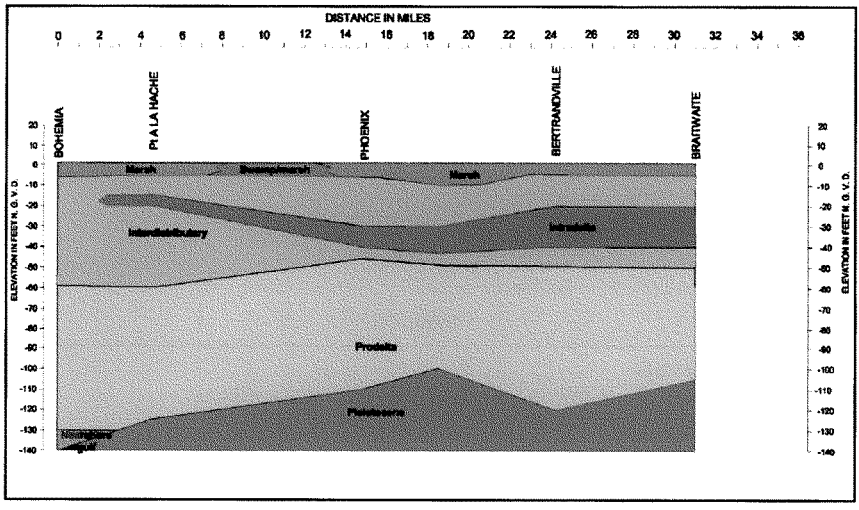


Figure 6 - Deltaic Plain

In contrast to the deltaic plain, the chenier plain formed by longshore transport of fine-grained Mississippi River sediments that were deposited to the west of the deltaic plain. These sediments, transported by westward flowing nearshore currents, were eventually deposited along the existing shoreline as mudflats. When deposition ceased or declined due to shifting Mississippi River courses, these deposits were reworked by coastal processes, concentrating the coarse grained sediments, and forming shore-parallel ridges called "cheniers" (Gould and McFarlan, 1959; Byrne et al, 1959). Introduction of new sediment by westward shifts of the Mississippi River delta resulted in the isolation of these ridges by accretion of new material on the existing shoreline. Numerous cycles of deposition and erosion have been responsible for creating the alternating ridges separated by marshlands, which are characteristic of the chenier plain. The Atchafalaya River is currently supplying the chenier plain with fine sediments via westward flowing longshore currents.
In the northern portion of the chenier plain, marsh deposits up to 10 feet in thickness overly Pleistocene deposits. In the southern half, chenier ridges up to 10 feet thick, composed of sand and shell material separated by marsh deposits approximately 5 feet thick, overly approximately 20 feet of lacustrine, tidal flat, and Gulf bottom clays. These clays are generally soft with relatively high water contents and low strengths. These clays sit directly on Pleistocene deposits. A cross section showing an example of chenier plain geology is shown in figure 7.

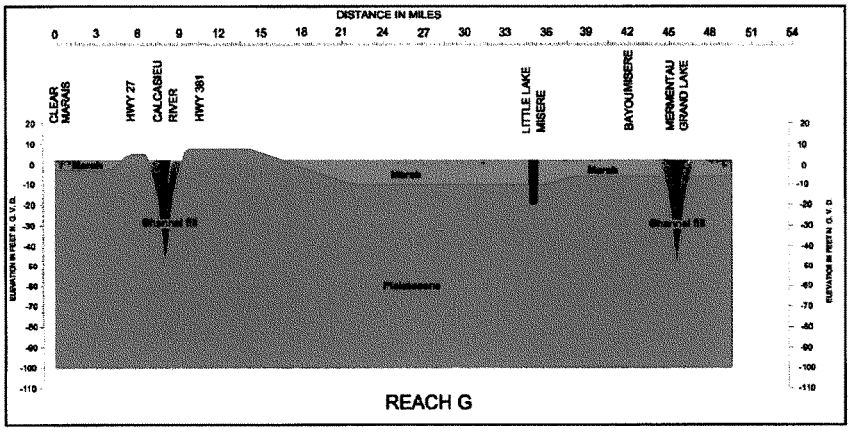


Figure 7 - Chenier Plain

Groundwater

Groundwater is at or near the surface throughout most of the coastal zone. The silt and sand rich depositional environments such as point bar, intradelta, natural levee, beach, and nearshore gulf are generally connected hydraulically to the adjacent water body (i.e. river, lake, distributary channel) and the groundwater level in these deposits generally reflects the level/stage of the adjacent water body. This is especially true in deposits adjacent to the Mississippi and Atchafalaya Rivers.

Numerous deep regional aquifers exist in South Louisiana. The coastal lowlands aquifer system of Louisiana consists of alternating beds of sand, gravel, silt, and clay deposited under fluvial, deltaic, and marine conditions. The aquifer system is comprised of sediment from late Oligocene age to Holocene that thicken and dip toward the Gulf Coast. The sediments are highly heterogeneous with sand beds that are not traceable for more than a few miles. The Chicot aquifer underlies most of southwestern Louisiana and extends from central southwestern Louisiana to the Gulf of Mexico and from Sabine Lake to St. Mary Parish. The Chicot aquifer is up to 800 feet thick at its most northern extent and extends to an unknown depth beneath the Gulf of Mexico. The Southeastern Louisiana aquifer system, also known as the Southern Hills aquifer system, consists of about 30 named aquifers. The Southeastern aquifer extends approximately from the Mississippi River to the Pearl River in Louisiana. The aquifers range in thickness from 50 to 1100 feet with thickness increasing toward the south.

Relative Subsidence

The entire coastal zone is experiencing relative subsidence. Relative subsidence is defined here as the net effect of numerous processes that result in the downward displacement of the land surface relative to sea level. Relative subsidence is controlled by several factors which

include eustatic sea level, geosynclinal downwarping, compaction of Holocene deposits, and faulting. Recent studies have shown that subsurface fluid withdrawal (oil, gas, water) may also be a major contributor to relative subsidence and resulting wetland loss (Morton et al, 2002). An important man-made contributor to relative subsidence is drainage for agriculture, flood protection, and development.

Eustatic sea level refers to the global fluctuations in sea level primarily due to changes in the volume of major ice caps and glaciers, and expansion or contraction of seawater in response to temperature changes. Recent studies have predicted an increase in the rate of eustatic rise due to global warming (IPCC, 2001).

Downwarping of the Gulf Coast Geosyncline accounts for a small percentage of the observed relative subsidence in coastal Louisiana (Kolb and Van Lopik, 1958). For millions of years, fine sediments have been deposited along the continental margin downwarping the basement and creating a gradually subsiding trough. The downwarping continues as new sediments are added to the basin.

Compaction of Holocene deposits has been considered the primary contributor to relative subsidence in the coastal plain. The three major components of Holocene sediment compaction include: 1) primary consolidation, 2) secondary compression, and 3) oxidation of organic matter (Terzaghi, 1943; Roberts, 1985). Primary consolidation occurs as the volume of the soil mass is reduced due to dewatering under a sustained load. Secondary compression results from a decrease in soil volume due to rearrangement of the internal soil structure. Oxidation of organic matter through chemical reactions reduces the soil volume.

Compaction of Holocene sediments varies widely throughout the coastal zone and is closely linked to the thickness and age of deposits. Fine-grained deposits with high water contents characterize the coastal zone. Where the thicker deposits are found, there is more interstitial water available for removal, which leads to high rates of relative subsidence as they compact. Older deposits have already undergone most of the primary consolidation and secondary compression and therefore exhibit lower relative subsidence rates than recently deposited sediments. The age, thickness, and to some extent the type of deposits are responsible for much of the variability in relative subsidence rates across the coast.

Movement on the downthrown side of deep-seated fault blocks is a well documented process in coastal Louisiana. However, their effects on the shallow subsurface and surface are poorly understood. Recent investigations (Gagliano, 2005) have identified likely areas of fault-induced subsidence but the magnitude and spatial extent of their impact are still being investigated. The Baton Rouge fault is probably the best known example of an active fault that has caused some structural damage. This fault crosses the PU1 barrier alignment approximately 6 miles south of Slidell. Most fault planes in coastal Louisiana generally trend east-west and may contribute to increased maintenance at the location where they intersect the alternatives. A minor amount of movement along fault planes can have significant impacts on wetlands where marsh accretion barely exceeds relative subsidence. Figure 8 shows the location of major fault trends of south Louisiana. A more detailed

analysis of faulting and the potential for other seismic activity will be conducted during the next design phase of the project.

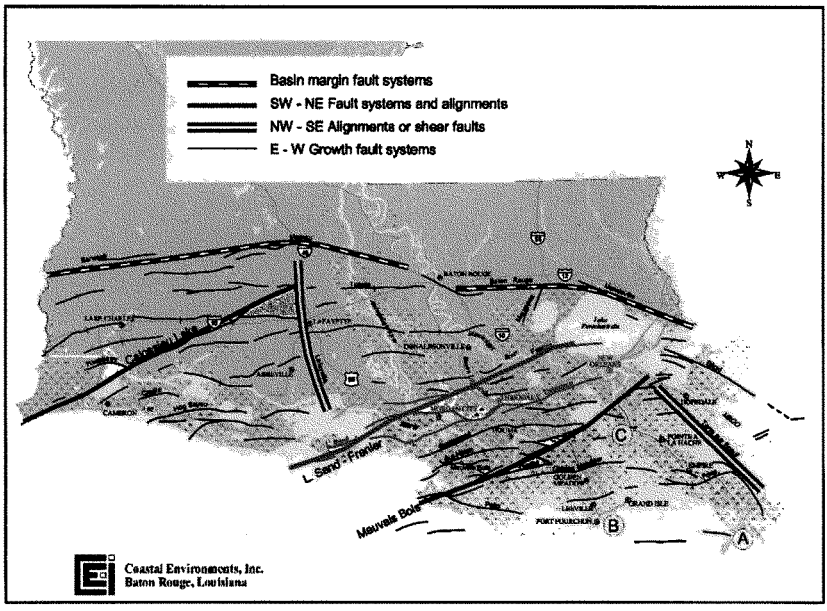


Figure 8 - Major Fault Trends Of South Louisiana

Forced drainage of wetlands results in lowering of the water table resulting in accelerated compaction and oxidation of organic material. Areas under forced drainage can be found throughout coastal Louisiana.

Relative subsidence rates vary considerably across coastal Louisiana. In general, natural rates of relative subsidence are highest near the coast and at the mouth of the Mississippi River where young thick sediments are present. Reported rates range from less than 0.5 feet to over 5.0 feet per century. Relative subsidence rates of several feet have been documented in developed areas of Jefferson and Orleans parishes due mainly to forced drainage. In addition, large areas of coastal land loss are found associated with failed land reclamation projects. Currently, no coastwide system for quantifying and predicting relative subsidence on a regional scale has been established. Therefore, relative subsidence rates are generally estimated using a combination of benchmark leveling, tide gauge measurements, and radiometric dating of buried marsh horizons.

Borrow Sources

Large volumes of clay, silt, and sand will be required for construction of the LACPR alternatives. Because the planning area extends across the entire State, numerous borrow sources will be required. Portions of the levee material will be available adjacent to the new levees but some may need to be hauled from distances of up to 60 miles away. Potential silt and sand sources include the Mississippi, Atchafalaya, and Vermilion Rivers, as well as tidal deltas and offshore shoals in the Gulf of Mexico. Sources of silt and clay include natural levee deposits of past and present Mississippi River courses and Pleistocene terrace deposits.

GEOTECHNICAL DESIGN

General

The large scope of work (from Sabine River to the Pearl River) necessitated a generalized assessment of the geologic foundation conditions in the planning area. The alignment was divided into four similar reaches or areas where the soil layering (stratigraphy) remains relatively consistent (see plates 1 and 2 for the location of these reaches). Soil unit weights and shear strengths of the strata were assigned to each of the four reaches based upon geotechnical engineering experience in the region with various projects in the vicinity. The objective to provide comparisons for three different levels of hurricane risk reduction levees and two different methods of levee design was achieved. Certainly, more detailed designs based upon site-specific foundation conditions will result in cost advantages in some reaches and cost disadvantages in others; however, the level of geotechnical reliability will be greatly enhanced. These site-specific investigations and designs will be conducted during the next phases.

Foundation Design Reaches

The three levels of risk reduction analyzed were for El. +25, +30, and +40. These elevations were selected based on preliminary modeling available at the start of design. It was assumed that the range of elevations covered by these designs (25' to 40') would include most elevations needed for the hurricane risk reduction alternatives. These designs were used to develop costs which were interpolated/extrapolated to determine costs for individual portions of project alternatives depending on the required levee design elevation. Existing ground for all reaches was assumed to be zero for all alternatives for the purpose of foundation design. Reach 1 extended from the Lake Pontchartrain south shore at the Rigolets through Orleans, St. Bernard, Plaquemine, Jefferson, and Lafourche Parishes to Larose, Louisiana and consisted of a 30-foot Marsh deposit and a 30-foot Interdistributary deposit overlying a 60-foot Prodelta deposit and then Pleistocene clay. This reach consisted of the worst soils with regards to shear strength and settlement potential. Reach 2 extended from Larose through Terrebone Parish to Gibson, Louisiana and consisted of a 10-foot Marsh deposit, a 20-foot Interdistributary deposit, a 90-foot swamp deposit and then substratum sands. Reach 2 was considered to be slightly better soils than Reach 1 with regards to shear strength and settlement potential. Reach 3 extended from Gibson through St. Mary and Iberia Parishes to

the Cote Blanche Salt Dome and consisted of a 15-foot Marsh deposit and then an 85-foot swamp deposit overlying substratum sands. Reach 3 was considered to be slightly better soils that Reach 2 with regards to shear strength and settlement potential. Reach 4 extended from Cote Blanche through Vermilion, Cameron, and Calcasieu Parishes to the Sabine River and consisted of a 10-foot Marsh deposit overlying Pleistocene clay. Reach 4 also was assumed to encompass from the north shore of Lake Pontchartrain to the Pearl River near Slidell, Louisiana due to similar foundation conditions as in southwest Louisiana. Reach 4 consisted of the best soils with regards to shear strength and settlement potential.

Typical Design Sections

The two methods of levee design considered for each reach were soil-cement columns and geotextile-reinforced embankment. See figures 9 and 10 and plates L-1 thru L-24 for typical sections for each reach. Each of these methods produces reduced berms and less levee footprint than a levee without these improvements. While the geotextile-reinforced embankment produces reduced berms and footprints by reinforcing the embankment, the soil-cement columns actually improve the foundation soils below the levee footprint by the deep mixing of cement in a series of "columns" and drastically improves the shear strength and compressibility of the clay soils.

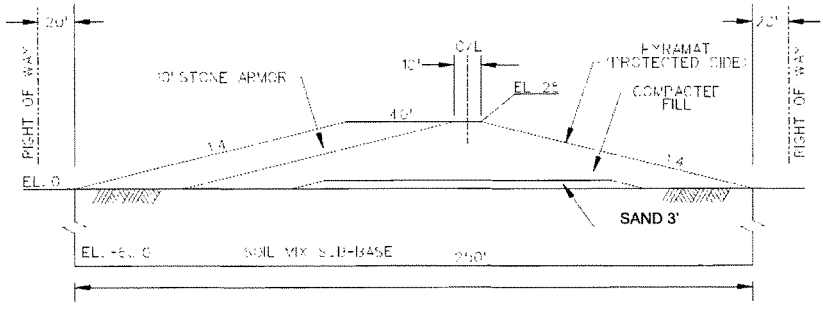


Figure 9 - Typical Soil Cement Section (Clay Fill)

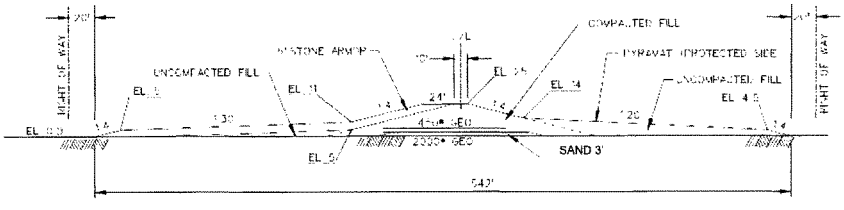


Figure 10 - Typical Geotextile-Reinforced Embankment Section (Clay Fill)

Deep Mixing Technology would be used to inject a binder mixture into the upper soft to very soft soils. In all reaches except Reach 4, a 60-foot depth column was used, while in Reach 4 the depth was reduced to 10-feet due to the presence of shallow Pleistocene clays at this depth. In this case, Type I Portland cement would be used as the binder, although some additives, such as fly ash or lime, could be considered.

Two Deep Mixing processes are commonly used. In the Dry Mix process, the binder mixture is injected into the soil pneumatically through the mixing tool. The natural moisture content of the soil is used in this process to hydrate the cement, which gives the improved foundation its more competent characteristics. In the Wet Mix process, the end result is the same, but the method of mixing the binder with the soil is somewhat different. The binder is first mixed with water and then injected hydraulically. This creates a slurry column, which cures in a way similar to that from the Dry Mix process. Each method has advantages and disadvantages, but either would work for this application.

The costs are approximately the same for the two methods. The dry method requires a much lighter rig, but is limited in depth to approximately 75-feet. The wet method requires less torque, can improve the foundation to depths of approximately 150-feet, and is much quicker, but produces about 20% of the column volume as excess material on the site.

For all reaches and levels of levee embankment, 30% of the foundation soil would be replaced with soil cement columns. The columns would be approximately 2.6-feet in diameter and would be placed in an overlapping pattern in rows 7-feet on center. The rows would extend from the protected-side levee toe to the flood-side toe of the armored rock. Where berms are present and extend beyond the toe, the soil-cement rows would not continue past where the main levee slope on the protected-side or the armored rock slope on the flood-side would have intersected the natural ground if no berms were present. The columns would be installed perpendicular to the levee centerline to serve as hard clay shear walls, and would achieve shear strength of approximately 6,000-psf. The equivalent shear strength of the soil-column matrix beneath the levee would be 2,000-psf for a 30% replacement ratio.

Immediately after the soil cement columns are installed, the compacted earthen levee could be constructed by conventional means. Construction lifts would have to be sequenced during the initial 28 days curing time to prevent overloading the columns before they reach their full design strength. This would be dictated by the site-specific in-situ foundation conditions. After 28 days curing time, the levee could be raised to its full height. A layer of geotextile varying in strength from 1460 to 1640 pounds-per-inch at 5% strain (depending upon the reach) is required for internal stability within the levee embankment only for the El. +40 alternative.

Flood-side slope protection was designed by the Coastal Structures Group of ERDC and consisted of varying thicknesses of rock from the levee crown down to the levee or berm toe. Details of this design can be found in Annex 4 of the Engineering Investigations enclosure to the LACPR Preliminary Technical Report. The rock thickness, since it was considerable, was taken into account for all stability analyses. Although its thickness added to the active

loading near the levee crown, it helped the levee stability in the flood-side berms. Therefore, construction of the levee will be staged, such that the levee embankment will be limited to a certain elevation (to be determined during detailed design) until the rock is placed in the flood-side berm, then levee embankment will be allowed to proceed to the final levee grade. Thickness of rock varied from 6- to 7-feet for the geotextile-reinforced levee alternative to 8- to 10-feet for the soil-cement levee alternative. The amount of rock cover is dependent upon the levee and berm slopes; the flatter the slope, the smaller the amount of rock required for protection due the wave-breaking effects of the longer slope. A separator fabric is also recommended between the rock armoring and the levee fill to prevent migration of rock into the fill material and loss of embankment material from wave action. Overtopping protection would also be provided on the crown and protected-side levee slope but was not considered significant enough to include in the stability analyses due to its limited height and extent over the levee. Armoring requirements are being reanalyzed as part of the ongoing Hurricane and Storm Damage Risk Reduction System work and designs will be revised during the next phase of design based on the results of this work.

For both alternative levee designs for all reaches, a three-foot sand base was included to provide a base for construction activities. The sand base was limited in extent such that 10-feet of clay cover would be provided above the sand due to seepage concerns. Compacted clay fill was assumed for all levee and berm material. This will provide the necessary strength for the levee embankment height and for the berm to support the rock armoring protection layer.

The length of geotextile required for each section varied from 75- to 320-feet for the geotextile-reinforced alternative. The required strength at 5% strain of this geotextile varied from 200- to 2,020-pounds-per-inch. In some sections for both geotextile-reinforced levees and soil-cement columns, geotextile reinforcement is required to stabilize the flood-side berm. The length of geotextile required for some sections of the soil-cement alternative varied from 40- to 190-feet, and the required strength at 5% strain of this geotextile varied from 200- to 1,700-pounds-per-inch. This geotextile reinforcement was required for only the internal stability within the levee embankment for only the El. +40 alternative and for the flood-side berm.

All slope stability analyses were performed using the Method of Planes. The flood-side designs provided for a minimum design factor of safety of 1.3. The protected-side designs were evaluated for two water loadings. For the extreme case of water to the levee crown, a minimum design factor of safety of 1.2 was required. For the "operating" case with the water level 10-feet below the levee crown, a minimum design factor of safety of 1.3 was required. The stability analyses for the geotextile alternative are shown on plates G1 to G16 and the stability analyses for the soil-cement alternative are shown on plates G17 to G40 for each of the four reaches and each of the three levee elevations. Only the controlling water load case is shown for the protected-side stability plates.

The Tables 11 to 13 show the required levee footprint for each of the three levee crowns considered. These footprint distances do not include any adjacent borrow pit requirements that may be utilized in areas where feasible and economical. The geotextile-reinforced

alternative was not considered practical for the levee El. +40 in Reaches 1 and 2, given the excessive levee footprint of ~1,300-feet required in Reach 3. For levee crown El. +40, the levee required footprints range from 1,040- to 1,280-feet for the geotextile-reinforced alternative and range from 450- to 800-feet for the soil-cement alternative. The required levee footprints for the geotextile-reinforced alternative are 2.3 times larger than the soil-cement alternative.

Table 11
Levee Footprints El. +40

Crown El. +40 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	--	800
2	--	650
3	1280	550
4	1040	450

For levee crown El. +30, the required levee footprints range from 580- to 1180-feet for the geotextile-reinforced alternative and range from 290- to 440-feet for the soil-cement alternative. The required levee footprints for the geotextile-reinforced alternative range from 2 to 2.7 times larger than the soil-cement alternative.

Table 12
Levee Footprints El. +30

Crown El. +30 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	1180	440
2	760	340
3	720	290
4	580	290

For levee crown El. +25, the required levee footprints range from 440- to 830-feet for the geotextile-reinforced alternative and are 250-feet for each of the soil-cement alternatives. The required levee footprints for the geotextile-reinforced alternative range from 1.7 to 3.3 times larger than the soil-cement alternative.

Table 13 Levee Footprints El. +25		
Crown El. +25 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	830	250
2	540	250
3	520	250
4	440	250

For all levee sections, both geotextile-reinforced and soil-cement alternatives, thru – levee seepage will not be an issue since levee embankment material will consist of acceptable clay material. When more site-specific geotechnical data are available, more detailed underseepage analyses will be conducted. These data include minimal areas where sand may exist at shallow levels underneath the proposed levee. In those areas where required, seepage berms or relief wells would be considered to provide adequate factors of safety.

Settlement

Foundation settlement is a key component of the estimated levee construction cost, particularly for the geotextile-reinforced levee alternative, which would require numerous lifts to construct to the project design grade. Consolidation and lateral spread of the soft clays in the foundation would produce substantial settlement of the levee foundation. The settlement analyses considered ultimate consolidation settlement based upon empirical values relating liquid limits to the compression index. Additionally, lateral spread was assumed to be 25% of the ultimate consolidation values for the geotextile alternative and 15% for the soil-cement column alternative. Shrinkage settlement was assumed to be 10% of the fill height. The sum of these three values provides an estimate of the ultimate settlement of the levee. For the soil-cement column levee alternative no settlement was assumed within the soil-cement columns themselves, however, settlement in the fill above the columns and the foundations soils below the columns is anticipated and estimated. Tables 14 to 16 summarize the estimated ultimate settlement of the proposed levee.

Table 14 Ultimate Settlement Estimate El. +40		
Crown El. +40 Reach	Geotextile Reinforced Levee ft.	Soil-Cement Column Levee ft.
1	-------	10.6
2	-------	11.6
3	19.7	7.3
4	9.3	5.0

Table 15 Ultimate Settlement Estimate El. +30		
Crown El. +30 Reach	Geotextile Reinforced Levee ft.	Soil-Cement Column Levee ft.
1	20.4	8.3
2	17.4	9.1
3	16.3	5.8
4	7.7	3.8

Table 16 Ultimate Settlement Estimate El. +25		
Crown El. +25 Reach	Geotextile Reinforced Levee ft.	Soil-Cement Column Levee ft.
1	17.8	7.2
2	14.7	7.8
3	14.3	5.0
4	6.8	3.2

The geotextile-reinforced alternative produced settlements ranging from 1.9 to 2.9 times larger than those produced with the soil-cement column alternative.

An empirical relationship relating liquid-limit values to coefficient of consolidation (U.S. Navy, 1971) was used to estimate a hypothetical lift construction schedule for the soil-cement alternative. Table 17 summarizes the results of this estimation:

Table 17 **Soil-Cement Column Alternative Settlement - Lift Requirements**			
Reach	El. +40	El. +30	El. +25
1	2 - 6' Lifts @ t= 6 & 20 yrs.	1 - 7' Lift @ t= 9 yrs.	1 -6' Lift @ t = 11 yrs.
2	2 - 6' Lifts @ t= 5 & 20 yrs.	1 - 7' Lift @ t= 6 yrs.	1 - 7' Lift @ t= 7 yrs.
3	1 - 7' Lift @ t= 7 yrs.	1 - 6' Lift @ t= 8 yrs.	1 - 5' Lift @ t= 12 yrs.
4	1 - 5' Lift @ t= 9 yrs.	1 - 4' Lift @ t= 12 yrs.	1 - 3' Lift @ t= 12 yrs.

Reaches 1 and 2 with crown El. +40 are the only ones that require two subsequent lifts. All other reaches and elevations would only require one subsequent lift. Lift heights vary from 3-feet in Reach 4 (El. +25) to 7-feet for Reaches 1 thru 3 at various elevations. The time to

first lift varies from 5-years for Reach 2 (El. +40) to 12-years for Reaches 3 and 4 at various elevations.

Tables 18 and 19 show the hypothetical lift construction schedule for the geotextile alternative.

Table 18
Geotextile Reinforced Levee Alternative Settlement - Lift Requirements

Reach	El. +40	El. +30	El. +25
1	N/A	1 - 6' Lift @ t= 1/2 yrs. 2 - 7' Lifts @ t= 3.5 & 14 yrs 1 - 5' Lift @ t= 39 yrs.	1 - 6' Lift @ t= 2 yrs. 2 - 7' Lifts @ t= 8 & 34 yrs.
2	N/A	1 - 6' Lift @ t= 2 yrs. 2 - 7' Lifts @ t= 8 & 34 yrs.	2 - 6' Lift @ t= 2 & 19 yrs. 1 - 5' Lift @ t= 38 yrs.
3	1 - 6' Lift @ t= 1/2 yrs. 2 - 7' Lifts @ t= 3.5 & 16 yrs 1 - 4' Lift @ t= 40 yrs.	1 - 6' Lift @ t= 1 yrs. 2 - 7' Lifts @ t= 8 & 34 yrs	2 - 6' Lift @ t= 4 & 14 yrs. 1 - 5' Lift @ t= 35 yrs
4	1 - 6' Lift @ t= 2 yrs. 1 - 7' Lift @ t= 19 yrs.	1 - 6' Lift @ t= 3 yrs. 1 - 3' Lift @ t= 25 yrs.	1 - 5' Lift @ t= 6 yrs. 1 - 3' Lift @ t= 27 yrs.

Table 19
Geotextile Reinforced Levee Alternative Settlement - Lift Requirements

Reach	El. +20	El. +15
1	1 – 5' Lift @ t= 3 yrs. 2 – 7' Lifts @ t= 15 & 35 yrs.	1 - 5' Lift @ t= 4 yrs. 1 - 6' Lift @ t= 15 yrs 1 - 5' Lift @ t= 35 yrs.
2	1 – 4' Lift @ t= 4 yrs. 2 – 5' Lifts @ t= 18 & 36 yrs.	1 - 4' Lift @ t= 4 yrs. 2 – 3.5' Lifts @ t= 18 & 36 yrs.

In areas where a straddle enlargement of an existing levee and berm is possible, the settlement is overestimated since it is based on zero initial ground surface elevation. Therefore, the number of lifts and the quantity of levee fill material described in this report is overestimated for those areas.

Overtopping Weir Section

An additional section was developed for the overtopping weir in PU1 and PU2. This section is shown on plate L-25 and described under the levee design section of this appendix.

Hollow Concrete Levee

As part of the ongoing hurricane protection work, as well as the LACPR effort, an evaluation of a hollow concrete levee concept was undertaken. A description of this effort is given in the Innovative Designs Section of this report.

Additional Investigations

The following additional work will be done during detailed design : 1) increase geotechnical reliability with additional borings and CPTs along the proposed alignment, 2) study utilizing higher replacement ratio for soil-cement columns to alleviate berms for all reaches for El. +40 and for Reaches 1 and 2 for El. +30, 3) study the feasibility of adjacent borrow for more economical levee construction and effects upon the levee footprint, 4) perform underseepage analyses in areas where shallow sands may exist to determine if seepage berms, relief wells or other measures are required to reduce seepage concerns, 5) reevaluate settlement for areas where the proposed levee will straddle an existing levee of substantial grade, 6) reevaluate settlement to investigate whether soil-cement levee sections could be economically overbuilt to avoid lift construction, and 7) further investigate different methods of levee construction including investigating the use of different fill materials, investigating the use of aggregate filter stone and performing a more detailed analysis of the concrete levee concept . Costs for these efforts have been included in the E&D portion of the cost estimates.

LEVEE DESIGN

GENERAL

The five planning units were broken down into reaches and sections for ease in comparing alternatives. These sections are shown on plates 1 and 2. Existing LIDAR information was used to develop quantities for both the typical geotextile-reinforced and soil-cement column improvement sections for each reach. Quantities were developed for elevation 25 ft, 30 ft and 40 ft levees.

FLOOD PROTECTION ALTERNATIVES

For this effort, two typical levee sections were considered. The first is a conventional earthen levee utilizing geotextile layers for load distribution and reinforcement for stability. The second is also an earthen levee design, but built upon an improved foundation of soil-cement mix columns. Plates L-1 thru L-24 show typical sections for each reach and elevation.

Geotextile Alternative

The first alternative is a clay levee with geotextile reinforcement located in the center just above the base in two layers. The design levee section has 1V on 4H side slopes and a 10' wide crown. The design heights are 25', 30' and 40'. Floodside and landside stability berms

are required to provide stability due to the soft foundation. The floodside slope and berm are protected from wave action by an 6' - 7' thick stone blanket. The levee crown, landside slope and the landside berm are protected from erosion with a high performance turf reinforcement mat anchored to the levee and berm designed to hold up under over topping conditions. For preliminary design purposes, the proprietary product Pyramat® was selected as representative of the required quality and performance characteristics. In the reaches where the levee is being constructed over wetlands a 3' sand base is placed down the center of the levee to provide a construction platform. Under the poorest foundation conditions the footprint widths for the 25', 30' and 40' high levees are approximately 830', 1180' and 1280' respectively. This alternative at the 40' design height is not practical to build in soils Reaches 1 and 2 due to severe settlement that would be experienced in the compressible soils.

Soil Mix Alternative

The second design alternative is a clay levee supported by cement soil columns. The columns would be approximately 2.6-feet in diameter and would be placed in an overlapping pattern in rows 7-feet on center. The depth of the columns for the best soils condition is 10' and in the poor soils it is 60'. The columns extend between the levee toes for levees without stability berms. Where there are stability berms the soil columns extend approximately 40' past the levee toes. The design section is also 1V on 4H with a 10' wide crown and the design heights are 25', 30' and 40'. On the poorest foundations stability berms are required for the 40' and 30' levees but for the 25' levee stability berms are not required. On the best foundation conditions stability berms are required for only the 40' levee and only on the protected side. The use of geotextile reinforcement is limited to a single layer located in the center just above the base of the levee and only for the 40' high levee. The flood side levee slopes and berm are protected by an 8' – 10' thick blanket of stone. On levees without flood side stability berms the stone blanket is increased to 10'. Under the poorest foundation conditions (reach 1) the footprint widths for the 25', 30' and 40' high levee are approximately 250', 440' and 800'. The levee crown, landside slope and the landside berm are also protected from over topping with a high performance turf reinforcement mat.

The levee alignment along the south shore of Lake Pontchartrain in Jefferson and Orleans Parish requires the levee foot print and flood side berms to extend out over the water. In such conditions only sand is used in the berms under the rock armor. In addition a sand working platform is constructed to above the lake level under the levee.

Levee Design for Overflow and Overtopping

This levee alternative is another form of the geotextile alternative constructed to an elevation of 12.0' to facilitate overflow and overtopping by the storm surge and temporary retention of the storm surge. The geotextile reinforcement is still located in the center just above the sand base. The design levee section has 1V on 4H side slopes and a 10' wide crown. The floodside and landside stability berms are required to provide stability due to the soft foundation. The floodside levee slope is protected from wave action by a 7' thick stone blanket with a 5' thick rock berm extending 25' beyond the armor blanket. This rock berm serves as a combination stability and wave berm. The landside of the levee has a 5' thick

rock berm extending 70 feet from its intersection with the levee landside slope. This berm also serves as a landside stability berm and as armor for the overflow and overtopping. The levee crown and landside levee slope above the landside rock is protected from erosion with a high performance turf reinforcement mat anchored to the levee designed to hold up under overflow and overtopping conditions. For preliminary design and cost purposes, the proprietary product Pyramat® was selected as representative of the required quality and performance characteristics. Investigations have been initiated as part of the current storm damage reduction system work to determine what is the best method for protection from overtopping and erosion. The results of these studies will be incorporated during detailed design. A typical section of this levee section is shown on plate L-25. This levee section will be used in two reaches. The first is across the opening between Lake Pontchartrain and Lake Borgne. The second reach extends along the south side of the GIWW between Larose and Belle Chasse.

CONSTRUCTION CONDITIONS

For this effort, the planning area is divided into four soils reaches with Reach 1 having the poorest foundation condition and improving through Reach 4, which has the best foundation condition. Foundation conditions generally improve from east to west across the coastal area of south Louisiana from the Mississippi River delta to the Sabine River. Soil conditions are generally better in southwest Louisiana, where the underlying, firmer Pleistocene soils are found closer to the surface.

Another factor influencing construction conditions is the location of levee design alignments between wetlands and land above sea level, either distributary ridges or the Pleistocene prairies that are found in Reach 4. The largest portion of proposed levee alignments falls in wetlands including parallel borrow pits. The second group of design alignments falls along the interface between wetlands and ridges or prairies, with the levee footprint falling in the wetlands and the parallel landside borrow areas falling on higher ground. For the final set of design alignments, both levee and borrow pits are on high ground.

Constructing levees completely founded in wetland requires a sand base along the levee centerline to serve as construction access and a dry working platform. This is typical of the levee alignments along the GIWW and south of the GIWW. In soils Reach 4 located west of New Iberia, the good foundation soils are close to the surface in the wetlands. For preliminary design, 80% of the required quantity of earthen levee material is assumed opposite cast with 20% hauled in from 30 miles away from the Pleistocene prairie. Proposed levees east of New Iberia (reach 3) located totally in wetlands have a lower percentage opposite cast: around 40% with 60% haul. East of Golden Meadow (reach 1) the percent goes to 20% opposite cast and 80% haul with haul distances of 50 to 60 miles round trip. Sand for construction access is assumed to be barged from the Atchafalaya River area or Mississippi River. Truck haul to construction sites from the barge to placement is short since waterways run alongside or near the levee alignments. Levee alignments east of Golden Meadow (reach 1) will use sand from the Mississippi River and those to the west (reach 2 and 3) from the Atchafalaya River area. Stone armor will be barged in from out of state and will follow for the most part the same access used for the sand levee base from the point of barge delivery.

Construction along the interface between wetland and highland has the levee located on wetland which requires the placement of a sand base for access and to provide a working platform. Most of the borrow falls alongside and parallel to the levee on the protected side which is dry. High percentages of opposite cast are assumed (80% to 90%). The balance required material is assumed truck hauled for a 20-mile round trip from the tributary ridges or Pleistocene prairie. Alignments in Plaquemines Parish (reach 1) have extremely limited local borrow sources resulting in an estimated round trip haul of 120 miles. The greatest portion of sand will be barged to the approximate area of the job site since few waterways are near these alignments. Truck haul from the barge drop off ranges from 20 to 80 miles round trip. Alignments near the Atchafalaya River (reach 3) and Mississippi River (reach 1) sand sources will use truck hauls with 10 to 20 mile rounds trips. Armor stone will be barged in from out of state utilizing access similar to sand, where the material is moved by barge to the approximate area of the construction site.

Levees constructed on Pleistocene soils found in Reach 4 have their alignments approximately following the 10' contour in the southwest part of the state. Favorable soil conditions classified as Reach 4 also occur at an additional short reach on the eastern end of the levee alignments on the north shore of Lake Pontchartrain. All of these levees will be constructed using 90% opposite cast and 10% truck haul with a round trip of 10 miles. Borrow pits are all located on Pleistocene soils. Sand is not required for construction access on these alignments. Armor stone for these alignments is truck hauled between 10 and 20 miles round trip from the nearest waterway which allows barge access.

AVAILABILITY OF BORROW

While assumptions have been made regarding the availability of local borrow for levee construction, this is a critical construction issue that warrants much closer analysis. The quantity of material required to build even the shortest of the proposed alignments will be in the hundreds of millions of cubic yards. Finding such a quantity of suitable material in south Louisiana would be difficult as well as environmentally undesirable. One alternative to reducing the need for clay borrow from wetlands is to reconfigure the levee design cross section to use more sand and at the same time maintaining a 10' to 15' clay cap over the sand to prevent wash out of the sand core. This will be further investigated during detailed designs.

STRUCTURES

GENERAL

No hurricane protection project could span coastal Louisiana without crossing dozens of navigable waterways, distributaries, urban drainage canals, and roadways. For the LACPR effort a set of typical structures were designed using a parametric approach to parallel the levee design effort. Details of these designs can be found in the report titled "Preliminary Technical Report for the Louisiana Coastal Protection and Restoration Study Structural Components" dated May 2006. With final grades unknown until hydraulic analyses were

completed, the team developed schematic designs for the necessary structures to assumed elevations of 30, 35 and 45 feet. These elevations were selected based on the results of preliminary modeling, assuming that the structures would need to be higher than the surrounding levees because of hydrodynamic design conditions. Invert elevations were also assumed based on existing surveys and other existing data.

Structures designed for LACPR generally fall into the following categories: sluice gate structures, sector gated structures, roller/swing gate structures, tainter gated structures, butterfly gate structures, locks, box culvert structures, new pumping stations, and hardening and fronting protection for existing pumping stations. A description of each type of structure is shown below. Plates S1 – S23 show typical sections and other details. Table 20 shows the number and type of structures identified for the evaluated alternatives in each planning unit. This listing does not including existing and new pumping stations.

Table 20
Structures Identified By Planning Unit

Planning Unit	Structure Type	Number of Structures
1	56' Sector Gate	9
	Tainter Gate	2
	Butterfly Gate	1
	110' Sector Gate	4
	Sluice Gate[2]	3
	Road and Railroad Gates	17
2	56' Sector Gate	2
	Tainter Gate	2
	110' Sector Gate	4
	Road and Railroad Gates	8
	110' Lock	1
3a	56' Sector Gate	16
	Tainter Gate	2
	110' Sector Gate	4
	Sluice Gate	12
	250' Sector Gate	2
	110' Lock	2
3b	56' Sector Gate	3
	110' Sector Gate	14
	Sluice Gate	6
4	56' Sector Gate	7
	Tainter Gate	1
	110' Sector Gate	5
	Sluice Gate	9

[2] Number of sluice gates does not include environmental structures.

SLUICE GATE STRUCTURES

Drainage structures with sluice gates will provide drainage through the flood protection at various locations within the planning area (see plate S-7). Each structure will consist of a pile founded, reinforced concrete structure with trash screens, operating platforms, and provisions for dewatering. A 24-foot wide roadway will provide access across the structure. The sluice gate structures will connect into the existing flood protection on each side of the structure with a T-wall. The concept for these structures was taken from the 2003 Morganza to the Gulf of Mexico, Bayou Grand Caillou Alternative Study. The sluice gates will have the capability to be operated manually or will be mechanically actuated with portable motors.

Three different structures of this type are included. One will have 4 - 10' x 10' sluice gates, an invert of El. –12.0 and will be utilized at several locations along the alignment. A second structure of this type will have 4 or 5 – 8' x 10' gates and an invert at El. –8.0. This structure will be utilized at locations where the intersecting waterways that are smaller and shallower. These structures are also provided at locations where previous studies envisioned the use of multi-barreled box culverts. Given the levee sizes considered for LACPR, a pile founded, sluice gate structure appears to be much more practical and cost effective than long box culverts under wide and tall levees. A third type of sluice gate structure will be provided where only a few small openings are required to pass drainage flows. This structure will be a modified, inverted T-wall monolith with 5' x 5' surface mounted gates and an invert at El. –5.0. The number of gates at each site will be based on the flow requirements.

A detailed load analysis, considering both vertical and horizontal loads, was performed on each of the typical sluice gate structures for each of the three levels of risk reduction. For the T-type structure, the analysis considered an inverted T-wall structure with the top of the base at El. –5.0 and an increased wall depth to accommodate the gated openings at the base of the wall.

A foundation plan was developed for these structures using the USACE program CPGA (X0080) and the calculated vertical and horizontal loads. Typical pile capacities were assumed to be in excess of 100 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of analysis. Minor overstresses were allowed in the foundation piling.

SECTOR GATE STRUCTURES

Sector gated structures will provide flood protection (closure) during storm events while allowing normal navigation at many of the waterways intersecting the flood protection alignment (see plates S-8 to S-15). These structures were sized based on the apparent width of the existing waterway. The sill elevation at each location was selected based on the prevailing bottom elevation at the site and is typically either El. –9.0 or El. –15.0. Standard sector gate widths of either 56 feet or 110 feet were used. A special structure that is 250 feet wide with a deeper sill will be provided at the Houma Navigation Canal where larger vessels are anticipated. Each sector gate structure will be a pile founded, reinforced concrete

structure at the required sill elevation and width to maintain navigation in the waterway. Double sector gates were provided at waterways with the highest levels of navigation and/or at major waterways where the number of vessels seeking safe harbor would be the highest (see plates S-16 to S-17). The double-gated structure will accommodate the higher traffic volume in either case. The structure will have emergency and/or maintenance stop logs and separate control houses on each wall. A timber guidewall with a protective cellular dolphin at the end will be provided on both sides of each approach channel to the structure.

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith for several of the required sector gate structures. The detailed analyses considered two of the three levels of risk reduction. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads. Given the scope of LACPR, minor over-stresses were allowed in the foundation piling. Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0.

SPECIAL SECTOR GATE STRUCTURES

Special, 250-foot wide sector gated structures are assumed as part of the closure and flood protection at the Houma Navigation Canal (HNC) waterway to provide for the navigation needs in these waterways. The sill elevation was selected based on the prevailing bottom elevation at the site, El. –20.0 at the HNC. Each sector gate structure will be a pile founded, reinforced concrete structure at the required sill elevation with the appropriate width to maintain navigation in the waterway. The structure will have emergency and/or maintenance stop logs and separate control houses on each wall. Due to the nature of the navigation in these waterways, a cellular, concrete guidewall with protective dolphins at the end will be provided on both sides of each approach channel. These guidewalls will also be longer than those provided at the typical sector gated structure. At these locations, the overall depth of the structure will necessitate the use of adjacent, non-overflow sections until the channel slope becomes high enough to use conventional T-walls for the tie-in to the adjacent levees. At the deeper structures, with sill elevations at or below El. –20.0, the use of floodwalls is not adequate to "span" between the deep primary structure and the shallower foundation for the floodwall structure. Given their overall height, these structures must resist lateral loads nearly as high as the primary structure. The non–overflow monoliths are substantial structures with wide bases to bridge the deeper sections of the channel and/or excavation until the use of the standard floodwalls was deemed appropriate.

The main structure is envisioned as a single segment to be floated into position, however the adjacent non-overflow sections and the tie-in floodwalls were considered to be floated into position in separate phases after the navigation structure has been completed enough to allow for some minimal navigation. The basis for this method of construction is the construction plan for the Inner Harbor Navigation Canal (IHNC) Lock Replacement.

A detailed load analysis, considering both vertical and horizontal loads, was performed on the structure required for an El. 45.0 level of protection. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads.

Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0.

TAINTER GATE STRUCTURES

Tainter gated structures will be provided as part of the closure and flood protection at the larger waterways (see plate S-18). These structures were sized based on the available flow area in the existing waterway. The sill elevation at each location was selected based on the prevailing bottom elevation at the site. Tainter gate widths of either 50 feet or 63 feet were used in this study. The gate radius was determined using the required sill elevation at the anticipated location of the trunnion girder. The basic layout for the tainter gate structures will be similar to the Old River Auxiliary Control Structure and the Red River Waterway dams. Each tainter gated structure will be a pile founded, reinforced concrete structure at the required sill elevation with the appropriate number of steel tainter gates with post-tensioned anchorages to maintain the flow conditions in the waterway. A two-lane highway bridge will be incorporated into the structure for access and maintenance. The structure will have emergency and/or maintenance stop logs, an exposed cantilevered machinery platform and four cellular dolphins to serve as guardwalls for the structure. The location and size of the tainter gated structures are shown in Table 21.

TABLE 21
SUMMARY – TAINTER GATED STRUCTURES

STRUCTURE LOCATION	PLANNING UNIT	SILL ELEVATION	GATE WIDTH	NUMBER OF GATES
Bayou Grand Caillou	3a	-12.0	50 feet	4
Bayou Perot	2	-16.0	50 feet	16
Mermentau River	4	-12.0	50 feet	5
Vermillion River	3b	-12.0	50 feet	4
Calcasieu River	4	-25.0	63 feet	8
Chef Pass	1	-25.0	63 feet	12
Rigolets Pass	1	-30.0	63 feet	15

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith for several of the required tainter gate structures. The detailed analyses considered two of the three levels of risk reduction. A foundation plan was developed for these structures using the USACE program CPGA (X0080) and the calculated vertical and horizontal loads. Minor over-stresses were allowed in the foundation piling. Typical pile capacities were assumed to be in excess of 110 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0.

BUTTERFLY GATE STRUCTURE

A butterfly-gated structure will be provided at the Rigolets Pass as part of the closure complex for Lake Pontchartrain at this location (see plate S-19). This structure was sized based on the available flow area in the Rigolets and is presently deemed necessary to allow for flows through the Rigolets for as long as possible. The sill elevation at the Rigolets was selected based on the prevailing bottom elevation in the channel. Butterfly gates open and close on their own in response to head and flow conditions due to the offset pintle location. The butterfly gate design allows for the automatic operation of the gates in case of a reversal in the direction of flows through the pass thus allowing for continual uninterrupted flow through the Rigolets Pass.

The tainter and butterfly gated structures at this location will be constructed separately so as not to cutoff flow through the pass. The structure will be constructed in the dry with a series of sheet pile cells for the cofferdam. The assumed number of gates across the pass was based upon a rough calculation of the end area of the pass at the nearby U.S. Hwy 90 bridge as presented in the 1973 Lake Pontchartrain and Vicinity Barrier Plan, Rigolets Control Structure and Closure Plan DDM. The concept for the butterfly-gated structure will be similar to that presented in the 1989 Lake Pontchartrain & Vicinity, High Level Plan, London Avenue Outfall Canal GDM.

The structure will be a pile founded, reinforced concrete structure at the required sill elevation with steel butterfly gates to maintain the flow conditions in the waterway. A two-lane highway bridge will be incorporated into the structure for access and maintenance. The structure will have emergency/maintenance stop logs and 4 cellular dolphins to protect the structure.

A typical section of a butterfly-gated structure is depicted on plate S-19.

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith with 2-40-foot wide bays and 12-foot wide divider piers. The detailed analyses considered two of the three levels of risk reduction. A foundation plan was developed for the structure using the USACE program CPGA (X0080) and the calculated vertical and horizontal loads. Typical pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0.

ROLLER/SWING GATE STRUCTURES

Preliminary design considerations indicated that the use of swing gates was not practical for the various gate sizes under consideration. The gate hinge loads for this type of gate would be quite excessive due to the large gate weights and, in addition, moving such a large gate in windy conditions was deemed impractical and dangerous. For these reasons, all required highway and rail crossing structures were evaluated as conventional roller gate structures with both a combined storage monolith and tie-in floodwall structures on each side to transition into the full levee section (see plates S-20 and S-21).

Each gate will be supported by a pile founded, reinforced concrete base and walls. A cursory load analysis, considering both vertical and horizontal loads, was performed on a typical roller gate structure for each of the three levels of risk reduction. A foundation plan for the gate and storage structure, using 24-inch diameter pipe piles, was developed for these structures using graphical methods. Standard T-wall monolith costs for each design level, based on the required levee transition lengths, were included in the cost estimates for each structure. Since these structures will be constructed at grade, no costs for cofferdams were included in the estimates for the gated structures. Costs for highway bypasses, traffic control and signage and for repaving were also included in the estimate. Costs for both 32 and 34-foot wide gate openings and sill elevations at +4.0 and +8.0 were determined. Where the standard 32-foot structure width and/or sill elevation varied, costs were estimated by using factors to revise the items affected by the changes. At locations where 40-foot wide gates were required at rail crossings, the costs for highway bypasses and paving items were deleted and costs for the added width and both track falsework, on two track lines, and required railroad insurance were added.

SECTOR GATED LOCK STRUCTURES

Sector gated lock structures will be provided as part of the closure and flood protection at almost all of the larger, navigable waterways (see plates S-22 to S-28). The sill elevation at each location was selected based on the prevailing bottom elevation at the site. A lock width of 110 feet was generally used in this study. The gate monoliths at each lock structure will be a pile founded, reinforced concrete structure at the required sill elevation. The floodside gate monolith will extend to the required study elevation and will also serve, along with the tie-in floodwalls, as the main line of protection. The lock walls for the protected side gate and the lock chamber will extend only up to El. 9.0. In general the chamber sections will be earthen with timber guidewalls with rock dikes and riprap for scour protection. At Amelia and Calcasieu the design assumes a pile founded concrete chamber section. Each gate structure will have emergency and/or maintenance stop logs and separate control houses to operate the gates. Timber guidewalls and cellular dolphins that serve to protect the guidewalls will be provided on both sides of each approach channel.

A detailed load analysis, considering both vertical and horizontal loads, was performed on each of the gate monoliths at each lock. The detailed analyses considered two of the three levels of risk reduction. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads. Given the scope of the study, minor over-stresses were allowed in the foundation piling. Typically, steel pipe piles were used for the taller, floodside gate monoliths while 14" square prestressed concrete piles were provided for the foundations for the protected side gate monolith. Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0.

INVERTED "T" TYPE WALLS

Each T-wall structure will consist of a wide concrete base with walls to the required study elevations and will be supported by either 24 or 30-inch diameter pipe piles of the length required to develop the appropriate pile capacity (see plate S-6). The use of 30-inch piles is

currently limited to the designs for protection to El. 45.0. A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical 60-foot long monolith for each of the three levels of risk reduction. Horizontal loadings considered included both water to the top of the wall and unbalanced soil loads applied at the base. The wall design considered an impact force of 250-kips, applied at the top of the wall and spread out over a 5-foot width at the point of impact. A more accurate assessment of vessel impact will be required for the design criteria at each structure location. Wave loads were not considered at this time, but the final design will incorporate these loads and, quite possibly, a re-curve type of wall to better distribute and minimize the required wave forces.

A foundation plan was developed for these structures using the USACE program CPGA (X0080) and the calculated vertical and horizontal loads. Given the scope of the study, minor over-stresses were allowed in the foundation pilings. Typical pile capacities were assumed to be in excess of 100 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0.

INNOVATIVE DESIGNS

HOLLOW CORE LEVEES

As part of the ongoing hurricane protection work, as well as the LACPR effort, an evaluation of a hollow concrete levee concept was undertaken. Details of this evaluation can be found in the report titled "Concrete Levee and Concrete Sliding Flood Gate Concept Evaluation" dated 30 March 2007. The concept of the hollow concrete levee system is such that the section fills with water from the bottom as the storm surge rises. The combined weight of the concrete frame and its water filled voids inside the frame result in a gravity structure that is designed to resist hydrostatic forces and impact forces from vessel collision.

The hollow concrete levees are comprised of trapezoidal shapes similar to that of earthen levees. The levee superstructure sections are comprised of sloped side walls with a flat bottom slab with access to the interior via steel grating or manholes in the crest. Water inlets or ports are incorporated into the cross section near the levee base on the flood side to allow the section to flood with water to contribute to the overall weight for stability purposes. Shear keys in the base were designed to protect against sliding under design loading conditions. The substructure consists of a concrete base slab or pad that will be supported by steel pipe piles. It is anticipated that excavation and granular backfill will be required to construct the pile supported concrete pad. The concrete base slab serves a two-fold purpose. It distributes loads to the pile foundations as well as serves as a "roadway" for cast-in-place construction. A typical section is shown in figure 11.

Table 22 shows a summary of the probable cost of construction and the construction duration. These costs do not include O&M, Real Estate, environmental impacts, wetland mitigations, or other site-specific considerations.

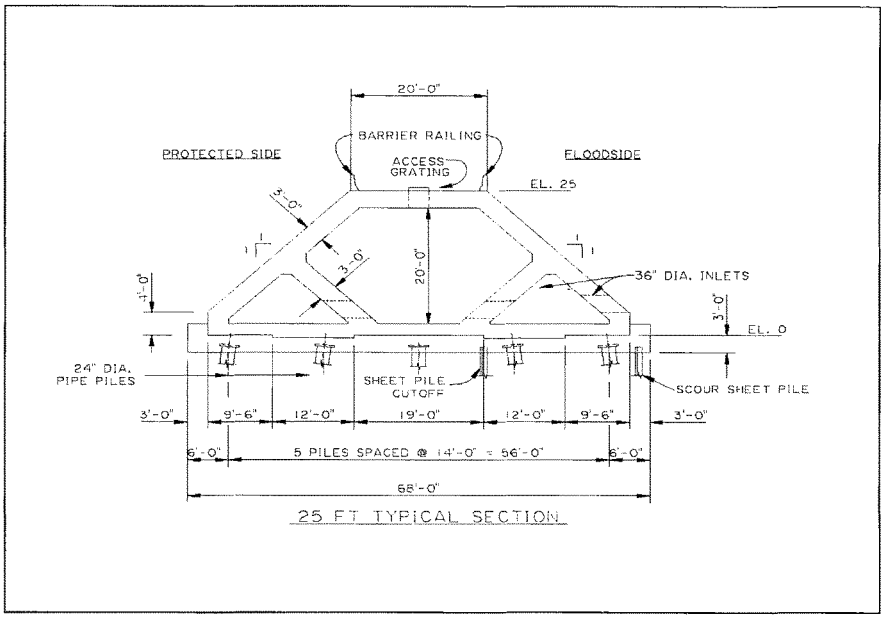


Figure 11 - Hollow Concrete Levee Section

Table 22 Summary of Concrete Levee Cost and Duration		
Description	**Cost**	**Duration**
Concrete Levee		
. 25' Crest Height	$287,485,000/mile	16 months/mile
. 30' Crest Height	$343,040,000/mile	17 months/mile
. 40' Crest Height	$510,576,000/mile	21 months/mile

COMPARISON OF BARRIER OPTIONS

A cost comparison was done among concrete levee, geotextile and soil mix levee sections. The comparison was done between costs per mile for sections in soil reach 1 which has the worst soil conditions. The results of this comparison are shown in table 23. Although for this comparison the total costs per mile are less for the soil mix section other factors should be taken into consideration when deciding which type of levee construction to use. The levee footprint of the concrete section is less than that of the soil mix or geotextile section. This can make a big difference in environmental impacts and real estate concerns. The initial construction time of the concrete section is shorter than the soil mix section. No additional lifts are needed for the concrete section. A disadvantage of the concrete section is that it

can't be easily raised to account for future conditions (relative sea level rise and degrading coastline). Because there is such a range in conditions along the proposed levee alignments it is anticipated that all of these levee types may be used for construction. During detailed design a decision will be made as to what type of levee construction is the best for each levee section based on location (environmental and real estate impacts, geotechnical conditions), required levee elevation, cost and any other pertinent factors.

Table 23
Cost and Footprint Comparison Among 30' Soil Mix, 30' Geotextile and 30' Concrete Levee

Type	Construction Cost ($/mile)*	Levee Footprint (ft)
30' Soil Mix	226,886,664	568
30' Geotextile	418,882,469	1,433
30' Concrete	343,040,000	228

*Costs include initial construction and future lifts

COASTAL RESTORATION FEATURES

GENERAL

Coastal restoration measures are included as part of the comprehensive protection plan. A group of coastal restoration measures were developed by the Habitiat Evaluation Team (HET). These measures were put together to develop alternative coastal restoration plans. The goal of these coastal restoration plans is to sustain the current coastline to maintain the current level of hurricane and storm damage risk reduction. Details of how these measures and alternatives were developed can be found in the Coastal Restoration Plan Component Appendix.

One of the alternative plans was selected as a representative plan for use in the Multi Criteria Decision Making Analysis (MCDA). Designs and costs were developed for those measures contained in the representative plan. Details of the design and cost of all coastal restoration features can be found in Annex 1 (Coastal Restoration Features).

MARSH CREATION AND RESTORATION

Designs and cost estimates were developed for marsh creation, ridge restoration, secondary barriers and shoreline protection measures. Cost estimates and implementation schedules were provided. Details of the design and cost estimates can be found in Annex 1.

BARRIER ISLANDS

Several studies have been and are being conducted on restoration of the Barataria and Terrebonne basin barrier islands. These include the ongoing Barataria Basin Barrier Island Feasibility Study. Existing costs and designs were used for where available and new designs and costs were developed when needed.

DIVERSIONS

Freshwater diversions are diversions designed to take river water and its accompanying sediment and divert it into the surrounding areas to create or enhance marsh. Designs and cost estimates were developed for the diversions included as part of the coastal component of the alternatives. See Annex 1 for additional details.

RELOCATIONS

GENERAL

Scope and Purpose

Relocation data was collected and tabulated by the U.S. Army Corps of Engineers, New Orleans District, Engineering Division, Relocations Section, for a feasibility level study. Relocations Section utilized the 1990 Louisiana Pipeline and Industrial Atlas, by Design Technics Corporation and information obtained through the Louisiana Department of Natural Resources, to identify oil and gas facilities located in the planning area. Also used were information gathered while working on previous feasibility studies and site visit information for other projects in the planning area. Relocations Section considered project design features and their effects on existing facilities to determine project relocation requirements. No contacts were made with facility owners.

Estimated Relocation Cost

Cost for relocation of pipelines, roadways, power and communication lines for the alternatives are shown in Annex 4. These amounts include 10% for owner's engineering and design, 8% for owner's contract administration, and 25% contingencies.

DESCRIPTION OF FACILITY RELOCATIONS

Method of Pipeline Relocation

Three methods of relocating affected pipelines were investigated:

(1) Placement of pipelines on the surface of the newly constructed levee.
(2) Construction of permanent pipeline bridges supported by pile founded piers.
(3) Directional drilling.

The conditions influencing the selection of a relocation method include the substantial settlement anticipated by the proposed hurricane protection levee alternatives addressed in this report. The cost of each method was investigated, analyzed and compared for the best method of relocation for each design alternative chosen. Other factors considered in

determining the best method include, initial cost, life cycle cost, maintenance, vandalism, potential impacts by future lifts and disruption of service.

Further discussion on the investigated methods of relocation follows:

Placement of Pipelines on the Levee Surface

This method of relocation is typically used for pipelines affected by main line Mississippi River and Atchafalaya Basin levees. Typically, a pipeline is installed on the levee net design section and follows the contour of the levee until it proceeds underground on the floodside and protected sides of the levee. Additional fill producing the gross grade is placed over the pipeline to provide protection. This method presented problems due to the anticipated settlement of levees on which pipelines would rest. Since the levee provides support for the pipeline, the anticipated levee settlement would result in undesirable stresses on the pipeline. The multiple lifts expected to meet project flood protection elevations also raised concerns about multiple relocations, a prospect which would not be welcomed by the pipeline owners. Although the cost for initial pipeline relocation by this method would probably be the least costly, future costs of subsequent adjustments, modifications, and relocations of the pipeline diminish this advantage.

Permanent Pipeline Bridge

This method of relocation involves installation of affected pipelines on bents supported by pile piers. The advantage of this method is that it diminishes the affects of the expected settlement. However, it is possible settlement of the permanent pipeline bridge will also cause stress on the pipeline, requiring additional modification of the bridge. Another disadvantage is that the pipeline is exposed and vulnerable to vandalism. Additionally, the presence of a permanent structure over the levee impedes levee maintenance and construction of future levee lifts.

Directional Drilling

Directional drilling of pipelines under existing levees is an acceptable method of pipeline relocation provided the pipeline is drilled deep enough to avoid fracturing the levee from the pressure of the drilling fluid. To meet geotechnical criteria, pipelines are directionally drilled at least 100 feet under the base of the levee section and selected berm sections to avoid stresses from levee and berm settlement. The pipelines are offset approximately 15 feet from the existing pipeline alignment to allow workspace. The owners will need to perform excavation of a push/false ditch in order to weld together and test the new pipe segments and to utilize this ditch area to pull the pipe back into the hole. The owners will tie the new pipeline into the existing pipeline at each end and remove the old section of pipe in between. The advantage of this method is that it eliminates additional modification and/or relocation in the future. Additionally, the pipeline cannot be vandalized or damaged by levee maintenance or future construction. The initial cost of directional drilling is greater than that of the two previous relocation methods studied. However, the cost of future modification and/or

relocations resulting from the use of the previous two methods would most likely make the final relocation costs comparable to directional drilling.

For the purposes of this report, we anticipate pipelines will be directionally drilled to accommodate the levee construction and to avoid any interference with that construction.

HIGHWAYS AND ROADS

The LACPR alternative alignments impact several state and parish highways and roads. In some cases, information was obtained from the applicable transportation agencies to determine the physical characteristics of their existing facilities as well as traffic requirements. Traversing the proposed flood protection requires either relocating the highways and roads over the protection, or construction of permanent floodgates at the points of intersection of the protection and highways and roads. These structures remain open except for times of anticipated flooding, at which time they are closed and remain closed until the flood threat subsides. Relocation of highways and roads atop earthen ramps is the preferred construction method and will be used whenever feasible. Relocation of the applicable roads and highways in this manner eliminates the need for floodgate operation during flood conditions, thus allowing egress from affected areas to areas outside of the flood protection for a longer period of time. Permanent floodgates will be used in other locations.

CRITERIA FOR RELOCATING FACILITIES

Utility Owners

The facility owners will accomplish the design and relocation of their facilities in accordance with USACE facility crossing criteria, the owners' individual criteria and in a manner that eliminates interference with the alternatives.

Highways/Roads

Roads within the planning area are of various widths, surfaces and design standards. The minimum Design Standards for Rural Highways and Roads of the Louisiana Department of Transportation and Development had been reviewed to apply design standards to the highways/roads relocations required by the alternatives. Required road relocations will be accomplished in a manner to eliminate interference with the alternatives.

PROCEDURE FOR ACCOMPLISHING RELOCATIONS

Owners will prepare plans and specifications for their relocations in a manner that will not interfere with the project. The Government will review the owner's plans and specifications to determine if the owner's proposals are compatible with the project and then the owners will construct their relocations. The local assuring agency for the work will advise the affected owners to effect their relocations.

The relocations work for pipelines to be relocated by directional drilling will be performed prior to construction. All other relocations will be performed concurrent with construction of project features.

OPERATION, MAINTENANCE, REPAIR, REPLACEMENT AND REHABILITATION (OMRR&R)

OMRR&R cost estimates were developed for the barrier features investigated. These costs include yearly operation and maintenance costs as well as refurbishment and major rehabilitation (every 10 to 12 years for major structures). The estimates are based on data from existing barriers of similar size, maintenance requirements, and operating criteria. Costs were developed for refurbishment and major rehabilitation based on data from existing locks, structures, and barriers of similar size maintenance requirements, and operating criteria (number of hours/days/months, staffing level, 10 to 12 year major rehab, etc.).

COST ESTIMATES

GENERAL

Cost estimates were developed for the many features of the LACPR study which include but are not limited to levees, structures, rock protection, clearing, soil mixing, marsh nourishment, diversions, beach nourishment, and ridge restoration. The estimated costs were based upon an analysis of each line item evaluating quantity, production rate, and time, together with the appropriate equipment, labor and material costs or on New Orleans District historical data. All of the construction work is generally common to the New Orleans District. In addition, all labor, equipment and materials are typical of this type of construction.

Due to time constraints on this study, the delay in developing final design data, and the number of alternatives, a parametric matrix approach was used to combine and ratio the various individual features that were estimated into the costs for each structural alternative. A structural alternative is an alternative consisting solely of structural measures. Structural measures for LACPR primarily consist of physical structures that reduce surge and wave run-up, such as continuous or ring levees on land connected to floodgates acting as waterway barriers, where necessary. For example, detailed calculations and costs were done for levee templates at El. 25, 30, and 40. When the final design elevations were determined, a ratio of the base costs was used to determine the cost for the alternatives.

The cost estimates for large structures such as locks, closure gates, and pump stations were developed as part of this study. These costs were also done using a parametric approach where detailed calculations and costs were done for structures at El. 30, 35 and 40 and interpolated to determine costs for final design elevations.

Cost estimates for structural and other features (nonstructural, coastal, mitigation) were developed as described in general below and in detail in Annex 2.

The costs prepared for and presented in this report are not to be used for making funding decisions. They are sufficient in detail for comparison of plans for initial screening, for use in performing MCDA and presenting the final array of alternatives. A more detailed cost estimate per regulation will be generated in the future upon completion of additional engineering analysis.

PROJECT COST

Structural Alternatives

Details on how the costs were developed for the structural alternatives can be found in Annex 2. Costs were developed for the different sections shown on plates 1 thru 5. Structures required for each section were identified and added to the section costs along with Real Estate costs. E&D (12%) and Construction management (8%) costs are included. These separate section costs were added together with relocations costs to develop costs for structural alternatives. Figure 12 is a chart showing how the different cost components were combined to develop costs for structural alternatives. Table 24 gives a list of these alternatives and a description of each. Figures 13 – 28, taken from the Structural Plan Component Appendix, show the basic alignments of these alternatives. A more detailed description of the alternatives can be found in the Structural Plan Component Appendix and the Evaluation Results Appendix. First costs including lift construction costs were provided for use in initial screening. Yearly costs for those alternatives carried forward for MCDA were provided for the 50 year planning cycle. This information was used to develop a present value estimate for each of these alternatives. These costs included OMRR&R. Table 25 shows the present value of life cycle cost estimates for the structural alternatives carried forward into the final round of MCDA for two scenarios. These scenarios are sea level rise alternative 1 (low sea level rise) and sea level rise alternative 2 (high sea level rise). These scenarios were accounted for by adding a future lift approximately midway through the project life. Present value of life cycle costs for complete alternatives (structural, non-structural and coastal measures can be found in the Metric Results Tables shown in the Evaluation Results Appendix. Alternatives shown in table 25 include the PU1 alternatives with the new north shore alignments. All costs are in 2007 price levels.

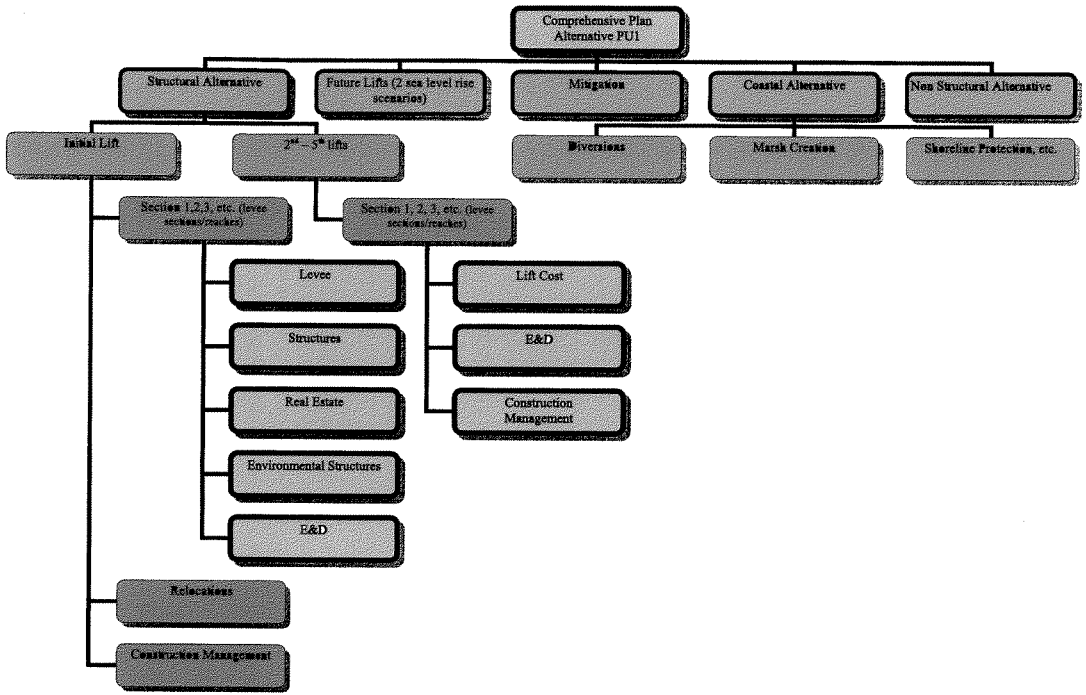


Figure 12 – Comprehensive Plan Alternative Cost Development

Table 24
Alternatives For Which Costs Were Developed
(100yr, 400yr and 1000yr Levels Developed for Each)

Planning Unit	Alternative Measure Code*	Description
1	HL-a-1	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Note that the Golden Triangle alignment is the current alignment for the authorized project.
1	HL-b-1	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne.
1	HL-a-2	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Also includes North Shore and West Shore Levees and a levee in upper Plaquemines Parish.
1	HL-b-2	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne. Also includes North Shore and West Shore Levees and a levee in upper Plaquemines Parish
1	HL-a-3	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Also includes Slidell and West Shore Levees and a levee in upper Plaquemines Parish.
1	HL-b-3	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne. Also includes Slidell and West Shore Levees and a levee in upper Plaquemines Parish.
1	HL-a-4	High Level Plan – Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Also includes West Shore Levee.
1	LP-a-1	Barrier Weir, Existing Levees - (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-b-1	Barrier Weir - Raise Existing Levees (includes the Lake

Table 24
Alternatives For Which Costs Were Developed
(100yr, 400yr and 1000yr Levels Developed for Each)

Planning Unit	Alternative Measure Code*	Description
		Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-a-2	Barrier Weir - Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Includes North Shore and West Shore Levees and a levee in upper Plaquemines Parish. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-b-2	Barrier Weir - Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne. Includes North Shore and West Shore Levees and a levee in upper Plaquemines Parish. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-a-3	Barrier Weir - Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Includes Slidell and West Shore Levees and a levee in upper Plaquemines Parish. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-b-3	Barrier Weir - Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows an alignment at the edge of Golden Triangle and Lake Borgne. Includes Slidell and West Shore Levees and a levee in upper Plaquemines Parish. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
1	LP-a-4	Barrier Weir - Raise Existing Levees (includes the Lake Pontchartrain and Vicinity Project Levees). Levee follows Golden Triangle alignment at the Confluence of GIWW and MRGO. Includes West Shore Levee. Also includes a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
Note: For PU1 the full range of estimates was prepared for both the original and revised north shore alignments.		

Table 24
Alternatives For Which Costs Were Developed
(100yr, 400yr and 1000yr Levels Developed for Each)

Planning Unit	Alternative Measure Code*	Description
2	WBI-1	West Bank Interior Plan (includes West Bank and Vicinity (WBV) levees with new sector gate).
2	G-1-1a	GIWW Alignment (Barrier Weir along GIWW), Existing WBV Levees with new sector gate, Larose to Golden Meadow levees.
2	G-1-4a	GIWW Alignment (Barrier Weir along GIWW), Existing WBV Levees with new sector gate, Larose to Golden Meadow levees, with Boutee and Des Allemands levees.
2	R-1-2	Ridge Alignment - Existing WBV Levees with new sector gate, Larose to Golden Meadow levees and Boutee levees.
2	R-1-3	Ridge Alignment - Existing WBV Levees with new sector gate, Larose to Golden Meadow levees and Boutee and Des Allemands levees.
2	R-1-4	Ridge Alignment - Existing WBV Levees with new sector gate, Larose to Golden Meadow levees and Boutee, Des Allemands and Bayou Lafourche levees.
2	G-1-1	GIWW Alignment (Barrier Weir along GIWW), Existing WBV Levees with new sector gate, Larose to Golden Meadow levees and Lafitte ring levees. This alternative replaced G-1-1a.
2	G-1-4	GIWW Alignment (Barrier Weir along GIWW), Existing WBV Levees with new sector gate, Larose to Golden Meadow levees, with Boutee and Des Allemands levees and Lafitte ring levees. This alternative replaced G-1-4a.
3a	PU3a-M-1	Morganza to Gulf (MTG) Alignment then GIWW to tieback west of Morgan City
3a	PU3a-R-1	MTG Alignment with tieback to high ground south of Thibodaux and Morgan City Ring Levee (designation changed to PU3a-M-2)
3a	PU3a-G-1	MTG Alignment at 100yr with GIWW interior levee, Morgan City Ring Levee. This alternative designation was changed from PU3a-R-2.
3b	PU-3b-G-1	GIWW Alignment, Patterson Ring Levee.
3b	PU3b-F-1	Patterson Ring Levee, Franklin to Abeville alignment (inland of the GIWW).
3b	PU3b-R-1	Ring levee alignment - Patterson Ring Levee, Franklin/Baldwin Ring Levee, New Iberia Ring Levee, Erath Ring Levee, Delcambre Ring Levee and Abbeville Ring Levee.
4	PU4-G-1	GIWW levee - GIWW Alignment east of Calcasieu River, Ring Levee west of Calcasieu River.
4	PU4-G-2	GIWW levee - GIWW Alignment east of Calcasieu with tie-in west of the Vermillion River near Kaplan, Ring Levee west of

Table 24 Alternatives For Which Costs Were Developed (100yr, 400yr and 1000yr Levels Developed for Each)		
Planning Unit	Alternative Measure Code*	Description
		Calcasieu River.
4	PU4-G-3	GIWW levee - GIWW Alignment east of Calcasieu River, Ring Levee west of Calcasieu River. with levee set at 12'.
4	PU4-R-1	Ring levee alignment - Ring Levee west of Calcasieu River, Lake Charles Ring Levee, Kaplan Ring Levee, Gueydan Ring Levee.
*A description of the alternative measure codes can be found in attachment 1 to Annex 2.		

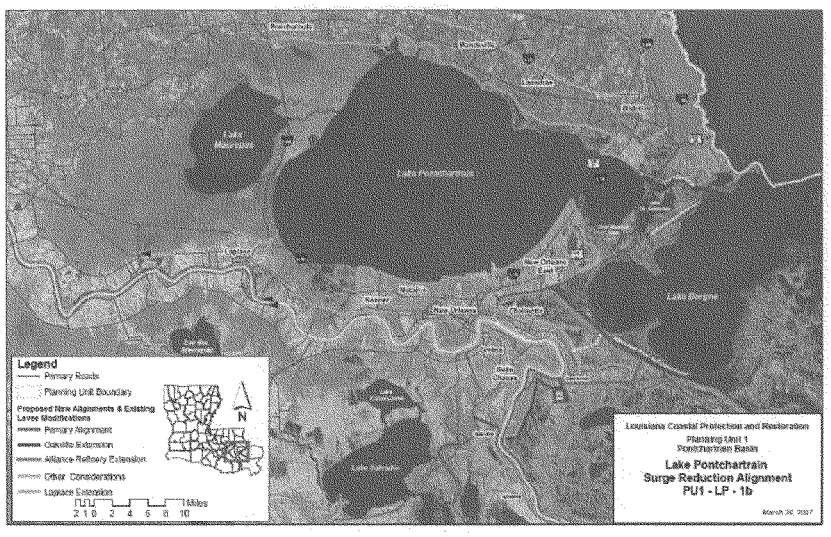


Figure 13 - PU1 Example of a Lake Pontchartrain Surge Reduction Alignment from the Plan Formulation Atlas (Barrier Plan) (Note that the north shore alignments were changed to those shown in figures 14 and 15.)

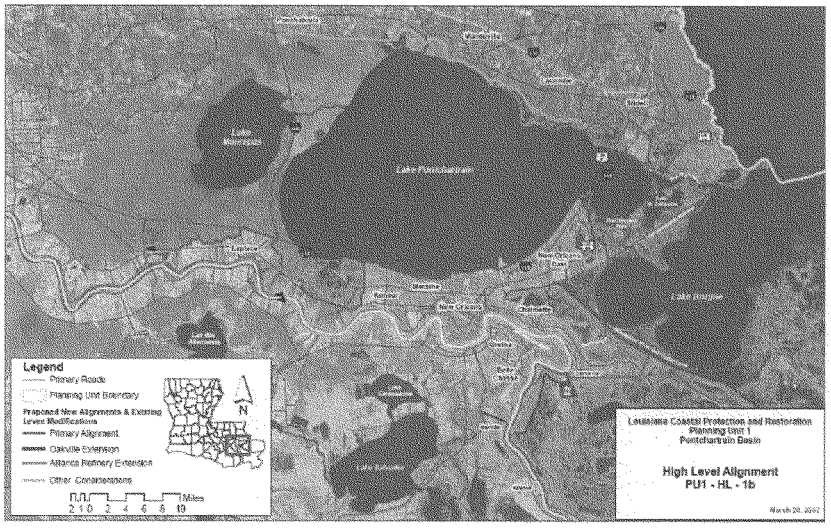


Figure 14 - PU1 Example of a High Level Alignment from the Plan Formulation Atlas (Note that the north shore alignments were changed to those shown in figures 14 and 15.)

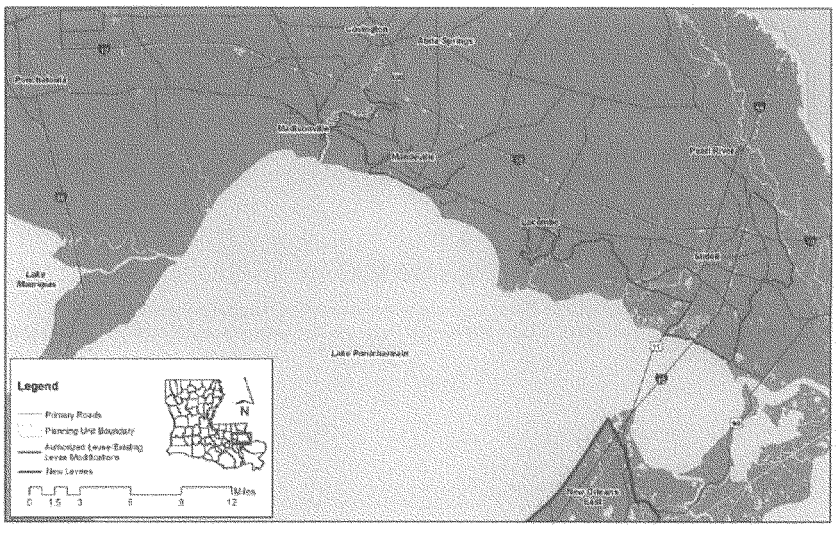


Figure 15 - PU1 Reformulated North Shore Alignment

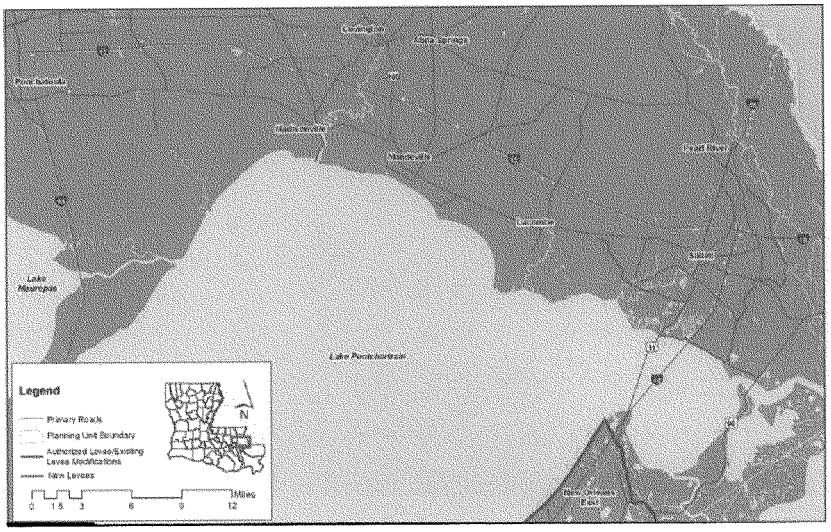


Figure 16 – PU1 Reformulated Slidell Ring Levee

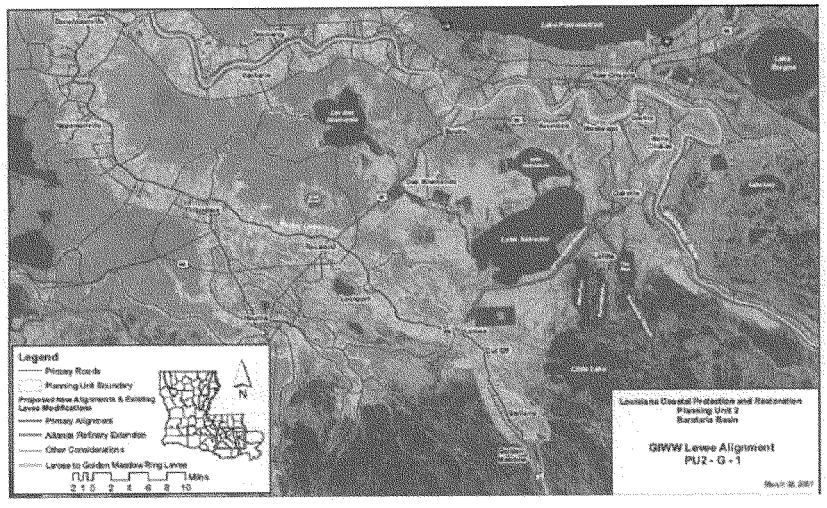


Figure 17 - PU GIWW Alignment from the Plan Formulation Atlas (Note that this alignment was modified to include Lafitte ring levees.)

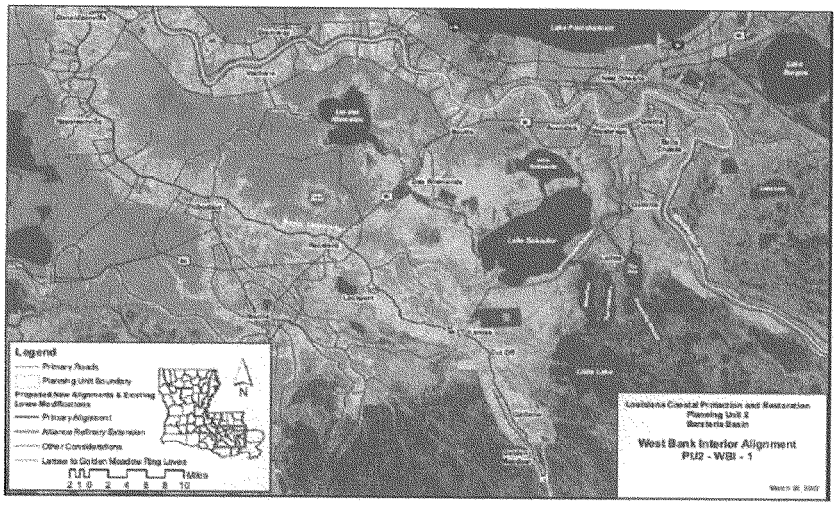


Figure 18 - PU2 West Bank Interior Alignment from the Plan Formulation Atlas

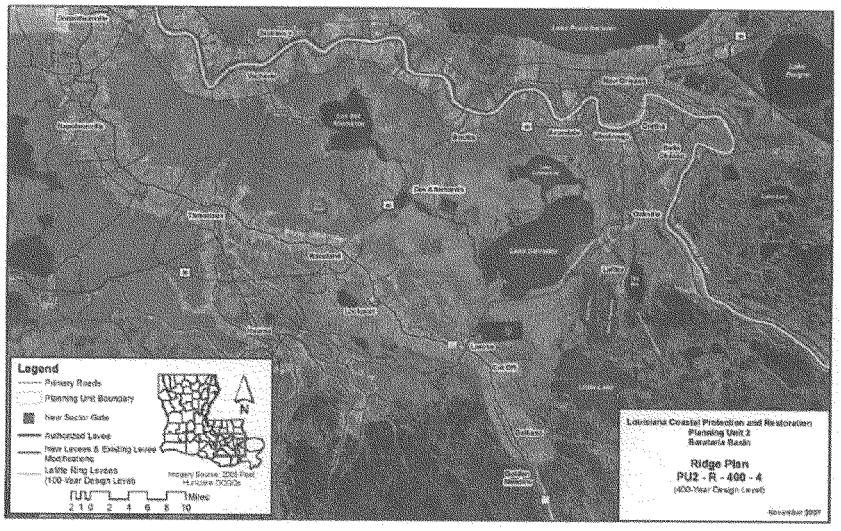


Figure 19 - PU2 Ridge Alignment

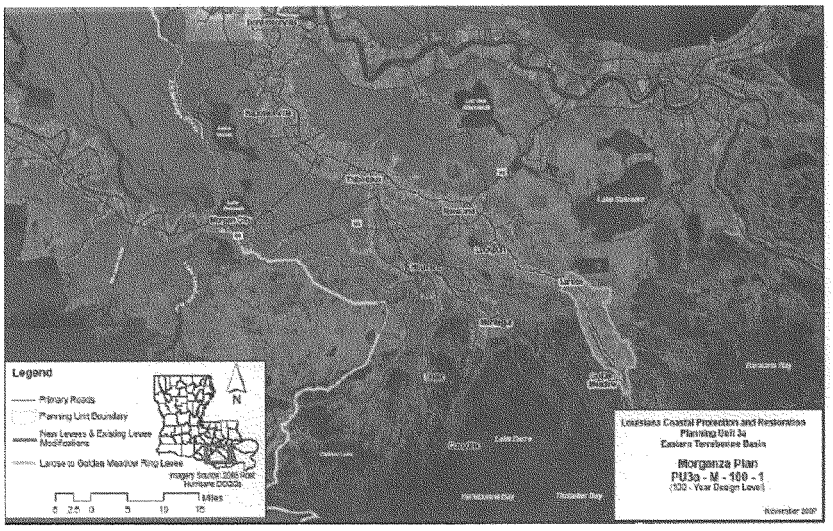


Figure 20 - PU3a Morganza Levee Alignment with Tie-in West of Morgan City

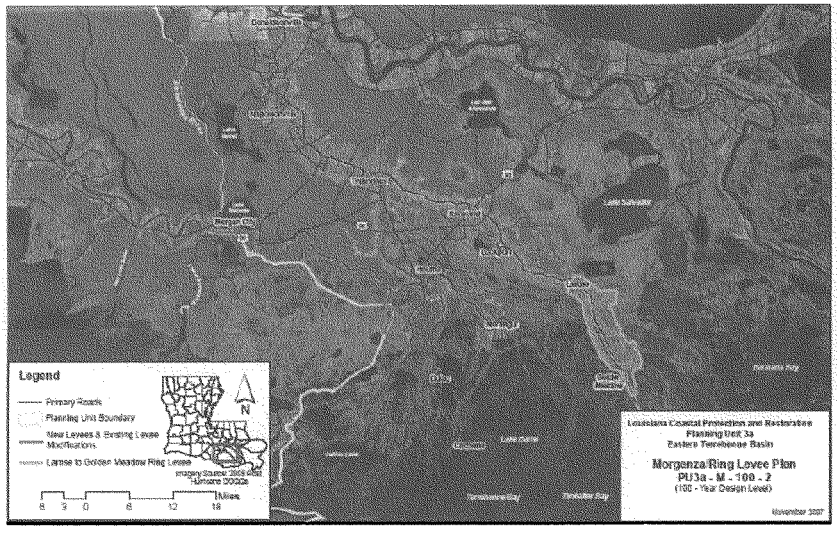


Figure 21 - PU3a Morganza Levee Alignment with Morgan City Ring Levee

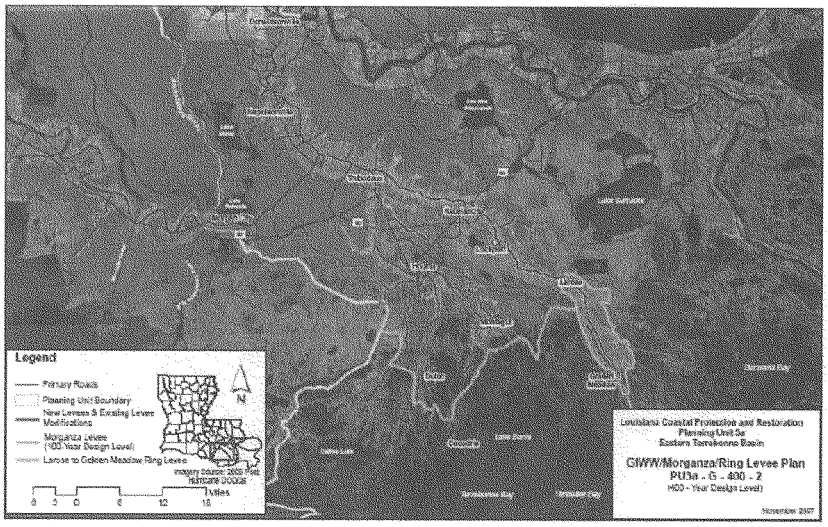


Figure 22 - PU3a GIWW Alignment

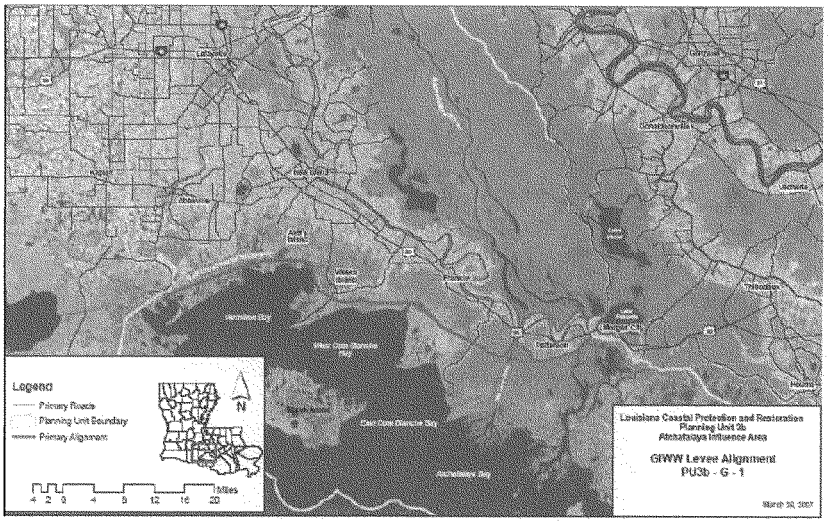


Figure 23 - PU3b Example GIWW Alignment from the Plan Formulation Atlas

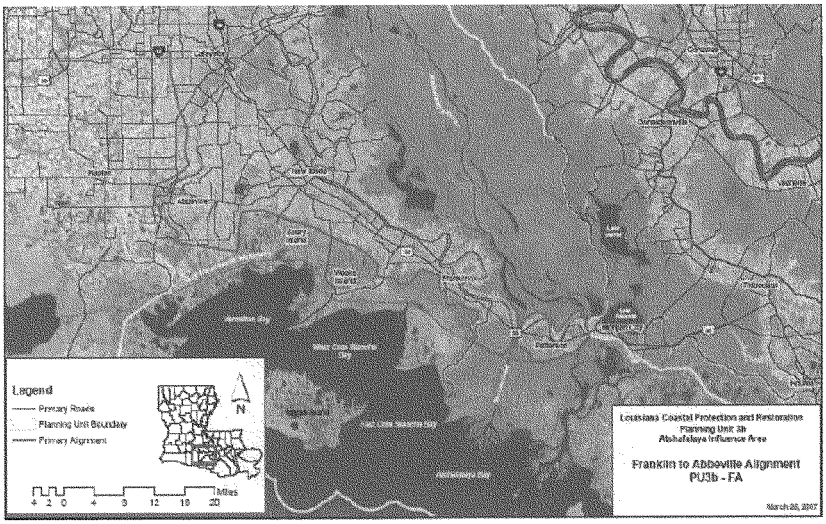


Figure 24 - PU3b Example Franklin to Abbeville Alignment from the Plan Formulation Atlas

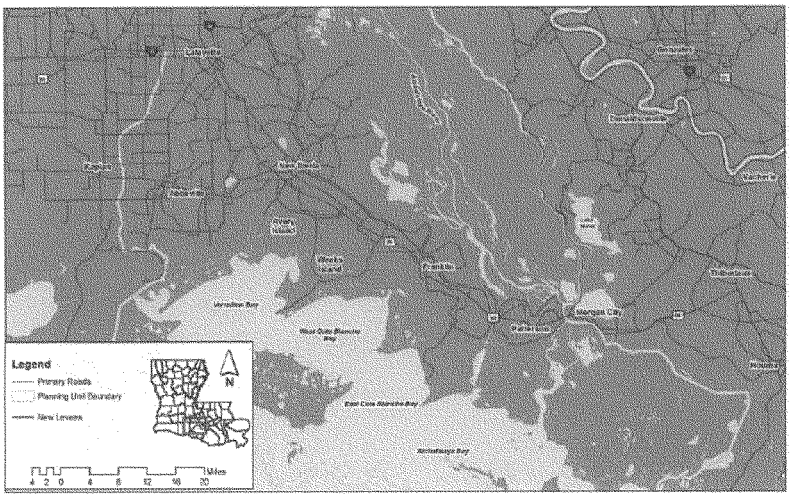


Figure 25 - PU3b Ring Levee Alignment

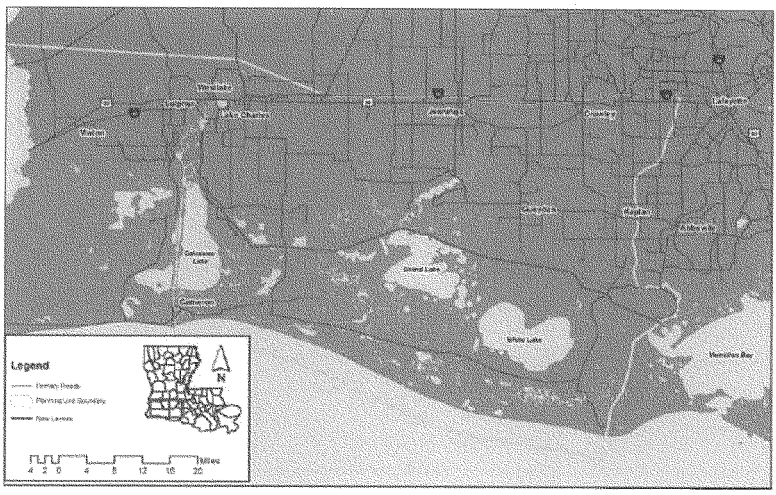


Figure 26 - PU4 GIWW Alignment

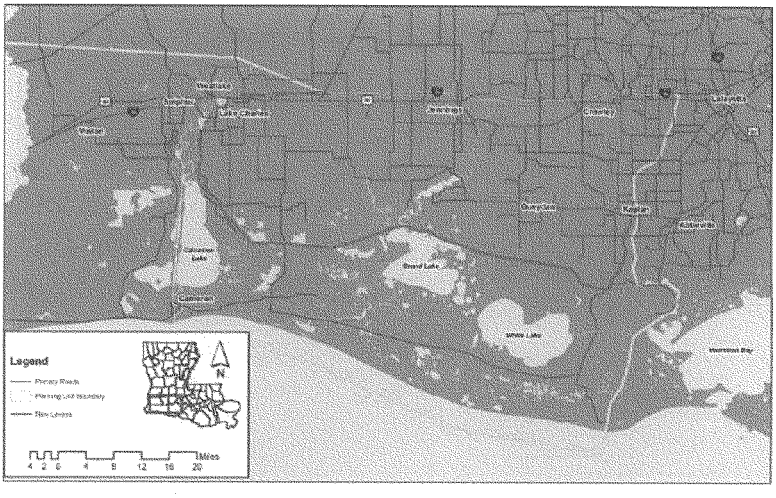


Figure 27 - PU4 Stand Alone GIWW Alignment

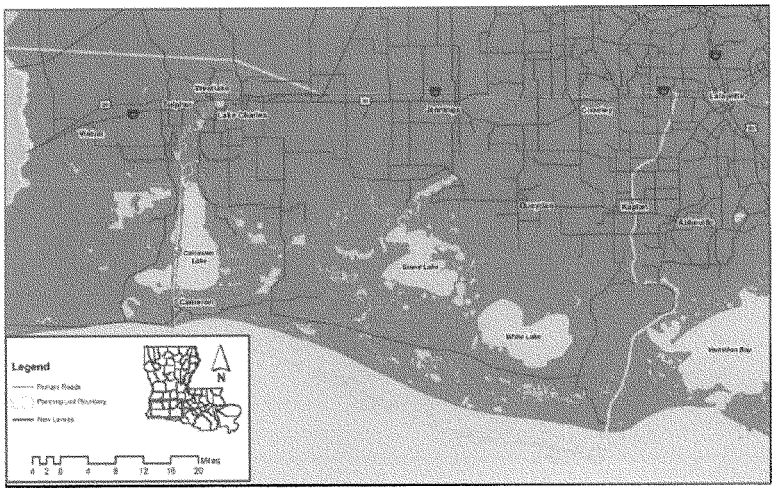


Figure 28 - PU4 Ring Levee Alignments

Table 25
Present Value Cost Estimates For Structural Alternatives Included in MCDA
(See Table 24 for a description of these alternatives)

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost $ (B)
1	PU1-LP-a-100-1	L	7.02
		H	7.13
	PU1-LP-a-100-2	L	22.44
		H	22.58
	PU1-LP-a-100-3	L	21.09
		H	21.28
	PU1-LP-b-400-1	L	25.54
		H	25.62
	PU1-LP-b-400-3	L	45.08
		H	45.24
	PU1-LP-b-1000-1	L	33.33
		H	33.56
	PU1-LP-b-1000-2	L	59.40
		H	59.61
	PU1-HL-a-100-3	L	15.89
		H	15.93
	PU1-HL-a-100-2	L	19.19
		H	19.25

Table 25
Present Value Cost Estimates For Structural Alternatives Included in MCDA
(See Table 24 for a description of these alternatives)

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost $ (B)
	PU1-HL-b-400-2	L	49.57
		H	49.81
	PU1-HL-b-400-3	L	44.90
		H	45.10
2	PU2-WBI-100-1	L	1.00
		H	1.02
	PU2-R-100-2	L	7.73
		H	7.77
	PU2-R-400-2	L	25.41
		H	25.52
	PU2-R-100-4	L	13.35
		H	13.42
	PU-2-R-400-4	L	31.47
		H	31.61
	PU2-G-1000-4	L	42.34
		H	42.46
	PU2-G-100-1	L	7.6
		H	7.75
	PU2-G-100-4	L	14.52
		H	14.7
	PU2-G-400-4	L	34.75
		H	34.88
	PU2-R-1000-4	L	39.17
		H	39.77
	PU2-R-100-3	L	10.15
		H	10.20
	PU2-WBI-400-1	L	18.29
		H	18.32
	PU2-R-400-3	L	28.32
		H	28.44
3a	PU3a-M-100-1	L	21.41
		H	21.93
	PU3a-M-100-2	L	18.98
		H	19.10
	PU3a-G-400-2	L	25.21
		H	25.28
	PU3a-G-1000-2	L	27.63
		H	27.70
3b	PU3b-G-100-1	L	15.21

Table 25
Present Value Cost Estimates For Structural Alternatives Included in MCDA
(See Table 24 for a description of these alternatives)

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost $ (B)
		H	15.24
	PU3b-F-100-1	L	13.92
		H	13.95
	PU3b-F-400-1	L	23.44
		H	23.64
	PU3b-F-1000-1	L	31.07
		H	31.09
	PU3b-RL-100-1	L	11.58
		H	11.61
	PU3b-RL-400-1	L	18.00
		H	18.02
4	PU4-G-100-1	L	11.99
		H	12.27
	PU4-G-100-2	L	11.78
		H	12.07
	PU4-G-400-3	L	11.73
		H	12.01
	PU4-G-1000-3	L	12.21
		H	12.49
	PU4-RL-100-1	L	2.70
		H	2.72
	PU4-RL-400-1	L	3.47
		H	3.49
	PU4-RL-1000-1	L	3.76
		H	3.80

Coastal Restoration Features

Cost for coastal restoration features were developed as described above in the Coastal Restoration Features Section and discussed in detail in Annex 1.

Mitigation

The number of acres of wetlands impacted was developed for each alternative. These acres are to be mitigated at a rate of 1.5 acres to each acre impacted. Current mitigation cost is estimated at $80,000 an acre.

Nonstructural Plan Components

Costs for nonstructural plan components were provided in the Nonstructural Plan Component Appendix. Details of these components and the information on the development of these estimates can be found in the referenced appendix.

ESTIMATES FOR COMPREHENSIVE PLAN ALTERNATIVES

Comprehensive plan alternatives (structural, coastal, nonstructural) are discussed in detail in the Main Report. Figure XX shows how different cost items were combined to develop comprehensive plan alternatives. Table 26 shows the present value costs for all alternatives carried forward into the final round of MCDA. Tables showing costs of individual plan components (coastal features, structural features, mitigation, nonstructural features) for each comprehensive plan alternative are included in Annex 2.

TABLE 26
LACPR ALTERNATIVES
First Cost for Alternatives For Scenarios Used For MCDA

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Planning Unit 1				
No Action	0	0	0	0
R2 (Coastal)	9,448,607,969	9,637,580,128	9,448,607,969	9,637,580,128
NS-100	13,976,786,412	14,165,758,572	13,707,342,705	13,896,314,864
NS-400	25,988,874,745	26,177,846,904	28,858,222,244	29,047,194,403
NS-1000	36,426,615,692	36,615,587,851	37,689,306,856	37,878,279,015
HL-a-100-2	28,809,443,453	29,114,686,066	28,809,443,453	29,114,686,066
HL-a-100-3	25,621,931,458	25,881,282,879	25,621,931,458	25,881,282,879
HL-b-400-2	63,140,024,338	63,815,915,224	63,140,024,338	63,815,915,224
HL-b-400-3	58,362,881,421	58,975,714,443	58,362,881,421	58,975,714,443
LP-a-100-1	16,115,149,095	16,524,784,241	16,115,149,095	16,524,784,241
LP-a-100-2	29,726,476,210	30,198,417,749	29,726,476,210	30,198,417,749
LP-a-100-3	29,037,286,948	29,607,034,907	29,037,286,948	29,607,034,907
LP-b-400-1	35,247,782,100	35,604,571,107	35,247,782,100	35,604,571,107
LP-b-400-3	54,602,752,270	55,111,460,583	54,602,752,270	55,111,460,583
LP-b-1000-1	45,525,510,246	46,168,402,646	45,525,510,246	46,168,402,646
LP-b-1000-2	71,345,488,335	71,956,944,660	71,345,488,335	71,956,944,660
C-HL-a-100-2	30,841,507,818	31,146,750,431	30,391,395,739	30,696,638,352
C-HL-a-100-3	27,798,358,539	28,057,709,960	27,364,359,000	27,623,710,421
C-HL-b-400-2	65,366,416,079	66,042,306,965	64,896,758,138	65,572,649,024
C-HL-b-400-3	60,748,289,027	61,361,122,049	60,415,682,905	61,028,515,928
C-LP-a-100-1	18,830,206,036	19,239,841,182	18,424,524,432	18,834,159,578
C-LP-a-100-2	31,704,982,805	32,176,924,344	31,208,858,542	31,680,800,081

TABLE 26
LACPR ALTERNATIVES
First Cost for Alternatives For Scenarios Used For MCDA

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
C-LP-a-100-3	31,126,205,421	31,695,953,380	30,642,400,367	31,212,148,326
C-LP-b-400-1	39,319,329,328	39,676,118,335	39,765,397,390	40,122,186,398
C-LP-b-400-3	56,822,605,370	57,331,313,684	56,370,258,215	56,878,966,529
C-LP-b-1000-1	50,003,978,929	50,646,871,330	51,286,163,182	51,929,055,582
C-LP-b-1000-2	73,758,273,372	74,369,729,697	73,044,983,733	73,656,440,058
Planning Unit 2				
No Action	0	0	0	0
R2 (Coastal)	11,290,051,065	11,312,631,167	11,290,051,065	11,312,631,167
NS-100	14,281,264,591	14,303,844,693	14,371,126,266	14,393,706,368
NS-400	22,007,914,671	22,030,494,773	30,699,665,341	30,722,245,443
NS-1000	31,521,628,364	31,544,208,466	39,530,749,953	39,553,330,055
G-100-1	18,527,843,400	18,842,508,657	18,527,843,400	18,842,508,657
G-100-4	24,017,746,420	24,404,081,288	24,017,746,420	24,404,081,288
G-400-4	45,939,377,941	46,236,618,190	45,939,377,941	46,236,618,190
G-1000-4	51,282,989,993	51,555,389,161	51,282,989,993	51,555,389,161
R-100-2	14,011,898,966	14,097,013,403	14,011,898,966	14,097,013,403
R-100-3	16,172,461,036	16,286,773,530	16,172,461,036	16,286,773,530
R-100-4	18,781,724,515	18,931,286,259	18,781,724,515	18,931,286,259
R-400-2	33,387,679,923	33,607,921,135	33,387,679,923	33,607,921,135
R-400-3	35,810,737,945	36,061,187,200	35,810,737,945	36,061,187,200
R-400-4	38,610,421,339	38,897,597,997	38,610,421,339	38,897,597,997
R-1000-4	46,686,471,376	47,912,192,131	46,686,471,376	47,912,192,131
WBI-100-1	11,948,584,459	12,021,391,685	11,948,584,459	12,021,391,685
WBI-400-1	31,010,321,230	31,200,785,597	31,010,321,230	31,200,785,597
C-G-100-1	20,670,262,205	20,984,927,462	20,722,808,709	21,037,473,966
C-G-100-4	25,813,320,568	26,199,655,436	25,841,372,318	26,227,707,186
C-G-400-4	47,546,688,976	47,843,929,224	48,034,335,706	48,331,575,954
C-G-1000-4	53,373,034,972	53,645,434,140	53,732,936,384	54,005,335,552
C-R-100-2	16,375,944,939	16,461,059,375	16,398,891,388	16,484,005,825
C-R-100-3	18,288,502,874	18,402,815,368	18,283,012,562	18,397,325,056
C-R-100-4	20,773,219,737	20,922,781,481	20,792,919,999	20,942,481,743
C-R-400-2	35,243,884,836	35,464,126,048	36,211,034,110	36,431,275,322
C-R-400-3	37,563,106,079	37,813,555,335	38,318,905,370	38,569,354,626
C-R-400-4	40,215,865,952	40,503,042,610	40,904,545,730	41,191,722,388
C-R-1000-4	48,649,556,447	49,875,277,202	49,094,687,594	50,320,408,348
C-WBI-100-1	14,908,146,322	14,980,953,548	14,999,456,823	15,072,264,049
C-WBI-400-1	33,448,013,929	33,638,478,296	34,627,916,714	34,818,381,080
Planning Unit 3a				
No Action	0	0	0	0
R1 (Coastal)	19,431,425,544	19,625,739,799	19,431,425,544	19,625,739,799
NS-100	24,923,295,980	25,117,610,236	24,912,137,029	25,106,451,285

TABLE 26
LACPR ALTERNATIVES
First Cost for Alternatives For Scenarios Used For MCDA

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
NS-400	26,906,161,065	27,100,475,320	26,801,113,801	26,995,428,056
NS-1000	27,624,326,837	27,818,641,092	27, 804,592,176	27,998,906,431
M-100-1	38,188,268,343	39,433,809,593	38,188,268,343	39,433,809,593
M-100-2 (was R-100-1)	36,469,900,220	36,898,978,039	36,469,900,220	36,898,978,039
G-400-2	41,876,971,393	42,218,447,087	41,876,971,393	42,218,447,087
G-1000-2	43,993,106,295	44,345,192,507	43,993,106,295	44,345,192,507
C-M-100-1	38,630,924,156	39,876,465,405	38,625,150,100	39,870,691,349
C-M-100-2	36,957,783,303	37,386,861,123	36,953,477,181	37,382,555,001
C-G-400-2	42,937,138,923	43,278,614,617	43,159,226,549	43,500,702,243
C-G-1000-2	45,733,087,045	46,085,173,257	45,912,522,916	46,264,609,128
Planning Unit 3b				
No Action	0	0	0	0
R1 (Coastal)	4,364,994,612	4,408,644,558	4,364,994,612	4,408,644,558
NS-100	6,208,801,097	6,252,451,043	6,258,099,063	6,301,749,010
NS-400	7,551,099,977	7,594,749,924	7,641,051,958	7,684,701,904
NS-1000	8,339,265,602	8,382,915,548	8,799,298,036	8,842,947,983
G-100-1	18,493,569,685	18,586,553,146	18,493,569,685	18,586,553,146
F-100-1	17,685,812,738	17,803,604,048	17,685,812,738	17,803,604,048
F-400-1	26,984,322,398	27,369,959,222	26,984,322,398	27,369,959,222
F-1000-1	35,286,667,844	35,357,865,681	35,286,667,844	35,357,865,681
RL-100-1	15,484,510,089	15,594,753,846	15,484,510,089	15,594,753,846
RL-400-1	21,656,143,131	21,756,499,550	21,656,143,131	21,756,499,550
C-G-100-1	18,664,660,787	18,757,644,248	18,672,714,765	18,765,698,226
C-F-100-1	17,942,124,885	18,059,916,195	17,952,066,339	18,069,857,648
C-F-400-1	27,189,604,379	27,575,241,203	27,300,924,604	27,686,561,427
C-F-1000-1	35,891,880,509	35,678,562,221	35,630,608,036	35,701,805,873
C-RL-100-1	16,089,722,754	16,199,966,511	16,120,956,597	16,231,200,354
C-RL-400-1	22,351,942,103	22,452,298,522	22,902,638,413	23,002,994,831
Planning Unit 4				
No Action	0	0	0	0
R1 (Coastal)	10,295,953,284	10,501,872,350	10,295,953,284	10,501,872,350
NS-100	11,799,111,008	12,005,030,074	11,800,759,074	12,006,678,139
NS-400	12,712,625,681	12,918,544,746	12,670,624,451	12,876,543,516
NS-1000	13,649,358,849	13,855,277,914	13,758,546,808	13,964,465,874
G-100-1	21,101,215,412	21,882,856,502	21,101,215,412	21,882,856,502
G-100-2	21,031,331,562	21,809,167,725	21,031,331,562	21,809,167,725
G-400-3	20,831,662,921	21,608,513,415	20,831,662,921	21,608,513,415
G-1000-3	21,273,016,896	22,055,110,177	21,273,016,896	22,055,110,177
RL-100-1	12,586,970,598	12,830,007,571	12,586,970,598	12,830,007,571
RL-400-1	13,443,833,884	13,696,639,399	13,443,833,884	13,696,639,399

TABLE 26
LACPR ALTERNATIVES
First Cost for Alternatives For Scenarios Used For MCDA

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
RL-1000-1	13,872,652,090	14,130,326,533	13,872,652,090	14,130,326,533
C-G-100-1	22,438,559,356	23,220,200,446	22,444,957,627	23,226,598,716
C-G-100-2	22,198,009,561	22,975,845,724	22,919,055,370	23,696,891,533
C-G-400-3	23,281,426,175	24,058,276,669	23,384,480,977	24,161,331,471
C-G-1000-3	22,291,343,071	23,073,436,351	22,302,237,190	23,084,330,471
C-RL-100-1	13,606,235,895	13,849,272,867	13,616,981,050	13,860,018,023
C-RL-400-1	14,484,955,687	14,737,761,201	14,975,768,607	15,228,574,121
C-RL-1000-1	16,373,894,553	16,631,568,995	16,403,530,126	16,661,204,569

Color Code
No Action
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Scenario 1 - Low relative sea level rise, High Employment, Dispersed Population
Scenario 2 - High relative sea level rise, High Employment, Dispersed Population
Scenario 3 - Low relative sea level rise, Business-as-usual, Compact Population
Scenario 4 - High relative sea level rise, Business-as-usual, Compact Population

FINAL ARRAY OF ALTERNATIVES

After completion of the alternative analysis including the MCDA the alternatives under consideration were narrowed down to a final array. Tables 27 and 28 show the first cost and fully funded cost of these alternatives.

TABLE 27
LACPR ALTERNATIVES
First Cost for Alternatives In The Final Array ($B)

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Planning Unit 1				
No Action	0	0	0	0
R2 (Coastal)	36.2	36.9	36.2	36.9
NS-100	41.7	42.4	41.4	42.1
NS-400	56.1	56.8	59.5	60.2
NS-1000	68.6	69.3	70.1	70.8
LP-400-1	44.2	45.2	44.2	45.2
C-LP-400-1	47.5	48.5	47.0	48.0

TABLE 27
LACPR ALTERNATIVES
First Cost for Alternatives In The Final Array ($B)

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Planning Unit 2				
No Action	0	0	0	0
NS-400	22.9	23.0	33.1	33.2
WBI-100-1	10.8	11.0	10.8	11.0
C-G-100-1	21.3	21.8	21.4	21.8
C-WBI-100-1	14.4	14.6	14.5	14.7
C-R-100-2	16.2	16.3	16.2	16.4
Planning Unit 3a				
No Action	0	0	0	0
NS-100	6.6	6.6	6.8	6.6
NS-400	9.0	9.0	8.8	8.8
NS-1000	9.9	9.8	10.0	10.0
C-M-100-1	23.0	24.3	23.0	24.3
C-M-100-2	21.0	21.3	21.0	21.3
Planning Unit 3b				
No Action	0	0	0	0
NS-400	3.8	3.8	3.9	3.9
NS-1000	4.8	4.8	5.2	5.3
C-G-100-1	17.2	17.2	17.2	17.2
C-F-100-1	16.3	16.4	16.3	16.4
C-RL-100-1	14.1	14.1	14.1	14.2
Planning Unit 4				
No Action	0	0	0	0
NS-100	1.8	1.8	1.8	1.8
NS-400	2.9	2.9	2.8	2.8
NS-1000	4.0	4.2	4.2	4.2
C-RL-100-1	4.3	4.4	4.4	4.4
C-RL-400-1	5.2	5.2	6.0	6.1
C-RL-1000-1	7.2	7.3	7.4	7.4

Color Code
No Action
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Scenario 1 - Low relative sea level rise, High Employment, Dispersed Population
Scenario 2 - High relative sea level rise, High Employment, Dispersed Population
Scenario 3 - Low relative sea level rise, Business-as-usual, Compact Population
Scenario 4 - High relative sea level rise, Business-as-usual, Compact Population

TABLE 28
LACPR ALTERNATIVES
Fully Funded Cost for Alternatives In The Final Array

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Planning Unit 1				
No Action	0	0	0	0
R2 (Coastal)	64.0	65.4	64.1	65.4
NS-100	70.8	72.0	70.4	71.7
NS-400	88.7	89.9	92.9	94.2
NS-1000	104.2	105.5	106.0	107.3
LP-a-100-1	76.0	77.8	76.0	77.8
C-LP-a-100-1	80.0	81.8	79.4	81.2
Planning Unit 2				
No Action	0	0	0	0
NS-400	30.2	30.4	42.9	43.0
WBI-100-1	15.4	15.6	15.4	15.6
C-G-100-1	30.6	31.4	30.7	31.5
C-WBI-100-1	19.8	20.0	19.9	20.2
C-R-100-2	22.4	22.7	22.4	22.7
Planning Unit 3a				
No Action	0	0	0	0
NS-100	8.2	8.2	8.2	8.1
NS-400	11.1	11.1	11.0	11.0
NS-1000	12.2	12.2	12.4	12.4
C-M-100-1	34.9	37.3	34.9	37.3
C-M-100-2	32.3	32.8	32.3	32.8
Planning Unit 3b				
No Action	0	0	0	0
NS-400	4.7	4.7	4.9	4.9
NS-1000	5.9	5.9	6.6	6.6
C-G-100-1	27.0	27.1	27.0	27.1
C-F-100-1	25.4	25.6	25.4	25.6
C-RL-100-1	21.8	21.9	21.8	22.0
Planning Unit 4				
No Action	0	0	0	0
NS-100	2.2	2.2	2.2	2.2
NS-400	3.6	3.6	3.5	3.5
NS-1000	5.0	5.0	5.1	5.2
C-RL-100-1	6.1	6.2	6.1	6.2
C-RL-400-1	7.4	7.5	8.5	8.6
C-RL-1000-1	10.2	10.3	10.3	10.4

TABLE 28
LACPR ALTERNATIVES
Fully Funded Cost for Alternatives In The Final Array

Alternative	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Color Code	Scenario 1 - Low relative sea level rise, High Employment, Dispersed Population			
	Scenario 2 - High relative sea level rise, High Employment, Dispersed Population			
No Action	Scenario 3 - Low relative sea level rise, Business-as-usual, Compact Population			
Nonstructural / Coastal	Scenario 4 - High relative sea level rise, Business-as-usual, Compact Population			
Structural / Coastal				
Comprehensive (Struct + NS + Coastal)				

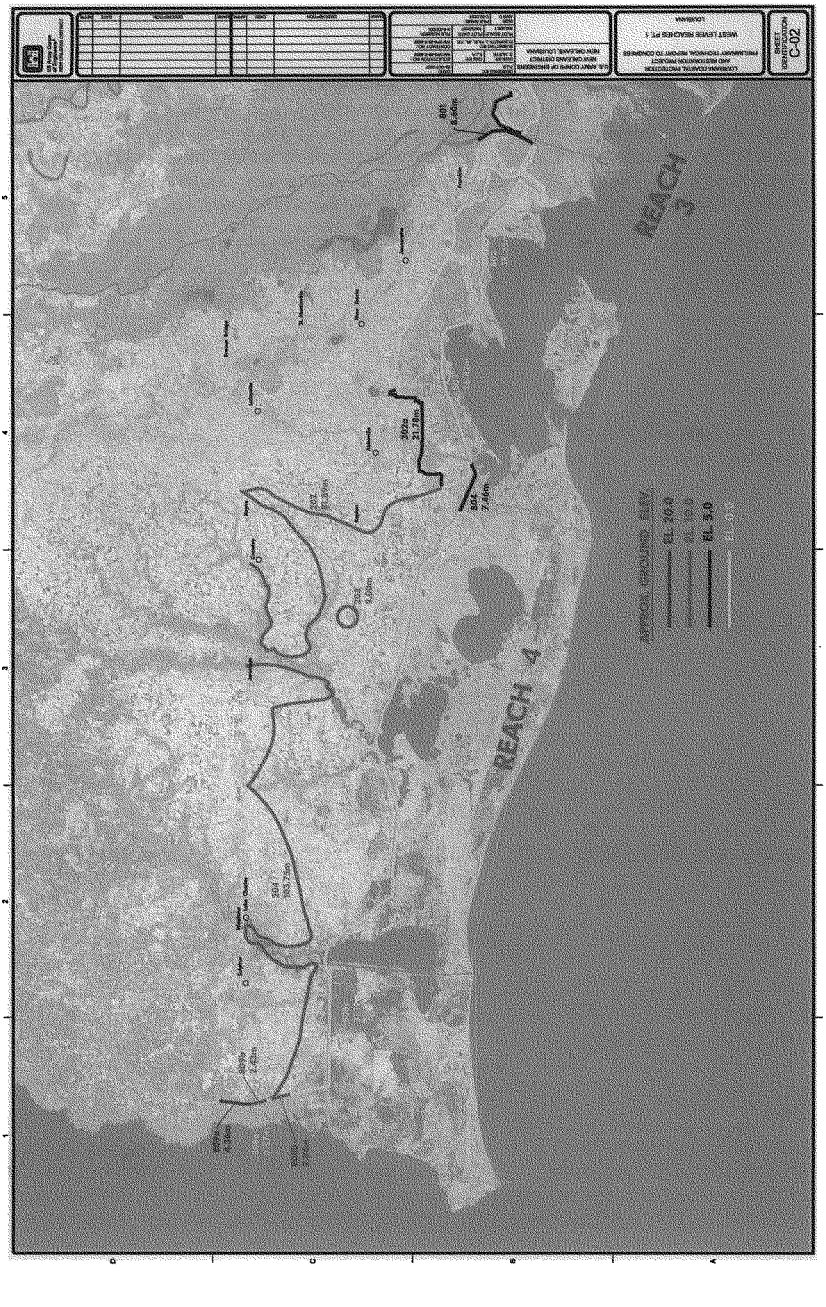
FUTURE STUDIES

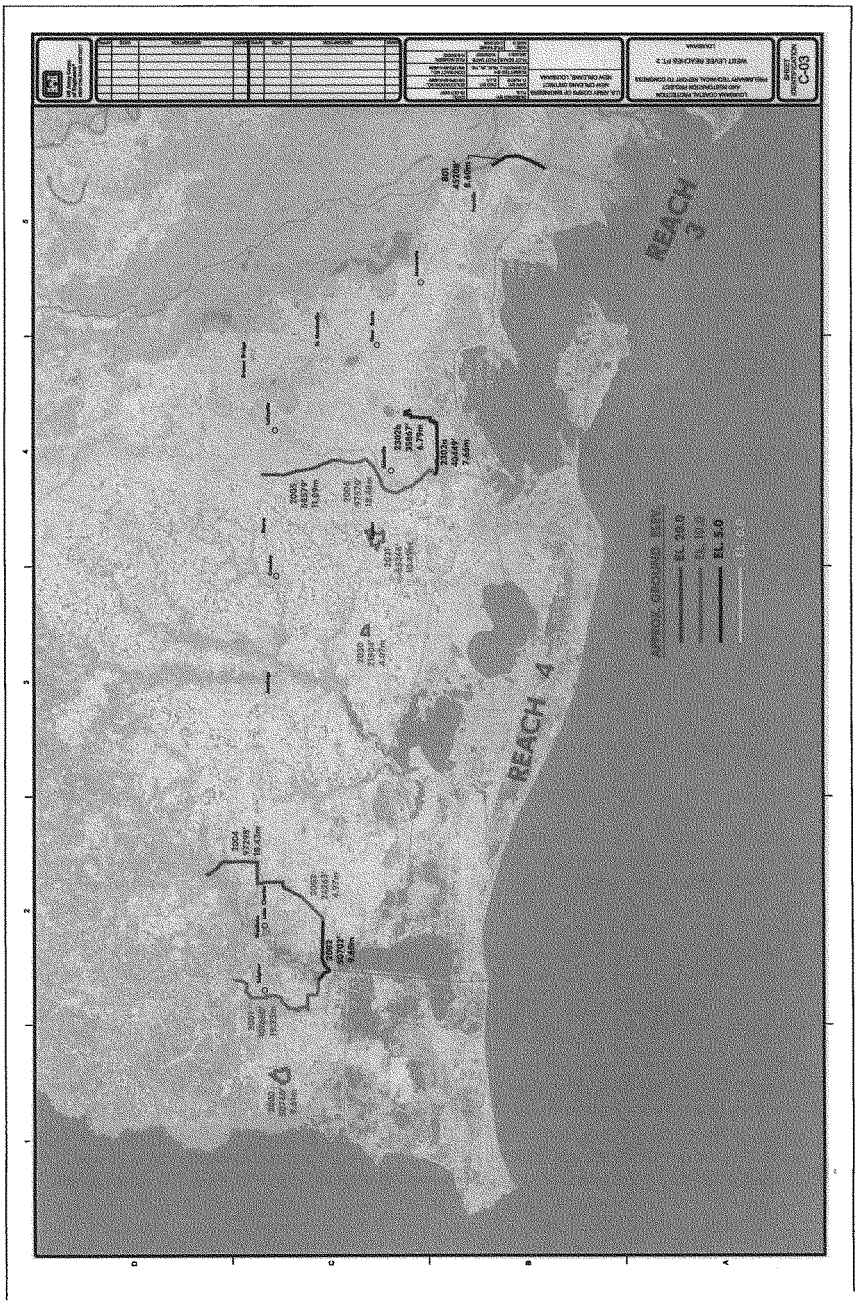
Feasibility level design studies will be required to support any construction recommendations. Once the recommended plan/plans are selected more detailed design efforts will need to be undertaken. These include site specific surveys and borings and alternative studies to determine which method of levee construction (soil mix, geotextile, hollow core or other innovative design) would be best for each area. Design studies will be needed to further develop the hollow core levee concept. Design studies will also look at composite floodwalls. Additional studies will also be needed to determine the impacts of the Lake Pontchartrain Barrier structures and to design the structures so any adverse impacts are minimal. Further studies will also be done to determine the feasibility of other innovative design approaches that were identified at the Engineering Technical Approaches and Innovations Workshop held in March 2006. Advantage will also be taken of the ongoing work being done by USACE on resiliency and armoring.

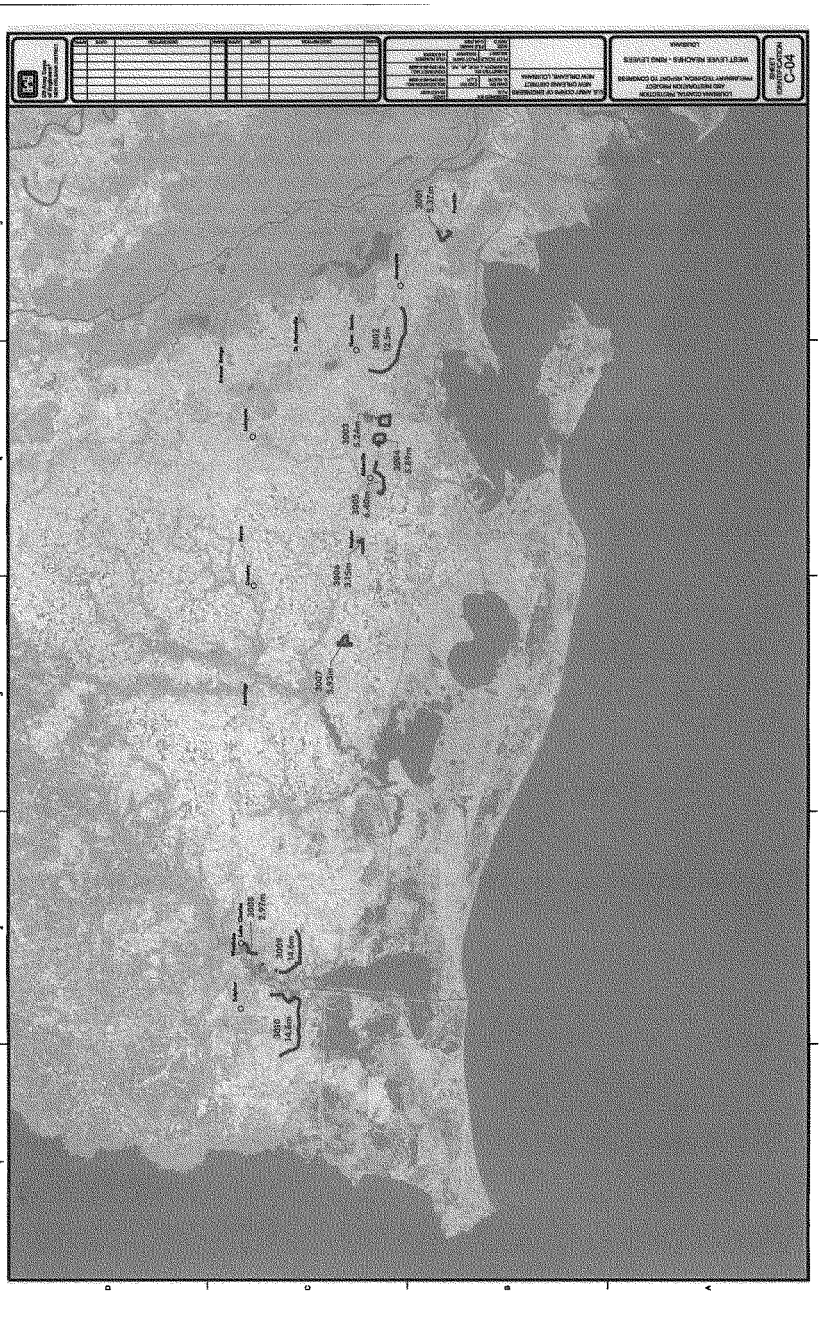
Further design studies will also be necessary to develop detailed designs of the coastal restoration features selected. These include site specific surveys and borings for proposed barrier island restoration, marsh creation/restoration, and ridge restoration features as well as development of detailed borrow plans for these features. Detailed design investigations will also be necessary for any proposed diversions.

PLATES

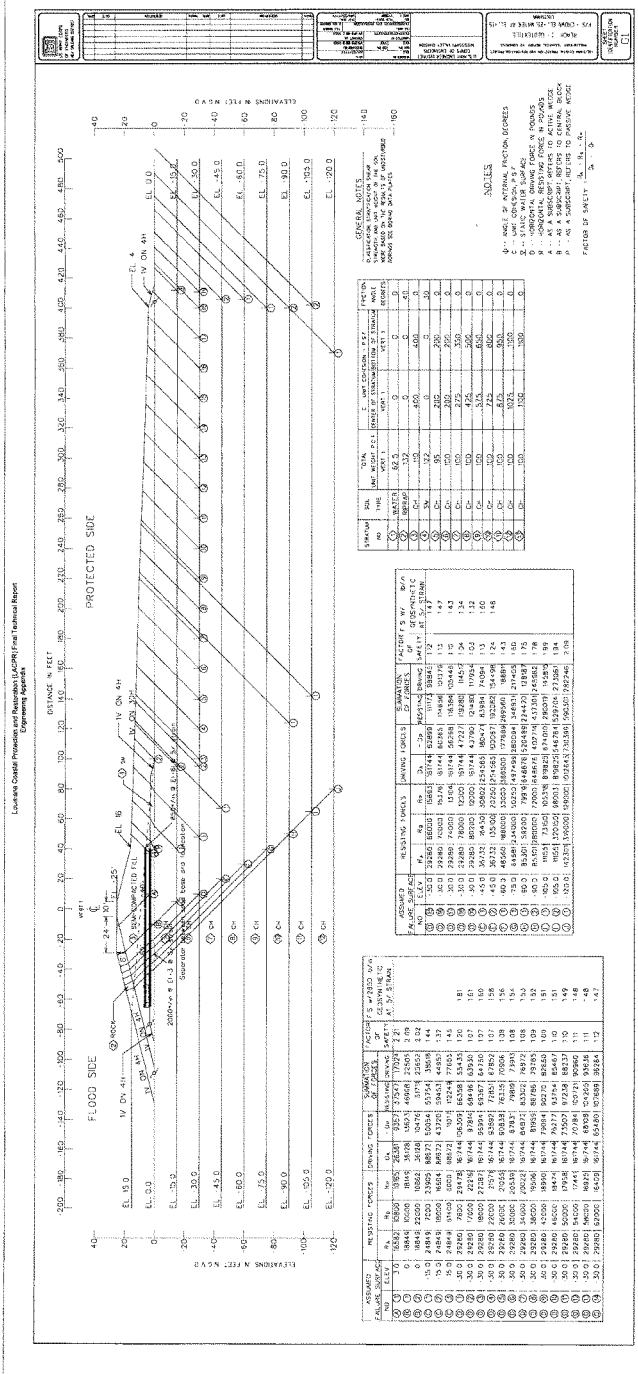
LOUISIANA COASTAL PROTECTION AND RESTORATION (LACPR) PRELIMINARY TECHNICAL REPORT
DRAFT LEVEE HEIGHTS AND BOX HEIGHTS
LOUISIANA
SHEET DESIGNATION C-01
U.S. ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT NEW ORLEANS, LOUISIANA
REACH 4
REACH 3
REACH 2
REACH 1
LEVEE ALIGNMENT
EL 19.0
EL 16.0
EL 13.0
EL 10.0
LEVEE CROSSING

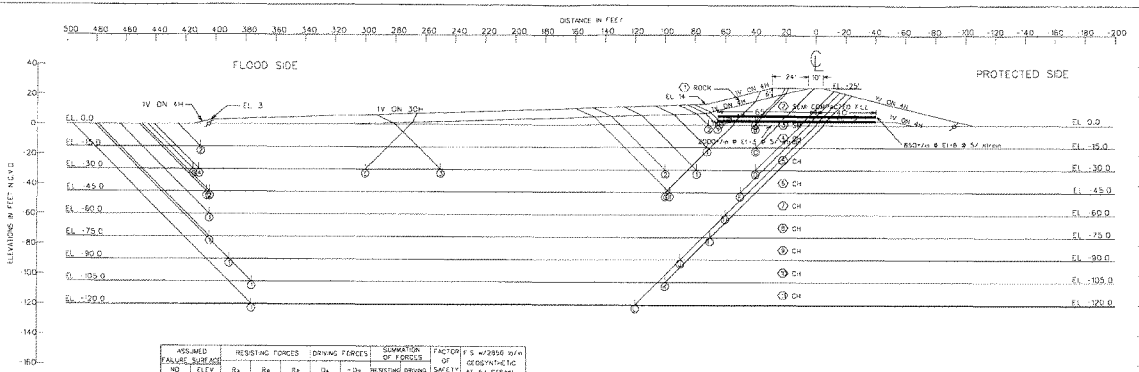

REACH 3
REACH 4
SPECIAL GROUND SITE
EL 20.0
EL 13.0
EL 5.0
EL 0.0
LOUISIANA COASTAL PROTECTION AND RESTORATION FEASIBILITY STUDY
WEST TERRE BEAUCHES PH. 1
LOUISIANA
SHEET
C-02


REACH 4
REACH 3
LOUISIANA COASTAL PROTECTION
AND RESTORATION PROJECT
WEST LAKE READ-RES PT.3
LOUISIANA
C-03



LOUISIANA COASTAL PROTECTION AND RESTORATION PRELIMINARY TECHNICAL REPORT TO CONGRESS
EAST LEVEE REACHES NEW NORTH SHORE
LOUISIANA
SHEET IDENTIFICATION NUMBER
C-05
U.S. ARMY CORPS OF ENGINEERS
LEGEND
Revised Alignment
Section
Covington
Abita Springs
Mandeville
Madisonville
Lacombe
Slidell
Pearl River
Lake Pontchartrain
Miles
0
2.5
5
N


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
PROTECTED SIDE
FLOOD SIDE
DISTANCE IN FEET
ELEVATIONS IN FEET NGVD
GENERAL NOTES
NOTES



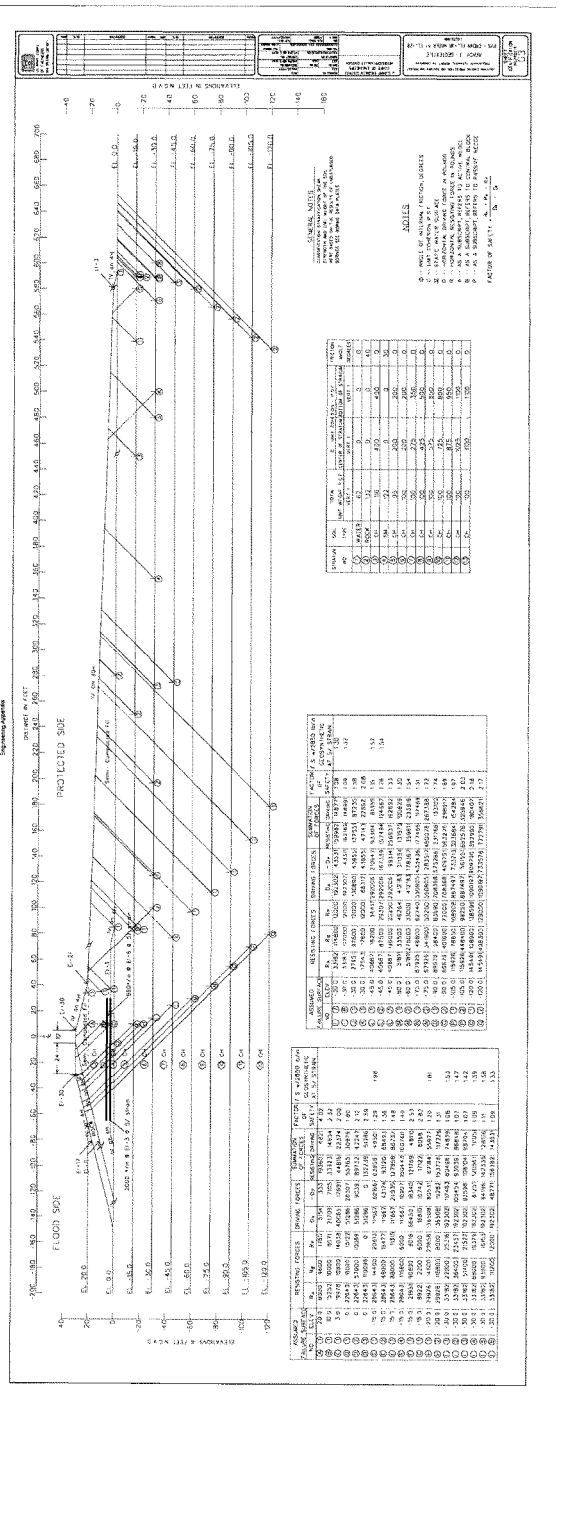
ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	FS w/2950 lb/in GEOSYNTHETIC At 5/ STRAIN
NO		CLEV	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
A	3	3.0	15101	9850	11129	27367	8706	35833	18859	1.92	
B	2	0	20233	5000	14380	35145	29521	39813	22191	1.79	
B	2	0	20233	8200	14215	35145	19915	40638	23233	1.75	
C	1	-15.0	29095	6400	19812	88405	46432	55227	41873	1.32	
D	1	-75.0	29095	74000	6300	88405	70999	109012	77306	1.41	
E	1	-30.0	33455	7800	25244	161878	101418	68489	60827	1.09	1.66
F	1	-30.0	33455	13000	24703	161878	97722	70156	64154	1.09	1.63
G	1	-30.0	33455	42000	20831	161878	75474	95288	86402	1.11	1.51
H	1	-30.0	33456	74200	12000	161878	43575	119655	118301	1.01	1.30
I	1	-30.0	33455	75000	12000	161878	43316	120455	118559	1.02	1.30
J	1	-30.0	15075	21200	12000	73854	43575	48275	30278	1.63	
K	1	-45.0	40859	18450	32643	254441	174394	89952	80177	1.12	1.55
L	1	-45.0	40859	123850	29750	254441	160429	185009	154012	1.20	1.42
M	1	-45.0	38699	107700	20250	203485	99770	164345	103715	1.58	
N	1	-60.0	52809	172000	33000	386982	178052	257899	198118	1.37	
O	1	-75.0	69212	217100	50250	598804	278174	336962	298330	1.54	
P	1	-90.0	91810	240800	72000	840893	405240	404610	234653	1.72	
R	1	-105.0	117212	262200	98250	813493	338585	477652	254908	1.87	
S	1	-120.0	148812	286800	129000	1000275	528204	555910	274089	2.04	

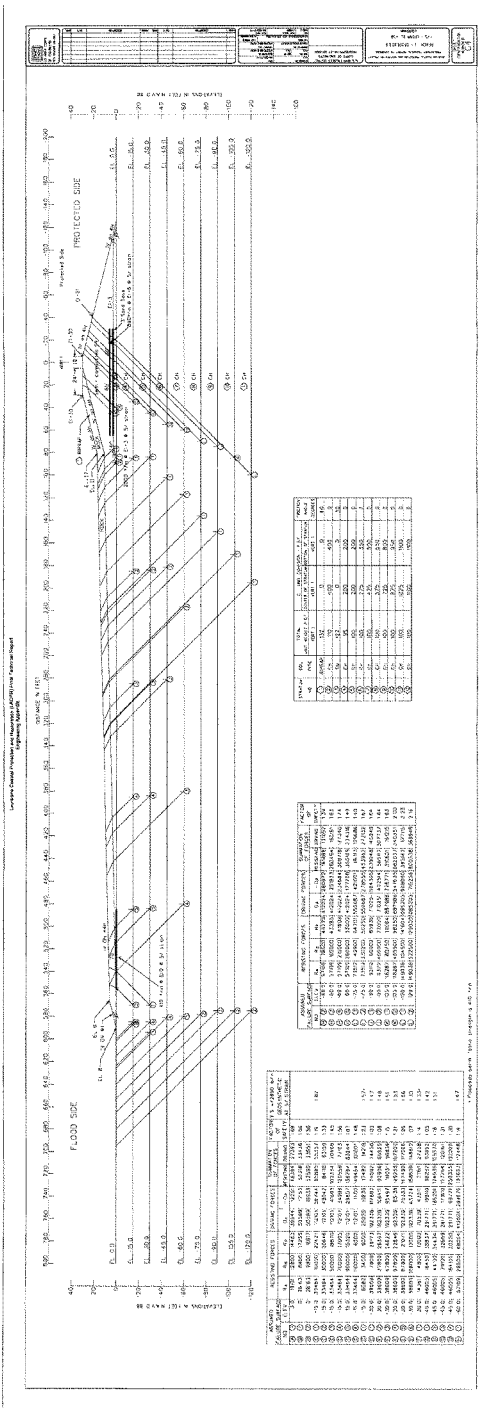
STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F	C = UNIT COHESION - P.S.F.		FRICTION ANGLE
			CENTER OF STRATUM HORIZ	BOTTOM OF STRATUM VERT	DEGREES
1	ROCK	132	0	0	40
2	Gc	110	400	408	0
3	SW	122	0	0	30
A	CH	95	200	200	0
B	CH	100	200	200	0
C	CH	100	275	350	0
D	CH	100	425	500	0
E	CH	100	575	650	0
F	CH	100	725	800	0
G	CH	100	875	950	0
H	CH	100	1025	1100	0
I	CH	100	1100	1100	0

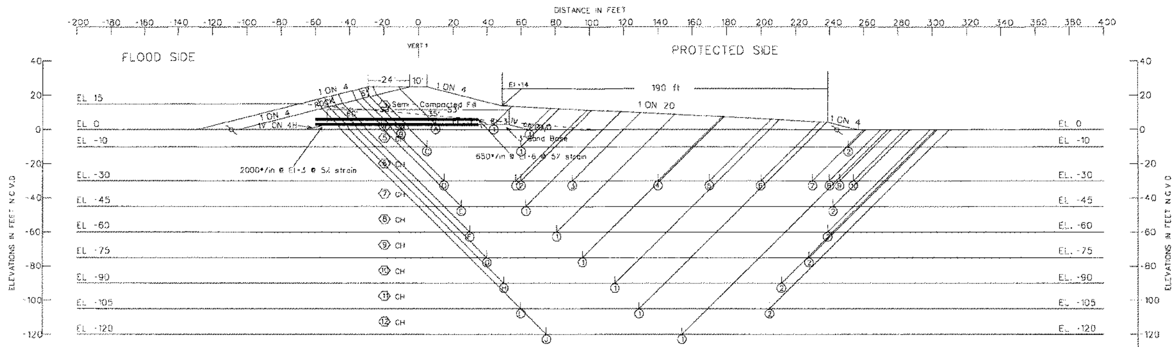
GENERAL NOTES

NOTES

SHEET IDENTIFICATION NUMBER 02







ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. W/2650 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓐ	①	3.0	16582	13600	8375	26382	6920	38757	19462	1.99	
Ⓑ	①	0	18849	15000	10118	36128	9131	43967	26997	1.63	
Ⓒ	①	-10.0	23175	11000	13870	68532	28355	48045	40177	1.20	1.99
Ⓒ	②	-10.0	23175	49200	4000	68532	5511	76375	63021	1.21	1.72
Ⓓ	①	-30.0	34079	12600	25223	163185	95194	71902	67991	1.06	1.53
Ⓓ	②	-30.0	34079	13500	75108	163185	94449	72687	68736	1.06	1.52
Ⓓ	③	-30.0	34079	22500	23965	163185	87698	80544	75487	1.07	1.49
Ⓓ	④	-30.0	34079	37500	22050	163185	77034	93639	86151	1.09	1.46
Ⓓ	⑤	-30.0	34079	46500	20917	163185	70551	101496	92234	1.10	1.45
Ⓓ	⑥	-30.0	34079	55500	19774	163185	65103	109353	98082	1.11	1.44
Ⓓ	⑦	-30.0	34079	64500	16000	163185	54182	114579	109003	1.05	1.34
Ⓓ	⑧	-30.0	34079	67500	16000	163185	49017	117579	114168	1.03	1.31
Ⓓ	⑨	-30.0	34079	69300	16000	163185	46692	119379	116493	1.02	1.30
Ⓓ	⑩	-30.0	34079	71700	16000	163185	45130	121779	118055	1.03	1.30
Ⓔ	①	-45.0	44531	17100	35673	256323	167943	97304	88380	1.10	1.46
Ⓔ	②	-45.0	44531	97650	27250	256323	104381	169431	151942	1.12	1.32
Ⓕ	①	-60.0	58560	30600	50165	370481	256596	139325	113885	1.22	1.50
Ⓕ	②	-60.0	58560	125400	43000	370481	184497	226960	165984	1.22	1.39

ASSUMED FAILURE SURFACE NO		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓖ	①	-75.0	77682	42000	69272	502644	365191	188954	137453	1.37
Ⓖ	②	-75.0	77682	141000	63250	502644	291525	281932	211119	1.34
Ⓗ	①	-90.0	101301	58500	92726	654024	490696	252527	163328	1.55
Ⓗ	②	-90.0	101301	145800	88000	654024	424869	335101	229155	1.46
Ⓘ	①	-105.0	129358	72450	120871	825823	637640	322679	188183	1.71
Ⓘ	②	-105.0	129358	152250	117250	825823	575758	398858	250065	1.60
Ⓙ	①	-120.0	163108	94600	151000	1018619	786460	408908	229731	1.78

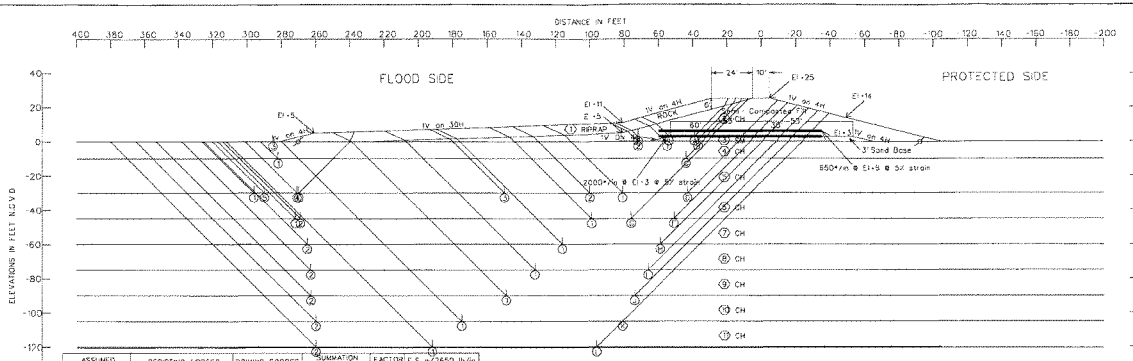
STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT 1	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT 1	BOTTOM OF STRATUM VERT 1	FRICTION ANGLE DEGREES
①	WATER	62.5	0	0	0
②	RIPRAP	132	0	0	40
③	CH	110	400	400	0
④	SM	122	0	0	30
⑤	CH	100	200	200	0
⑥	CH	100	300	300	0
⑦	CH	100	375	450	0
⑧	CH	100	525	600	0
⑨	CH	100	675	750	0
⑩	CH	100	825	900	0
⑪	CH	100	975	1050	0
⑫	CH	100	1125	1200	0
⑬	CH	100	1200	1200	0

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 2 - GEOTEXTILE
P/S - CROWN EL. +25, WATER AT EL. +15
LOUISIANA

SHEET IDENTIFICATION NUMBER



ASSUMED FAILURE SURFACE NO	ELEV	RESISTING FORCES R_A	R_B	R_D	DRIVING FORCES D_A	$-D_W$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. W/2650 lb/in GEOSYNTHETIC AT 5% STRAIN
(A) (1)	30	15178	8000	11408	27618	11018	32524	16600	1.96	
(A) (2)	50	15178	13200	8521	27618	5188	36839	22430	1.64	
(B) (3)	0	20744	3600	18422	35888	14736	42766	21152	2.02	
(B) (4)	0	20744	7000	10990	35888	5796	38834	27092	1.43	
(B) (5)	0	20744	4775	0	35888	0	68494	35888	1.91	
(C) (1)	-10.0	25512	47800	4000	67596	5090	77112	82596	1.23	1.74
(B) (2)	-30.0	37972	11400	27077	153387	89742	75449	73645	1.04	1.47
(B) (3)	-30.0	37972	17100	26586	153387	86918	81658	79459	1.07	1.45
(B) (3)	-30.0	37972	32100	23295	153387	73691	93357	83696	1.14	1.48
(B) (5)	-30.0	37972	58400	16000	153387	46848	122372	116387	1.05	1.32
(B) (5)	-30.0	37972	74100	16000	153387	44998	128372	118387	1.08	1.35
(C) (3)	-30.0	17704	7800	16000	66603	44998	41504	21605	1.92	
(C) (4)	-45.0	48023	21600	37475	256779	159840	107098	96939	1.10	1.43
(C) (5)	-45.0	45023	98100	27250	256779	103622	173373	153157	1.13	1.34
(C) (6)	-45.0	49649	86200	27250	235869	102583	165099	133286	1.24	1.48
(C) (7)	-60.0	62650	34200	52399	369682	250013	149249	119669	1.25	1.49
(C) (8)	-60.0	62650	123900	43000	369682	184217	229250	185465	1.24	1.41
(C) (9)	-75.0	81352	49500	71848	502171	360415	202703	141756	1.43	
(C) (10)	-75.0	81352	147750	63250	502171	286585	292352	215586	1.36	
(C) (11)	-90.0	104598	67500	95284	653717	490474	267382	163243	1.64	
(C) (12)	-90.0	104598	170100	88000	653717	410337	362698	243385	1.49	
(C) (13)	-105.0	132128	98700	117260	825281	629592	348088	195689	1.78	
(C) (14)	-105.0	132128	187950	117250	825281	558493	437328	266788	1.64	
(C) (15)	-120.0	165878	115200	151000	1013741	782028	432078	231713	1.86	
(C) (16)	-120.0	165878	196800	151000	1013741	727248	515678	285513	1.79	

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
(1)	RIPRAP	132	0	0	40
(2)	CH	110	400	400	0
(3)	SM	122	0	0	30
(4)	CH	100	200	200	0
(5)	CH	100	300	300	0
(6)	CH	100	375	450	0
(7)	CH	100	525	600	0
(8)	CH	100	675	750	0
(9)	CH	100	825	900	0
(10)	CH	100	975	1050	0
(11)	CH	100	1125	1200	0
(12)	CH	100	1200	1200	0

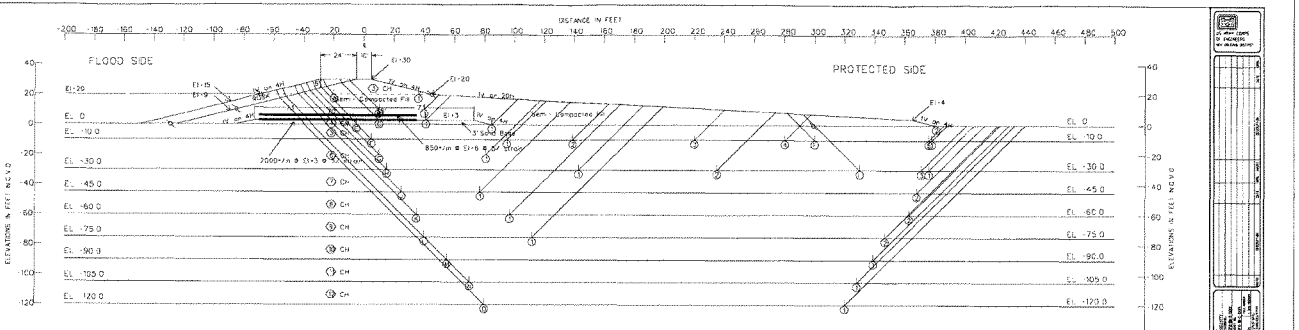
GENERAL NOTES

CLASSIFICATION SYMBOLOGY SYSTEM
STRENGTH AND UNIT WEIGHT OF THE SOIL
WERE BASED ON THE RESULTS OF LABORATORY
BORINGS SEE BORING DATA PLATES

NOTES

Φ - ANGLE OF INTERNAL FRICTION DEGREES
C - UNIT COHESION, P.S.F.
W - STATIC WATER SURFACE
R - HORIZONTAL RESISTING FORCE IN POUNDS
A - AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
B - AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P - AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_W}$



ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES R_A	R_B	DRIVING FORCES R_P	D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/2850 lb/ft GEOSYNTHETIC AT 3% STRAIN
A	1	20.0	5282	22406	1446	6256	223	29132	8033	4.83	
B	1	10.0	13252	12058	7869	21726	5778	25081	15933	2.20	
C	1	3.0	19978	12408	13705	40365	15671	45483	24394	1.86	
D	1	0	22643	18009	13714	51286	16970	54335	34338	1.58	
E	2	0	22643	71894	0	51286	0	94537	51286	1.84	
K	7	-10.0	26643	18009	16952	88648	3978	61595	50635	1.22	1.89
F	2	-10.0	26643	36900	15216	88648	33025	68719	86629	1.21	1.82
G	3	10.0	26643	43000	12980	88648	23150	81823	66498	1.23	1.74
H	4	-10.0	26643	55000	9903	88648	16942	91646	77704	1.26	1.73
K	5	10.0	26643	74200	4200	88648	534.3	104843	84303	1.24	1.65
F	3	10.0	10575	36900	4200	16723	3124	30423	17099	2.60	
G	3	20.0	31827	21300	23105	137448	74457	153127	62989	1.21	1.75
L	3	-30.0	37182	38400	26361	193990	102038	181948	91952	1.11	1.48
M	4	-30.0	37182	66000	22856	193990	89410	126038	112580	1.12	1.42
H	4	-30.0	37182	126800	16000	193990	46373	159962	141619	1.08	1.32
I	5	-30.0	27704	13800	16000	99487	4534.3	51904	24140	2.15	
J	5	-45.0	47687	234400	39554	294815	200519	170641	94307	1.17	1.54
F	4	-45.0	47687	154350	27250	294815	103570	223237	191249	1.20	1.38
N	5	-60.0	62653	37200	15911	414848	259780	153864	119258	1.29	1.58
O	5	-60.0	62653	196800	43019	414848	184403	307483	230445	1.31	

ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES R_A	R_B	DRIVING FORCES R_P	D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
G	1	-75.0	99926	340000	73578	555987	411645	708004	484128	1.46
K	2	-75.0	88926	290250	63250	555987	293567	714426	362403	1.43
M	1	-90.0	103678	256500	98050	713319	429800	449276	295198	1.64
N	1	-105.0	134925	270900	117250	891650	574789	837076	516871	1.65
M	1	-120.0	167491	288000	151500	1093982	748857	606491	345125	1.76

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT - PCF	C - UNIT COHESION - PSF CENTER OF STRATUM	BOTTOM OF STRATUM	FRICTION ANGLE DEGREES
		WET +	WET +	WET +	
1	WATER	62	0	0	0
2	RIPRAP	132	0	0	40
3	CH	110	400	650	0
4	SM	122	0	0	30
5	CH	100	200	230	0
6	CH	100	300	400	0
7	CH	100	375	450	0
8	CH	100	425	600	0
9	CH	120	575	750	0
10	CH	100	825	900	0
11	CH	100	975	1050	0
12	CH	100	1125	1200	0
13	CH	100	1200	1200	0

GENERAL NOTES

CLASSIFICATION STRATIFICATION, SHEAR STRENGTH AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS AND BORING DATA PLOTS.

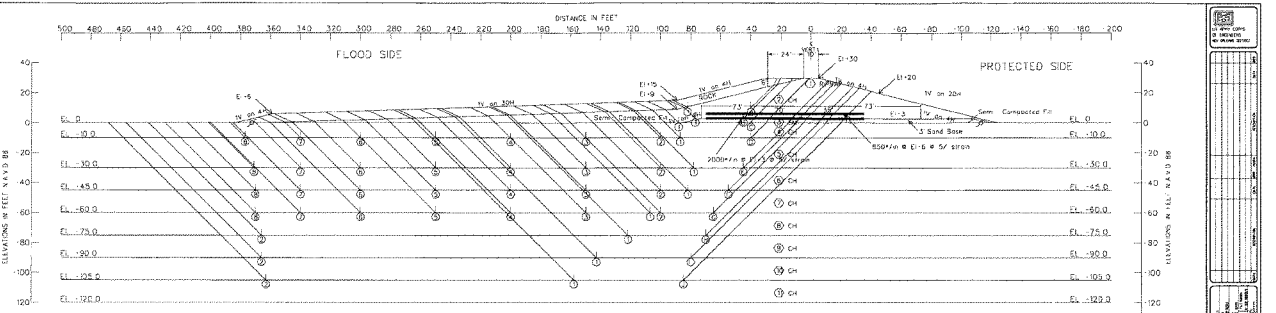
NOTES

ϕ - ANGLE OF INTERNAL FRICTION, DEGREES
C - UNIT COHESION, P.S.F.
W - STATIC WATER SURFACE
D - HORIZONTAL DRIVING FORCE IN POUNDS
R - HORIZONTAL RESISTING FORCE IN POUNDS
A - AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B - AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P - AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

PAGE 7 - SEGMENT 1A
SCALE: 1" = 100'

PAGE IDENTIFICATION NUMBER 07



ASSUMED FAILURE SURFACE		RESISTING FORCES		DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/2850 lb/in GEOSYNTHETIC AT 5% STRAIN
NO	ELEV	R_x	R_B	R_A	D_x	$-D_B$	RESISTING	DRIVING	
A	10	13213	16800	638	22542	20883	362710	204658	1.77
B	3.3	19101	12800	11787	39644	10933	436688	26711	1.52
C	0	28163	9600	14973	50589	13456	507361	37094	1.37
D	-10.3	33751	9400	18741	59726	16280	537018	51328	1.12	1.76
E	-10.0	32151	12000	16403	59435	24677	623361	54908	1.14	1.76
F	-10.3	32151	22000	17914	59435	30040	712851	59305	1.23	1.78
G	-10.0	32151	32000	15822	59435	29800	799731	63535	1.28	1.83
H	-10.3	32151	43000	14431	59435	27053	886182	67386	1.32
I	-10.0	32151	52000	12239	59435	25071	973901	70928	1.37
J	-10.0	32151	60000	12028	59435	25830	1041761	73605	1.42
K	-10.0	32151	67400	4090	59435	25841	1135511	83841	1.24	1.64
L	-30.0	42609	9900	25488	194024	11885	823936	82139	1.01	1.43
M	-30.0	42609	16500	29686	194024	10878	889971	87260	1.02	1.41
N	-30.0	42809	31500	28587	194024	9877.8	1017061	95745	1.06	1.44
O	-30.0	42809	46500	27305	194024	9089.9	1164144	102935	1.13	1.46
P	-30.0	42609	61500	26014	194024	8369.4	1302125	109330	1.19	1.49
Q	-30.0	42909	76500	24723	194024	7659.1	1438121	115477	1.25	1.52
R	-30.0	42609	88500	17632	194024	57869	1487411	126155	1.18	1.45
S	-30.0	42609	97800	16000	194024	47374	1584091	146550	1.07	1.30
T	-45.0	53005	12150	4128	294584	980799	1063781	1002	1.35

ASSUMED FAILURE SURFACE		RESISTING FORCES		DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/2850 lb/in GEOSYNTHETIC AT 5% STRAIN
NO	ELEV.	R_x	R_B	R_A	D_x	$-D_B$	RESISTING	DRIVING	
U	-45.0	53005	20250	40251	294584	186797	1140086	1086767	1.05	1.37
V	-45.0	53005	42750	39482	294584	175693	1351286	1189524	1.14	1.42
W	-45.0	53005	65250	38788	294584	165298	1564233	1292186	1.21	1.47
X	-45.0	53005	87750	38877	294584	155242	1776132	1393432	1.27	1.52
Y	-45.0	53005	110250	39386	294584	145482	1988041	1493009	1.33
Z	-45.0	53005	128250	27250	294584	110645	2085696	1681311	1.24	1.46
AA	-45.0	53005	141750	27250	294584	104035	2220098	1790549	1.17	1.34
BB	60.0	67909	25200	55933	414349	285649	1490421	1287008	1.16	1.42
CC	-90.0	67909	21000	35114	414349	267531	1450213	1286818	1.13	1.41
DD	-90.0	67909	51000	54823	414349	274222	1737321	1410127	1.24	1.48
EE	-60.0	67909	81000	53531	414349	261208	2074401	1531411	1.32
FF	-60.0	67909	111000	52239	414349	248493	2311918	1658839	1.39
GG	-60.0	67909	141000	48110	414349	235284	2570719	1780565	1.44
HH	-60.0	67909	153000	43000	414349	205700	2719079	2019449	1.32
II	-60.0	67909	183000	43000	414349	182783	2939079	2315586	1.27	1.42
JJ	-75.0	86512	39000	75408	555749	403401	3009020	1523648	1.32
KK	-75.0	86512	222000	63250	555749	286009	3717152	2697143	1.38
LL	-90.0	101187	36000	59228	714531	539358	2881751	1754621	1.52
MM	-90.0	101187	257400	88000	714521	409756	4558187	3047365	1.49
NN	105.3	137287	76650	127704	893098	698762	3416481	1983310	1.72
OO	105.3	137287	291900	117250	893098	5578331	5484371	3337365	1.62

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT PCF	C - UNIT COHESION - PSF		FRICTION ANGLE DEGREES
			CENTER OF STRATUM VERT 1	BOTTOM OF STRATUM VERT 1	
1	RIPRAP	132	0	0	40
2	CH	110	400	400	0
3	SM	122	0	0	30
4	CH	100	200	200	0
5	CH	100	300	300	0
6	CH	100	375	450	0
7	CH	100	525	600	0
8	CH	100	675	750	0
9	CH	100	825	900	0
10	CH	100	975	1050	0
11	CH	100	1125	1200	0

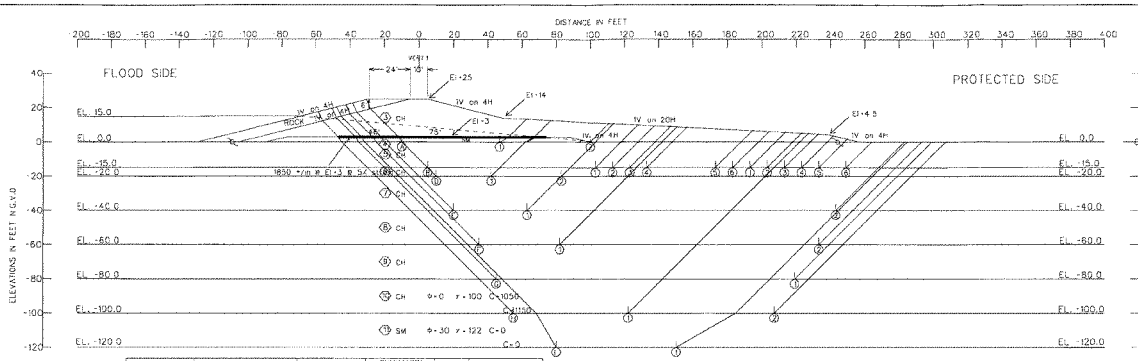
GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTH AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED SAMPLES SEE BORING DATA PLATES

NOTES

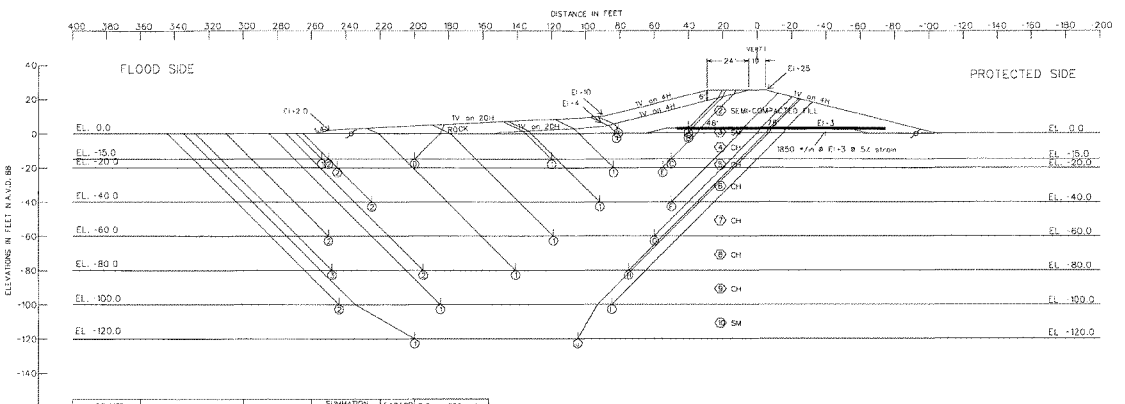
ϕ = ANGLE OF INTERNAL FRICTION, DEGREES
C = UNIT COHESION, P.S.F.
M = STATIC WATER SURFACE
D = HORIZONTAL DRIVING FORCE IN POUNDS
R = HORIZONTAL RESISTING FORCE IN POUNDS
A = AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B = AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P = AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_x + R_B + R_P}{D_x - D_B}$



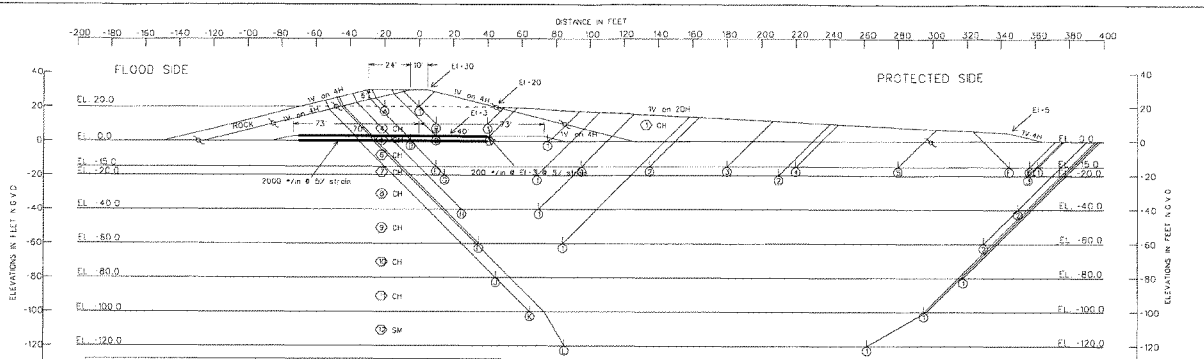
ASSUMED FAILURE SURFACE NO	ASSUMED FAILURE SURFACE ELEV	RESISTING FORCES Ra	RESISTING FORCES Rs	RESISTING FORCES Re	DRIVING FORCES Da	DRIVING FORCES - De	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY	F.S. w/1850 lb/cu.ft. GEOSYNTHETIC AT 5% STRAIN
A	1	0	18848	11400	15394	36128	10330	45842	25798	1.77	
A	2	0	18848	22000	8716	36128	6855	49564	29273	1.69	
B	1	-15.0	24848	19600	14030	88672	34514	58478	54158	1.08	1.49
B	2	-15.0	24848	21600	13649	88672	33150	60097	55322	1.08	1.48
B	3	-15.0	24848	23600	13268	88672	3187	61736	56861	1.09	1.48
B	4	-15.0	24848	25600	12887	88672	30500	63335	58172	1.09	1.47
B	5	-15.0	24848	33600	11363	88672	25514	69811	63158	1.11	1.46
B	6	-15.0	24848	35600	10981	88672	24333	71429	64339	1.11	1.46
C	1	-15.0	24848	37600	10600	88672	23179	73048	65493	1.12	1.45
C	2	-15.0	24848	39600	10219	88672	22050	74867	66622	1.12	1.45
C	3	-15.0	24848	41600	9838	88672	20946	76286	67724	1.13	1.45
C	4	-15.0	24848	43600	9007	88672	19757	77455	68915	1.12	1.45
C	5	-15.0	24848	45600	7407	88672	17250	77855	71422	1.09	1.40
C	6	-15.0	24848	48800	5000	88672	11523	78648	77149	1.03	1.32
D	1	-20.0	28099	11200	24021	110630	59980	63320	50650	1.25	1.69
D	2	-20.0	28099	25550	17652	110630	52070	71503	58569	1.22	1.60
E	1	-40.0	44527	23650	35852	221081	139429	104029	81656	1.27	1.95
E	2	-40.0	44527	122650	27250	221081	80177	194427	140904	1.38	
F	1	-60.0	69727	35250	50266	364634	252597	165343	112537	1.48	
F	2	-60.0	69727	148500	53250	364634	183472	271477	181162	1.50	
G	1	-80.0	101672	165300	87250	544565	329882	354222	214683	1.65	
G	2	-100.0	141343	77050	133317	759820	580786	351710	179054	1.96	
G	3	-100.0	141343	174800	129250	759820	516017	645393	243803	1.83	
H	1	-120.0	218316	243456	335556	1003046	773020	787330	230020	3.47	

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
1	WATER	62.5	0	0	0
2	RIPRAP	132	0	0	40
3	CH	110	400	400	0
4	SM	122	0	0	32
5	CH	95	200	200	0
6	CH	100	325	350	0
7	CH	100	450	550	0
8	CH	100	650	750	0
9	CH	100	850	950	0
10	CH	100	1050	1150	0
11	SM	122	0	0	32
12	SM	122	0	0	30



ASSUMED FAILURE SURFACE NO	ELEV	RESISTING FORCES Ra	Rg	Rp	DRIVING FORCES Da	-Dp	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1850 lb/in GEOSYNTHETIC AT 5% STRAIN
A	3.0	15113	16400	2743	27308	3998	35256	23372	1.68	
B	0	20249	18400	10266	35145	7048	38315	28098	1.39	
C	-15.0	27349	14000	15039	85061	29534	56388	55427	1.02	1.42
D	-15.0	27349	40000	8000	85061	19514	73349	73447	1.00	1.30
E	-15.0	7443	10800	6000	22131	10889	24243	11242	2.15	
F	-20.0	30599	10150	19467	104811	48712	60216	56099	1.07	1.47
G	-20.0	30599	66500	9250	104811	21370	106349	83441	1.27	1.54
H	-40.0	48704	23100	36403	220928	125612	106207	95296	1.11	1.37
I	-40.0	48704	58250	27250	220928	87257	172204	133871	1.29	1.45
J	-60.0	73060	44250	59798	306243	232284	177106	133959	1.32	
K	-60.0	73060	142500	53250	386243	176884	268810	189259	1.42	
L	-80.0	108212	62700	85764	543815	372239	258676	171556	1.51	
M	-80.0	108212	114000	87250	543815	339766	307462	204029	1.51	
N	-80.0	108212	164350	87250	543815	336011	357812	227804	1.57	
O	-100.0	146063	115000	129250	758409	524846	390311	233563	1.67	
P	-100.0	146063	182350	129250	758409	491653	458183	292756	1.74	
Q	-120.0	220521	315207	319082	997748	731167	854810	266581	3.21	

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION P.S.F. CENTER OF STRATUM VERT.	BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
1	REFRAP	132	C	0	40
2	CH	110	400	400	0
3	SM	122	0	0	30
4	CH	95	200	200	0
5	CH	100	325	350	0
6	CH	100	450	550	0
7	CH	100	650	750	0
8	CH	100	850	950	0
9	CH	100	1050	1150	0
10	SM	122	C	0	30
11	SM	122	0	0	30



ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S w/2200 lb/in GEOSYNTHETIC AT 5% STRAIN
A	1	20.0	5292	8050	7199	6258	5723	20491	1535	13.84	
B	1	10.0	15252	12000	7599	21709	5778	35061	15931	2.20	
C	1	3.0	19978	12400	13104	40065	15671	45482	24394	1.86	
K	1	0	22643	16000	15611	51286	17835	54254	33451	1.62	
E	2	-15.0	28643	17000	18762	111667	53537	64405	58230	1.11	1.56
F	2	-15.0	28643	25000	17238	111667	47036	70881	64631	1.10	1.51
G	3	-15.0	28643	34000	15523	111667	40112	78166	71555	1.09	1.46
H	4	-15.0	28643	42000	13999	111667	34402	84642	77265	1.10	1.44
F	5	-15.0	28643	54000	11713	111667	28823	94356	85044	1.11	1.42
G	6	-15.0	28643	69400	8000	111667	11567	104043	100100	1.04	1.30
M	1	-15.0	10840	5400	6000	21465	10811	20245	10654	1.90	
N	1	-20.0	31893	18900	22812	135689	76275	73805	59414	1.24	1.68
O	1	-20.0	31893	68250	17640	135689	49860	117583	85829	1.37	
Q	5	-20.0	31893	79350	9250	135689	20175	160493	115514	1.39	
H	5	-40.0	48432	24750	40010	255783	170575	113196	85108	1.33	
H	2	-40.0	48432	178750	27250	255783	80652	254632	175131	1.45	
L	1	-60.0	72943	35750	64715	410911	297147	174409	113764	1.53	
L	2	-60.0	72943	221250	53250	410911	190420	347443	220491	1.58	
L	3	-60.0	104926	295350	87250	600890	338900	451526	261990	1.72	
R	1	-100.0	149276	264500	79250	822931	552910	540676	270021	1.96	
L	1	-120.0	217453	393044	315769	1080986	772679	1096266	308287	3.56	

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	BOTTOM OF STRATUM VERT. 1	FRICTION ANGLE DEGREES
1	CH	110	400	400	0
2	WATER	62	0	0	0
3	RIPRAP	132	0	0	40
4	CH	110	400	400	0
5	SM	122	0	0	30
6	CH	95	200	200	0
7	CH	100	325	350	0
8	CH	100	450	550	0
9	CH	100	650	750	0
10	CH	100	850	950	0
11	CH	100	1050	1150	0
12	SM	122	0	0	30
13	SM	122	0	0	30

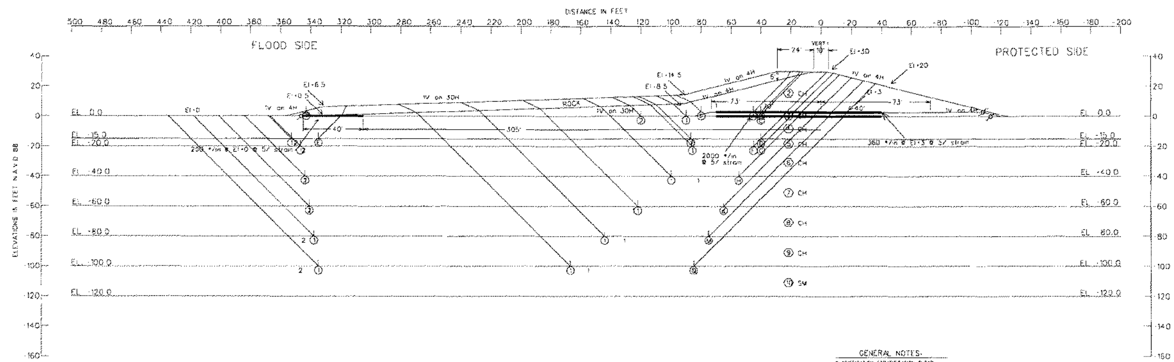
GENERAL NOTES

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTH, AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS SEE BORING DATA PLATES

NOTES

Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_P + R_B + R_P}{D_A - D_P}$



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/2360 lb/in GEOSYNTHETIC AT 5% STRAIN
NO	ELEV	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓐ ①	3.0	19101	14000	11391	39644	9918	44492	29726	1.50	
Ⓑ ①	3.0	19832	121200	391	41226	-253	141423	41480	3.41	
Ⓒ ①	0	26163	10000	14586	50589	12645	50749	37944	1.34	
Ⓒ ②	0	26163	16000	13811	50589	11011	55974	39578	1.41	
Ⓓ ①	-15.0	34116	9400	20276	111779	48560	63792	63219	1.01	1.45
Ⓔ ①	-15.0	8604	3600	6000	26543	10951	18204	15592	1.17	1.30*
Ⓕ ①	-20.0	37366	14350	23423	135636	65286	75139	70350	1.07	1.47
Ⓖ ①	-20.0	36714	107450	9250	136646	20711	153414	115935	1.32	
Ⓗ ①	-40.0	53859	24750	40546	255486	152372	119155	103114	1.16	1.40
Ⓘ ①	-40.0	53859	158950	27250	255486	80348	240059	175138	1.37	
Ⓙ ①	-60.0	78159	42750	65461	410411	274247	186370	136164	1.37	
Ⓚ ①	-60.0	78159	207000	53250	410411	180280	338409	230131	1.47	
Ⓛ ①	-80.0	110512	65550	98377	600413	431636	274439	168777	1.63	
Ⓜ ①	80.0	110512	249850	87250	600412	320508	447612	279904	1.60	
Ⓝ ①	-100.0	150362	94300	139267	824941	624136	383929	200805	1.91	
Ⓞ ①	-100.0	150362	287500	129250	824941	501032	567112	323909	1.75	

* FABRIC STRENGTH AT TOE OF BERM IS 200#/IN

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.	C - UNIT COHESION - P.S.F.		FRICTION ANGLE DEGREES
			CENTER OF STRATUM	BOTTOM OF STRATUM	
		VERT 1	VERT 1	VERT 1	
①	RIPRAP	132	0	0	40
②	CH	110	400	400	0
③	SM	122	0	0	30
④	CH	95	200	200	0
⑤	CH	100	325	350	0
⑥	CH	100	450	550	0
⑦	CH	100	650	750	0
⑧	CH	100	850	950	0
⑨	CH	100	1050	1150	0
⑩	SM	122	0	0	30

GENERAL NOTES:

CLASSIFICATION, STRATIFICATION, SHEAR STRENGTH, AND UNIT WEIGHT OF THE SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS. SEE BORING DATA PLATES.

NOTES

- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

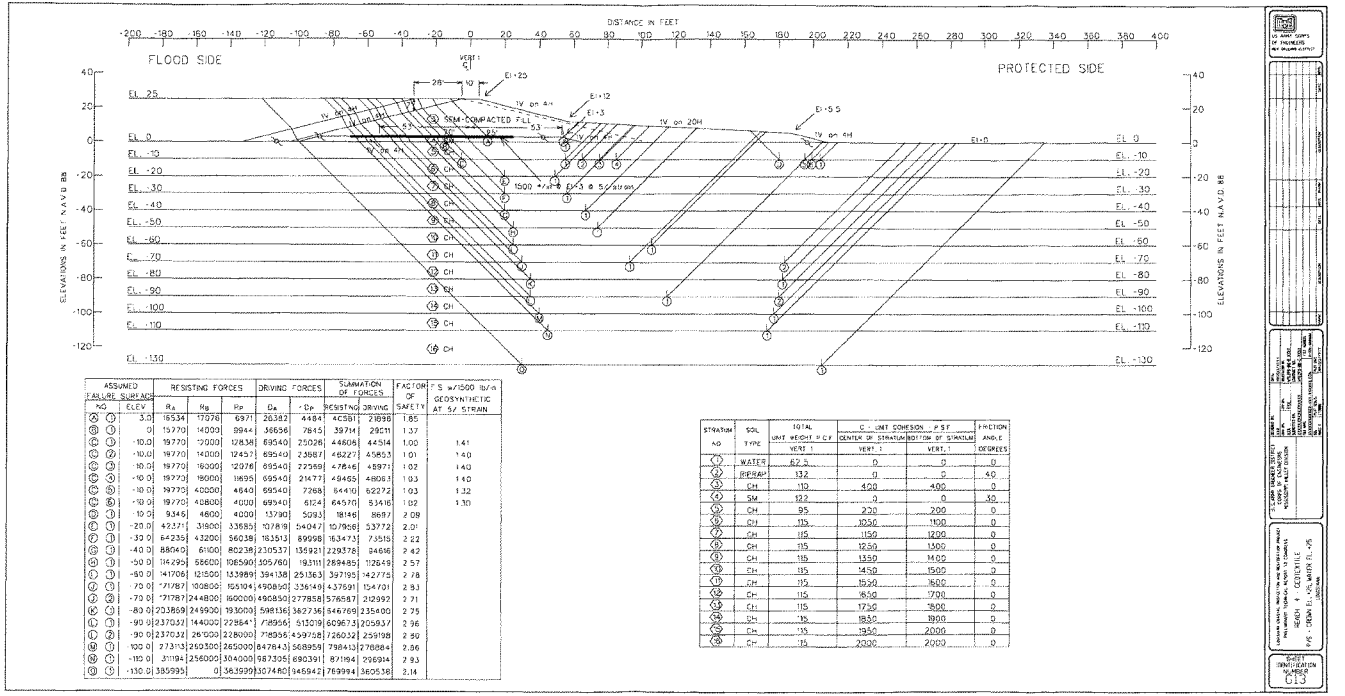
$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT, CORPS OF ENGINEERS, MISSISSIPPI VALLEY DIVISION

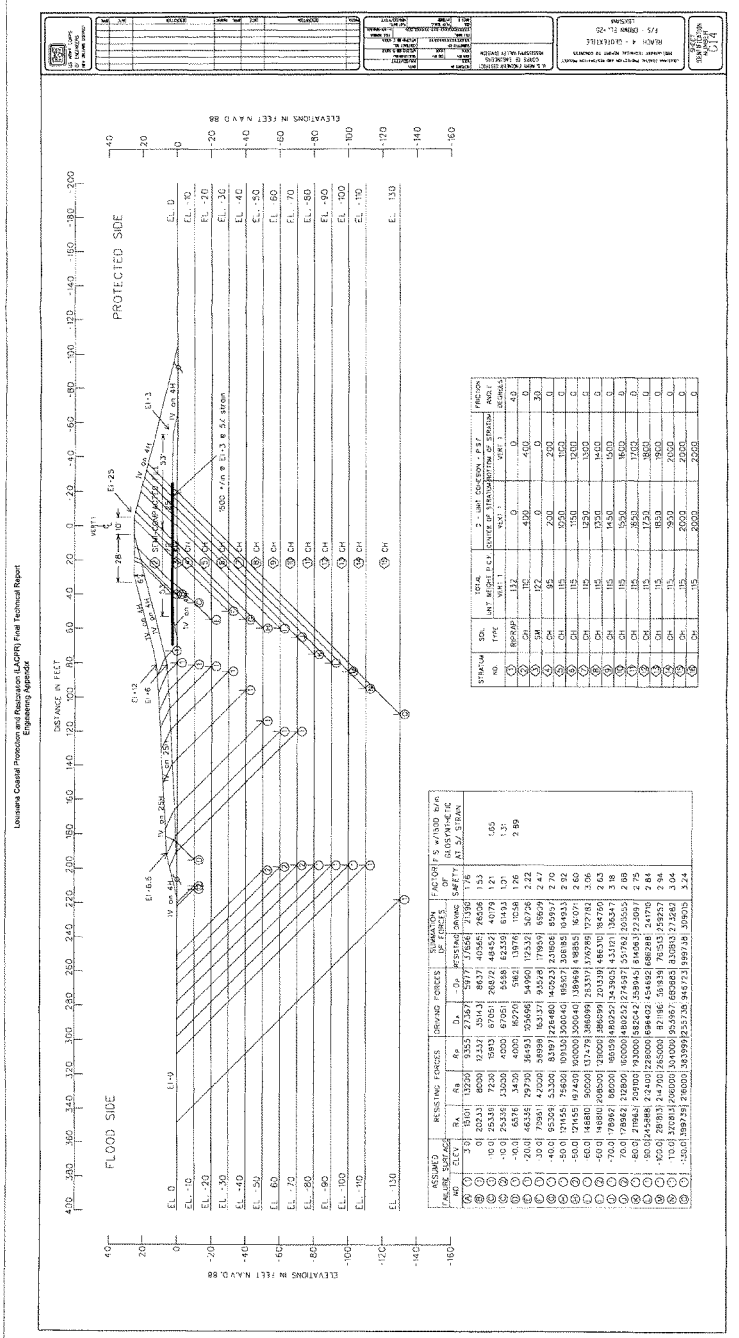
REACH 3 - GEOTEXTILE
F/S - CROWN EL. +30
LOUISIANA

SHEET IDENTIFICATION NUMBER G12



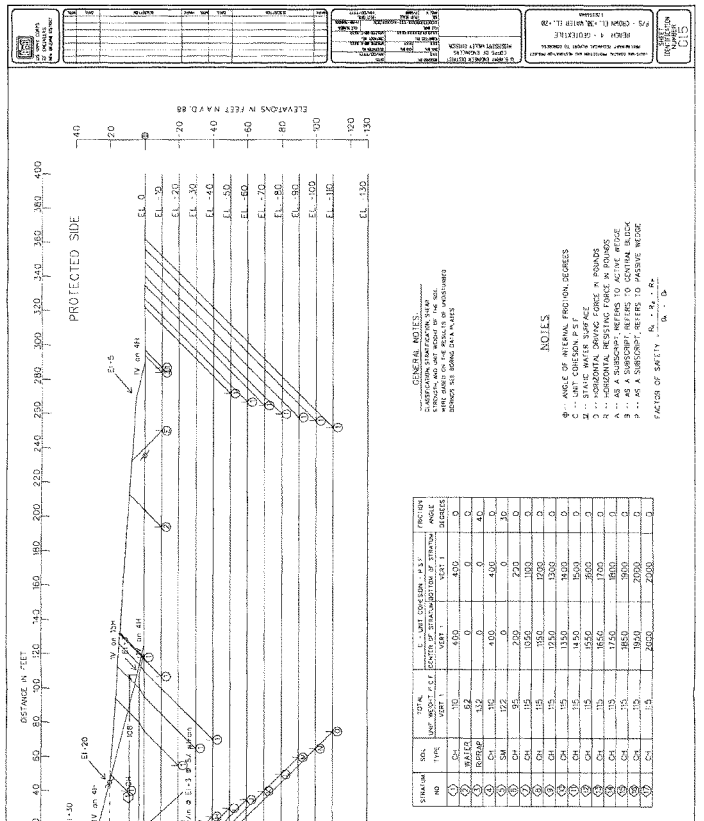
ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES Ra	Rs	Rp	DRIVING FORCES Da	-Dp	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F S w/1500 lb/ft GEOSYNTHETIC AT 5% STRAIN
A	1	5.0	16534	17076	6971	26383	4484	40581	21898	1.85	
B	1	0.0	15770	14000	8944	36656	7845	39714	29511	1.37	
C	1	-10.0	19770	12000	12838	69540	25026	44608	44514	1.00	1.41
D	1	-10.0	19770	14000	12457	69540	23887	46227	45853	1.01	1.40
E	1	-10.0	19770	16000	12078	69540	22569	47848	46971	1.02	1.40
F	1	-10.0	19770	18000	11695	69540	21477	49465	48063	1.03	1.40
G	1	-10.0	19770	40000	4640	69540	7268	64410	62272	1.03	1.32
H	1	-10.0	19770	40800	4000	69540	6124	64570	63416	1.02	1.30
I	1	-10.0	9345	4800	4000	13790	5093	18146	8883	2.04	
J	1	-20.0	42371	31800	33685	107818	54047	107856	53772	2.01	
K	1	-30.0	84235	43200	56038	183513	89998	163473	73515	2.22	
L	1	-40.0	88049	61100	80238	230537	135921	229387	94616	2.42	
M	1	-50.0	116295	58600	106590	305760	193111	289485	112649	2.57	
N	1	-60.0	141706	121500	133989	394138	251363	397195	142775	2.78	
O	1	-70.0	171787	100800	155104	490850	336149	437691	154701	2.83	
P	1	-70.0	171787	244800	160000	490850	277858	576587	312992	2.71	
K	2	-80.0	203859	249900	193003	598136	362736	646769	284060	2.75	
L	2	-90.0	237032	144000	228641	768966	513019	609673	205937	2.96	
O	2	-90.0	237032	261000	228000	768966	459758	726032	259198	2.80	
M	1	100.0	273113	260300	265000	847843	568959	798413	276884	2.86	
N	1	-110.0	311194	256000	304000	987305	690391	871194	296914	2.93	
O	1	-130.0	385995	0	383999	1307480	945942	769994	360538	2.14	

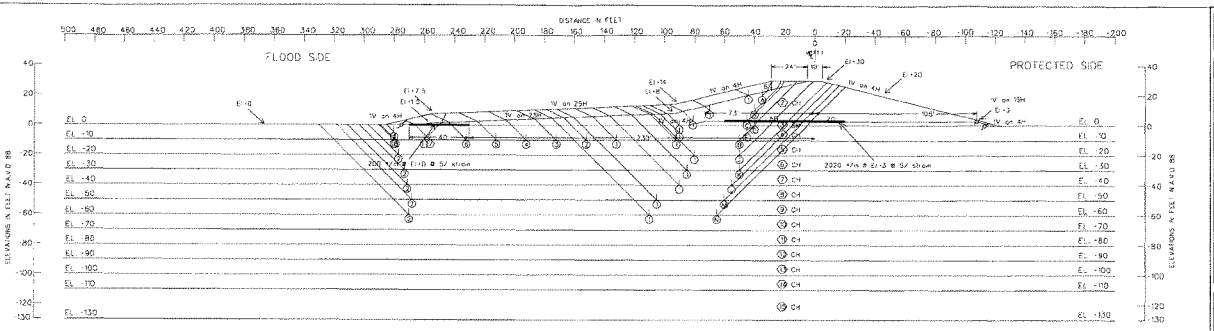
STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.	BOTTOM OF STRATUM VERT.	FRICTION ANGLE DEGREES
1	WATER	62.5	0	0	0
2	RIPRAP	132	0	0	40
3	CH	110	400	400	0
4	SM	122	0	0	30
5	CH	95	220	200	0
6	CH	115	1050	1100	0
7	CH	115	1150	1200	0
8	CH	115	1250	1300	0
9	CH	115	1350	1400	0
10	CH	115	1450	1500	0
11	CH	115	1550	1600	0
12	CH	115	1650	1700	0
13	CH	115	1750	1800	0
14	CH	115	1850	1900	0
15	CH	115	1950	2000	0
16	CH	115	2000	2000	0



STATION NO.	SOIL TYPE	TOTAL UNIT WEIGHT PCF	2 - UNIT COHESION - PSF		FRICTION ANGLE DEGREES
			TOP OF STRATUM	BOTTOM OF STRATUM	
		WET 1	WET 1	WET 1	
1	BEACH SAND	132	0	0	49
2	CH	110	420	420	0
3	SM	122	0	0	30
4	CH	115	420	420	0
5	CH	115	1050	1050	0
6	CH	115	1150	1200	0
7	CH	115	1250	1350	0
8	CH	115	1350	1450	0
9	CH	115	1450	1600	0
10	CH	115	1600	1700	0
11	CH	115	1700	1750	0
12	CH	115	1750	1850	0
13	CH	115	1850	1900	0
14	CH	115	1900	2000	0
15	CH	115	2000	2200	0
16	CH	115	2000	2200	0

NO.	ASSUMED FAILURE SURFACE ELEV	RESISTING FORCES Ra	Rs	DRIVING FORCES Da	Ds	SUMMATION OF FORCES RESISTING	DRIVING	FACTORS OF SAFETY OF	w/1500 lbs/ft GEOSYNTHETIC AT SLOPE
1	10	25536	7230	15913	6705	32766	22618	1.45	1.69
2	0	25536	13005	4050	67051	38541	71101	0.54	1.31
3	-10	25536	13005	4050	67051	38541	71101	0.54	1.31
4	-10	65376	3430	40051	16270	68806	56321	1.22	1.26
5	-20	65376	29730	36493	105696	95106	142189	0.67	2.89
6	-20	65376	29730	36493	105696	95106	142189	0.67	2.22
7	-30	73951	42500	58938	163171	116451	221867	0.52	2.47
8	-30	73951	42500	58938	163171	116451	221867	0.52	2.70
9	-50	121455	75800	101350	260000	197255	361350	0.55	2.32
10	-50	121455	75800	101350	260000	197255	361350	0.55	2.60
11	-50	121455	197400	101350	260000	318855	361350	0.88	3.60
12	60	148810	90500	137479	386999	283397	576998	0.49	3.06
13	-60	148810	208500	137479	386999	431989	576998	0.75	3.63
14	-70	148810	208500	180509	486301	488399	720288	0.68	3.18
15	-70	178967	86000	180509	486301	443021	720288	0.62	3.18
16	-70	178967	212800	180509	486301	577967	720288	0.80	3.63
17	-70	178967	212800	180509	486301	577967	720288	0.80	4.25
18	-90	214588	212800	238000	592442	739988	1048930	0.71	3.94
19	-90	214588	265000	238000	592442	792188	1048930	0.76	4.25
20	-100	281913	214700	265000	821196	1031613	1252857	0.82	3.94
21	-110	320813	265000	304000	953967	1220813	1473967	0.83	3.04
22	-120	359713	265000	343999	1255736	1459713	1799736	0.81	3.24


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
DISTANCE IN FEET
ELEVATIONS IN FEET N.A.V.D. 88
PROTECTED SIDE
FLOOD SIDE
GENERAL NOTES
NOTES
FACTOR OF SAFETY = Rs + Rp + Rr / Da - Ds



ASSUMED FAILURE SURFACE NO		ELEV	RESISTING FORCES R_x	R_y	R_z	DRIVING FORCES D_x	$-D_z$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	FACTORS w/2020 lb/ft GEOSYNT-ETC AT 5F/ STRAM
A	1	-20.0	4763	3025	3725	5737	1708	14088	4077	3.46	
B	1	-10.0	13289	12000	7965	22542	4539	33254	18003	1.85	
C	1	-3.0	19101	14460	10941	39544	9500	44442	30144	1.47	
D	1	0	28163	16200	14121	50285	11962	50284	38627	1.30	
E	1	-15.0	28163	9000	15967	67256	21988	53150	45061	1.18	
F	1	-15.0	28163	47003	20327	67256	16835	77165	55576	1.38	
G	1	-10.0	30163	8403	17755	85815	32397	56315	53218	1.06	
H	1	-10.0	30163	16400	16392	85815	28249	63383	57166	1.10	1.52
I	2	-10.0	30163	20400	15905	85815	26292	66488	59333	1.12	1.53
J	2	-10.0	30163	24400	15286	85815	24383	69852	61232	1.14	1.54
K	2	-10.0	30163	28400	14574	85815	22531	73237	63084	1.16	1.55
L	2	-10.0	30163	32400	14036	85815	20787	76821	64828	1.18	1.56
M	2	-10.0	30163	36400	13441	85815	19091	80064	66524	1.20	1.57
N	2	-10.0	30163	61200	7873	85815	13769	99236	77846	1.10	1.44
O	2	-10.0	30163	45800	4000	85815	3447	79963	60958	1.00	1.30
P	1	-10.0	7326	4000	4000	18112	5249	15326	12863	1.19	1.39*
Q	1	-23.0	53151	33000	38813	134711	66908	124864	67803	1.84	
R	2	-20.0	53151	249750	25000	134711	21191	327854	115520	2.84	
S	1	-30.0	79464	42000	61352	184603	107011	178816	87598	2.04	
T	2	-30.0	79464	267600	48000	184603	49376	390684	165765	2.69	

* Floodside berm fabric strength is 300 #/in

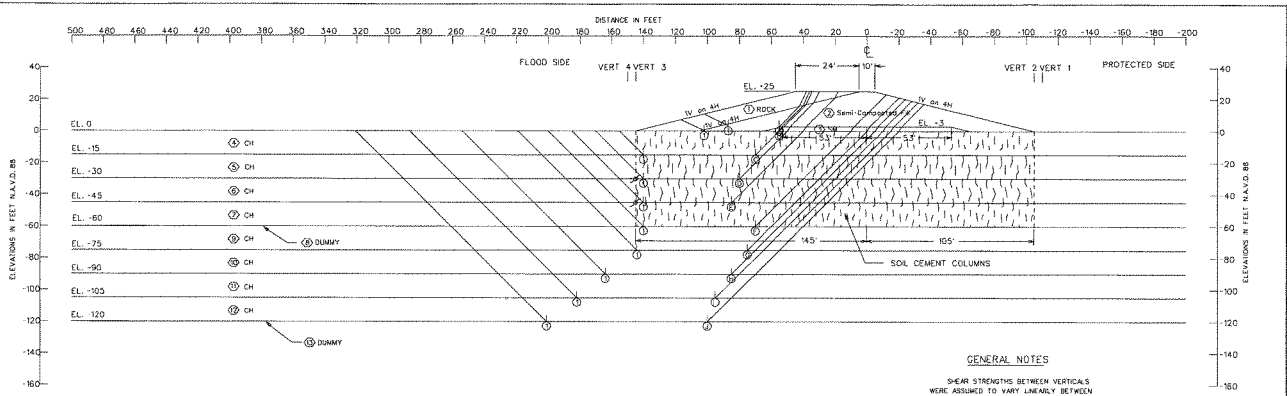
ASSUMED FAILURE SURFACE NO		ELEV	RESISTAG FORCES R_x	R_y	R_z	DRIVING FORCES D_x	$-D_z$	SUMMATION OF FORCES RESISTAG	DRIVING	FACTOR OF SAFETY
A	1	-40.0	948101	45500	85850	282925	156388	23000	154537	7.21
C	2	-40.0	986101	2908000	73000	252925	88469	4534*0	174459	2.60
G	1	-50.0	925755	63600	112120	241502	218205	300875	121497	2.44
K	2	-50.0	925755	892200	150300	241502	139798	519855	201104	2.58
N	1	-60.0	1539101	675900	140850	430348	291115	362069	139234	2.60
N	2	-60.0	1539101	3075900	129000	430348	199958	5904*0	230398	2.56

STRATUM NO	SOIL TYPE	TOTAL UNIT WEIGHT PCF	C - UNIT COHESION - PSF. (WHERE OF STRATUM BOTTOM OF STRATUM) VERT	VERT +	FRICTION ANGLE DEGREES
1	RIPRAP	142	0	0	40
2	EH	110	400	400	0
3	SM	122	0	0	30
4	CH	93	200	200	0
5	CH	115	1050	1100	0
6	CH	115	1150	1200	0
7	CH	115	1250	1300	0
8	CH	115	1350	1400	0
9	CH	115	1450	1500	5
10	CH	115	1550	1600	0
11	CH	115	1650	1700	0
12	CH	115	1750	1800	0
13	CH	115	1850	1900	0
14	CH	115	1950	2000	0
15	CH	115	2000	2000	0

GENERAL NOTES:
CLASSIFICATION/STRATIFICATION SHEAR STRENGTH AND UNIT WEIGHT BY EVC SOIL WERE BASED ON THE RESULTS OF UNDISTURBED BORINGS SEE BORING DATA PLATES

NOTES
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION P.S.F.
W -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT, REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT, REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_x + R_y \cdot R_z}{D_x - D_z}$



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
(A)	(3)	3.0	14631	12850	16417	28064	3977	43848	225877	1.91
(B)	(3)	.0	16919	19800	15938	37302	5111	51855	32191	1.60
(C)	(3)	-15.0	79623	140000	6000	86700	11098	225623	75602	2.98
(D)	(3)	-30.0	139940	120000	12000	156366	43723	271940	113845	2.35
(E)	(3)	-45.0	200903	110000	20258	258164	98846	331158	160318	2.07
(F)	(3)	-60.0	264434	35000	35000	336079	176469	329434	219801	1.50
(G)	(3)	-75.0	277045	44850	50235	340398	276797	372130	264199	1.41
(H)	(3)	-90.0	297948	65200	71985	402978	398802	433133	304176	1.42
(I)	(3)	-105.0	323123	82650	98235	584617	543924	504008	340693	1.48
(J)	(3)	-120.0	351724	111100	128985	668284	715143	591809	376739	1.57

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
(1)	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
(2)	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
(3)	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
(4)	CH	95	105	105	95	200	2000	2000	200	200	2000	2000	200	0
(5)	CH	100	105	105	100	200	2000	2000	250	200	2000	2000	200	0
(6)	CH	100	105	105	100	275	2000	2000	275	350	2000	2000	350	0
(7)	CH	100	105	105	100	425	2000	2000	425	500	2000	2000	500	0
(8)	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
(9)	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
(10)	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
(11)	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
(12)	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
(13)	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

NOTES

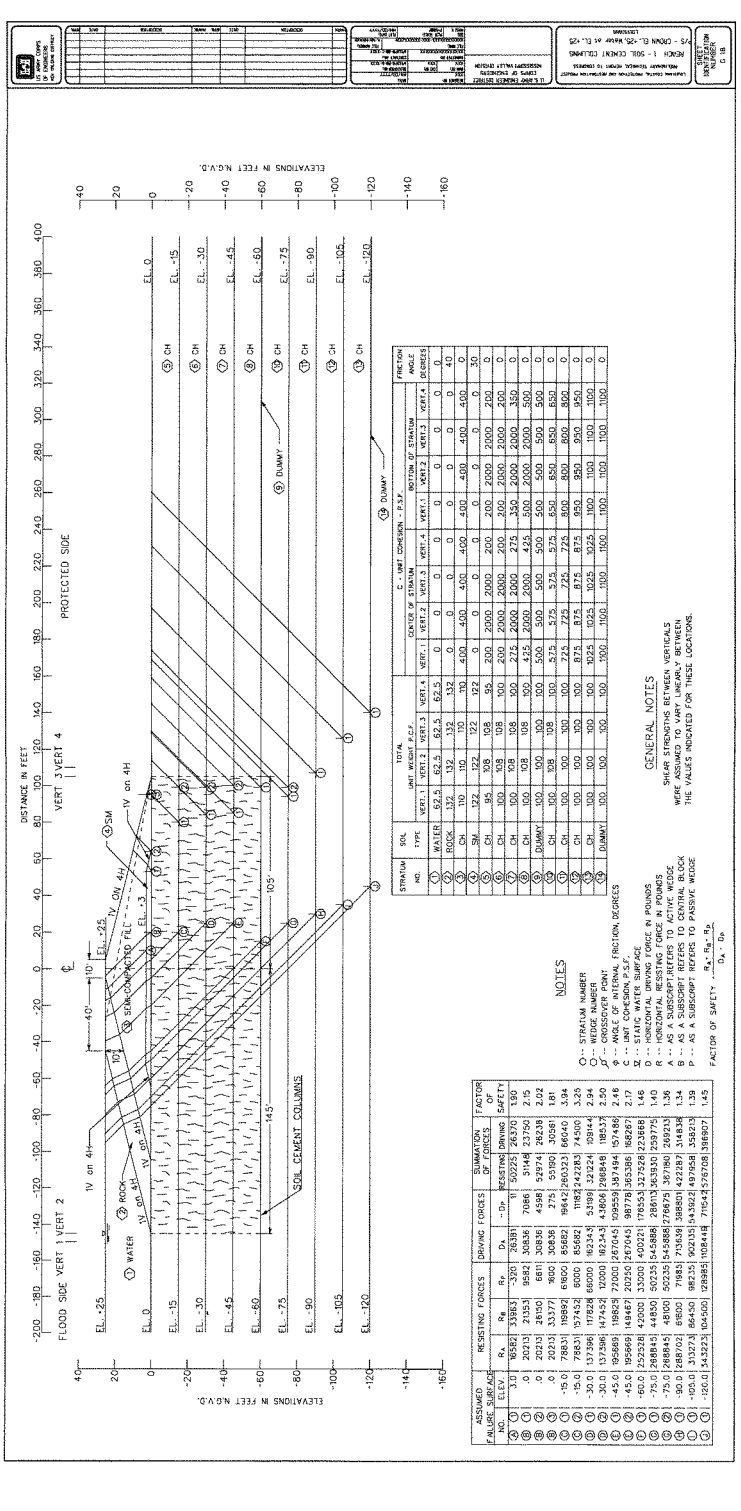
- (○) -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- // -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

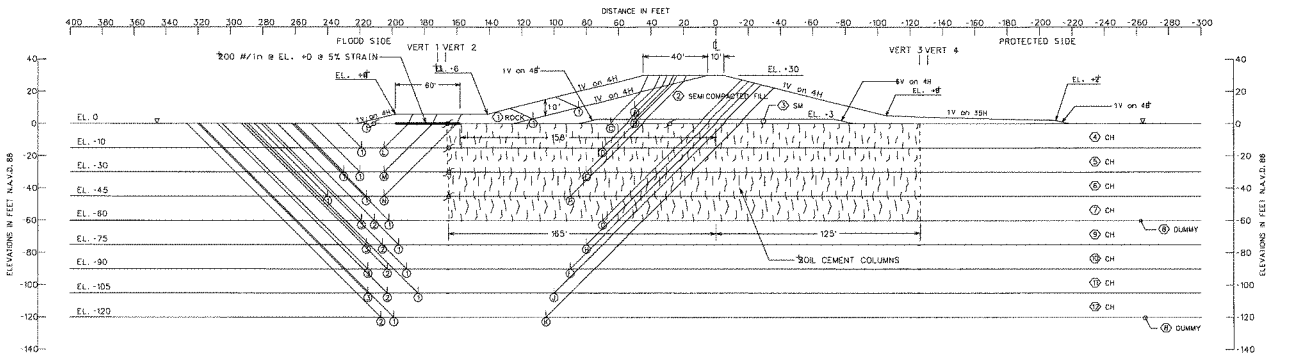
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

REACH 1 - SOIL CEMENT COLUMNS
F/S - CROW III-25
LONGWALL

SHEET IDENTIFICATION NUMBER
C 17


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
DISTANCE IN FEET
FLOOD SIDE
PROTECTED SIDE
VERT 1
VERT 2
VERT 3
VERT 4
ELEVATIONS IN FEET N.G.V.D.
1V on 4H
SOIL-CONCRETE FILL
SOIL CEMENT COLUMNS
WATER
ROCK
SM
CH
DUMMY
EL. 0
EL. -15
EL. -30
EL. -45
EL. -60
EL. -75
EL. -90
EL. -105
EL. -120
NOTES
O -- STRATUM NUMBER
W -- WEDGE NUMBER
-- CROSSOVER POINT
φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
γ -- UNIT WEIGHT, P.C.F.
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE
FACTOR OF SAFETY = $R_A + R_B + R_P$ / D_A D_B
GENERAL NOTES
SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSUMED TO VARY LINEARLY BETWEEN
THE VALUES INDICATED FOR THESE LOCATIONS.
ASSUMED FAILURE SURFACE NO.
ELEV.
RESISTING FORCES
DRIVING FORCES
SUMMATION OF FORCES
FACTOR OF SAFETY
STRATUM NO.
SOIL TYPE
UNIT WEIGHT P.C.F.
CENTER OF STRATUM
C - UNIT COHESION - P.S.F.
FRICTION ANGLE DEGREES
FOR 1 FOOT WIDTH OF LEVEE
REACH 1 - SOIL CEMENT COLUMNS
MISSISSIPPI RIVER GULF OUTLET
-25' - CROWN EL. +25, WATER AT EL. +25
DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
SHEET NO.
SHEET DESCRIPTION NUMBER
G 18



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	SAFETY
Ⓐ	① 10.0	12918	14000	13461	24859	4284	42380	20565	2.06
Ⓑ	① 3.0	18658	29200	15457	44057	4294	59315	39763	1.49
Ⓒ	① .0	24483	105500	348	48817	97	130331	48720	2.68
Ⓓ	① -15.0	84765	201679	9000	111339	10836	292444	100503	2.91
Ⓔ	① -30.0	145297	181880	12000	182561	43377	339177	149184	2.27
Ⓕ	① -45.0	208058	168856	20250	294867	99028	396984	193839	2.02
Ⓖ	① -60.0	265755	86000	33000	443354	182655	364755	260699	1.40
Ⓗ	② -60.0	265755	70500	33000	443354	178053	369255	265301	1.39
Ⓘ	③ -60.0	265755	74500	33000	443354	176206	373255	267148	1.40
Ⓙ	① -75.0	282890	71125	50235	593900	287264	404250	306636	1.32
Ⓚ	② -75.0	282890	77625	50235	593900	280402	410750	313498	1.31
Ⓛ	③ -75.0	282890	84125	50235	593900	276773	417250	317127	1.32
Ⓜ	① -90.0	303840	80800	71985	763427	413845	456625	349582	1.31
Ⓝ	② -90.0	303840	90400	71985	763427	404756	466225	358671	1.30
Ⓞ	③ -90.0	303840	100000	71985	763427	399609	475825	363818	1.31
Ⓟ	① -105.0	329243	79800	98235	952084	564509	507278	387575	1.31
Ⓠ	② -105.0	329243	97850	98235	952084	549877	525328	402207	1.31
Ⓡ	③ -105.0	329243	109250	98235	952084	544730	536728	407354	1.32
Ⓢ	① -120.0	357843	103400	128985	1163311	720268	590228	443043	1.33
Ⓣ	② -120.0	357843	112000	128985	1163311	715253	599228	448058	1.34

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF	F.S. w/200 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	SAFETY	
①	③ -15.0	7858	2800	6000	24129	10836	16658	13293	1.25	1.43
Ⓤ	① -30.0	13859	5000	12000	58632	43311	30859	25321	1.22	1.31
Ⓥ	① -45.0	22109	12250	20250	135635	98434	54609	37201	1.47	

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C = UNIT COHESION - P.S.F. CENTER OF STRATUM				C = UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	DEGREES
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑥	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
⑦	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
⑨	CH	100	100	100	100	500	500	500	500	500	500	500	500	0
⑯	CH	100	100	100	100	575	600	600	575	650	600	600	650	0
⑰	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
⑱	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
⑫	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
⑬	CH	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 200 LB/IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLING FAILURE WEDGE Ⓜ ①

R - DRIVING(F.S.) - RESISTING

T = (25321) (1.3) - 30859 = 2068 LB/FT

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

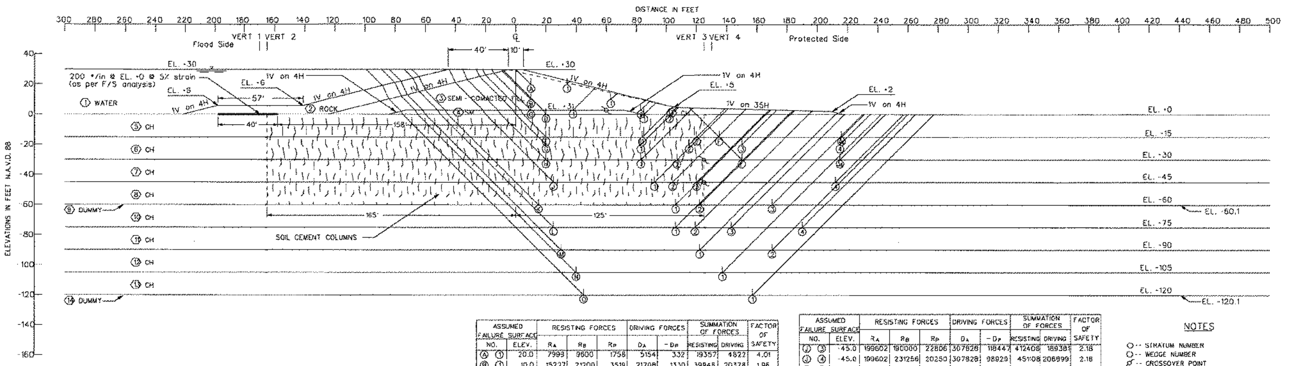
○ -- STRATUM NUMBER
⬡ -- WEDGE NUMBER
⌀ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
γ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

U.S. ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

REACH 1 - SOIL CEMENT COLUMNS
F/S - CROSS SECTION

SHEET IDENTIFICATION NUMBER
D-19



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
7	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
8	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
9	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
10	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
11	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
12	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
13	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
14	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A	1	20.0	7993	9600	1758	5154	332	19357	4822	4.01
B	1	10.0	15227	21200	3519	21708	1330	39946	20378	1.96
C	1	3.0	19837	11200	11999	40065	15465	43036	24600	1.75
C	2	3.0	19837	27972	4799	40065	2474	52608	37591	1.40
D	1	.0	23669	36325	6399	45991	4398	66393	41593	1.60
D	2	.0	23669	40155	3953	45991	1492	67777	44499	1.52
E	1	-15.0	81988	128000	64160	109414	27826	274148	81588	3.36
E	2	-15.0	81988	200000	9224	109414	18780	291212	90634	3.21
E	3	-15.0	81988	229079	6000	109414	10742	317067	98672	3.21
F	4	-15.0	9751	16200	6000	19035	10742	31951	8293	3.85
G	1	-20.0	101277	126000	83930	136011	40617	311207	95394	3.26
G	2	-20.0	101277	190000	29224	136011	30773	320501	105238	3.05
G	3	-20.0	101277	215879	13444	136011	27134	327600	108877	3.01
G	4	-20.0	101277	228879	8000	136011	18186	338156	118825	2.89
H	1	-30.0	140249	126000	123696	197673	72859	389945	124814	3.12
H	2	-30.0	140249	174000	69179	197673	82893	383428	134780	2.84
H	3	-30.0	140249	228880	12000	197673	43435	381129	154238	2.47
I	3	-30.0	15751	13000	12000	58143	43435	40751	14708	2.77
J	1	-45.0	199602	134000	129179	307828	134207	462781	173621	2.67
J	2	-45.0	199602	158000	74912	307828	126498	432514	181330	2.39

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
J	3	-45.0	199602	190000	22806	307828	118447	412408	189381	2.18
J	4	-45.0	199602	231256	20250	307828	98929	451108	208899	2.18
K	1	-60.0	256187	45500	82783	447939	211174	384470	236765	1.62
K	2	-60.0	256187	53500	35177	447939	200608	344864	247331	1.39
K	3	-60.0	256187	77500	33000	447939	168247	366687	259692	1.41
L	1	-75.0	272499	52650	52435	601601	317077	377584	284524	1.33
L	2	-75.0	272499	61100	52145	601601	303599	385744	298002	1.30
L	3	-75.0	272499	76700	50235	601601	298852	399434	302749	1.32
L	4	-75.0	272499	107250	50235	601601	282084	429984	319517	1.35
M	1	-90.0	292403	73600	72945	777381	430805	438946	346576	1.27
M	2	-90.0	292403	112000	71985	777381	410990	476388	366391	1.30
N	1	-105.0	317729	92150	98235	971201	569236	508114	401965	1.26
O	1	-120.0	346593	123200	126985	1188371	728493	598778	459878	1.30

NOTES

○ -- STRATUM NUMBER
⬡ -- WEDGE NUMBER
ø -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY $= \frac{R_A + R_B + R_P}{D_A - D_P}$

GENERAL NOTES

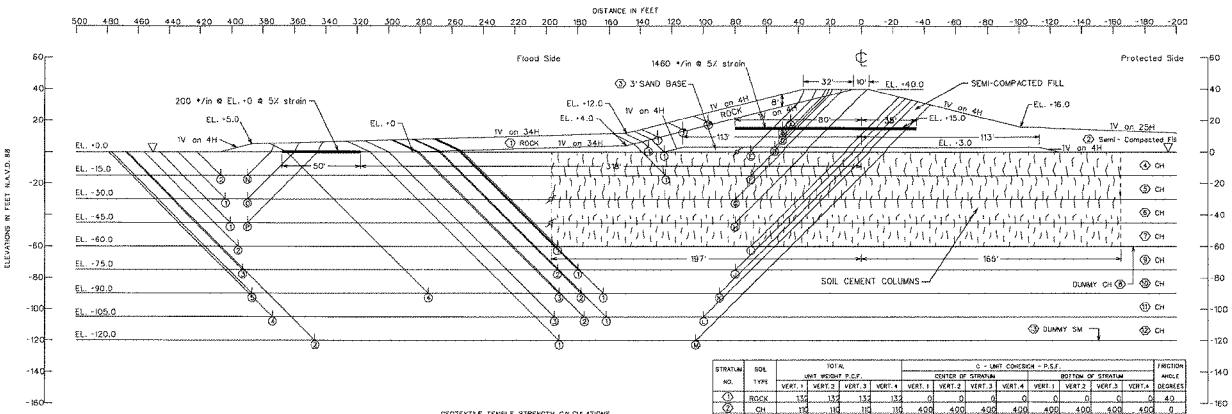
SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MISSISSIPPI VALLEY DIVISION

REACH 1 - SOIL CEMENT COLUMNS

SHEET IDENTIFICATION NUMBER G 28



ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1480 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓐ	①	20.0	12960	20800	7974	23572	810	41734	23262	1.79	
Ⓑ	①	15.0	16960	25200	8614	35681	12351	50774	36446	1.47	
Ⓒ	①	10.0	21290	31600	9255	51447	19481	62145	49499	1.26	1.81
Ⓓ	①	3.0	27101	32000	12797	76184	27447	71898	68747	1.05	1.30
Ⓔ	①	.0	33388	82887	16298	78091	15595	142571	62496	2.28	
Ⓕ	①	-15.0	95236	108000	75488	198538	52303	278724	107235	2.60	
Ⓖ	②	-15.0	95236	295977	6000	159338	10887	397213	148851	2.67	
Ⓗ	①	-30.0	156341	275378	12000	254328	43498	443718	210869	2.10	
Ⓘ	①	-45.0	218428	305378	20250	382164	99038	544054	283136	1.92	
Ⓙ	①	-60.0	274355	815061	48029	536074	256435	383884	279639	1.37	
Ⓚ	②	-60.0	274355	163090	33000	536074	178052	470355	358022	1.31	
Ⓛ	①	-75.0	290760	85000	65234	699453	384999	420986	314454	1.34	
Ⓜ	②	-75.0	290760	73450	64938	699453	373099	429148	326354	1.31	
Ⓝ	③	-75.0	290760	203450	50250	699453	279411	544460	420042	1.30	
Ⓞ	①	-90.0	311710	59200	87007	882062	537589	457917	344493	1.33	
Ⓟ	②	-90.0	311710	70400	86667	882062	524713	468797	357349	1.31	
Ⓠ	③	-90.0	311710	81600	86366	882062	511878	479676	370184	1.30	
Ⓡ	④	-90.0	311710	148000	79120	882062	483545	538830	398517	1.35	
Ⓢ	⑤	-90.0	311710	237600	72000	882062	405397	621310	476665	1.30	
Ⓣ	①	-105.0	337113	58900	112559	1083839	699927	508972	383912	1.33	

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. +15.0 CONTROLLING FAILURE WEDGE (B) - (T)

T = DRIVING(F.S.) - RESISTING
T = 68747(1.3) - 71898 = 17475 LB/FT

EL. -0.0 CONTROLLING FAILURE WEDGE (C) - (T)

T = DRIVING(F.S.) - RESISTING
T = (227091(1.3) - 27957 = 1565 LB/FT

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/200 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓛ	②	-105.0	337113	72200	112639	1083839	886410	529352	397429	1.31	
Ⓛ	③	-105.0	337113	90250	111013	1083839	871013	538378	428829	1.30	
Ⓛ	④	-105.0	337113	260300	98250	1083838	559424	695663	524415	1.33	
Ⓜ	①	-120.0	365713	95700	140598	1308226	854055	502011	454170	1.33	
Ⓜ	②	-120.0	365713	266200	129000	1308226	747635	760913	560590	1.36	
Ⓝ	②	-15.0	7491	2400	6000	22630	11100	15899	11530	1.38	
Ⓝ	③	-30.0	13757	2230	12000	66614	43905	27907	22709	1.23	1.33
Ⓟ	①	-45.0	22285	2800	20250	133968	99771	45335	34227	1.32	

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑥	CH	100	108	108	100	273	2000	2000	275	350	2000	2000	350	0
⑦	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
⑧	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
⑨	CH	100	100	100	100	573	575	575	575	650	650	650	650	0
⑩	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
⑪	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
⑫	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
⑬	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

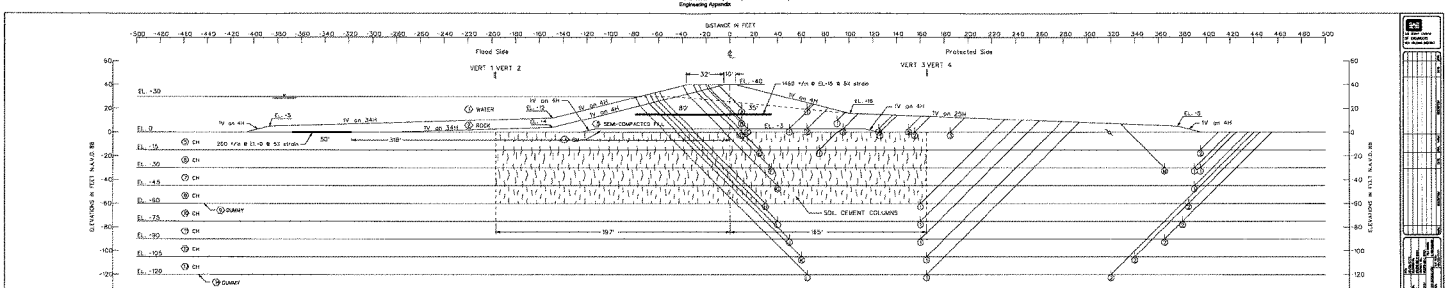
NOTES

○-- STRATUM NUMBER
○-- WEDGE NUMBER
𝒳-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
𝔄 -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

GENERAL NOTES

CLASSIFICATION STRATIFICATION
SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS

STATION NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. DEPTH OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
(1)	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
(2)	ROCK	145	145	145	145	0	0	0	0	0	0	0	0	45
(4)	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
(5)	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
(6)	CH	105	105	105	105	200	2000	2000	2000	200	2000	2000	2000	0
(7)	CH	105	105	105	105	200	2000	2000	2000	500	2000	2000	2000	0
(8)	CH	105	105	105	105	200	2000	2000	2000	1000	2000	2000	2000	0
(9)	CH	105	105	105	105	277	2000	2000	2000	277	2000	2000	2000	0
(10)	CH	100	100	100	100	423	2000	2000	2000	423	500	2000	2000	0
(11)	QUARRY	105	105	105	105	500	500	500	500	500	500	500	500	0
(12)	CH	105	105	105	105	277	277	277	277	500	500	500	500	0
(13)	CH	100	100	100	100	723	723	723	723	800	800	800	800	0
(14)	CH	100	100	100	100	873	873	873	873	950	950	950	950	0
(15)	CH	105	105	105	105	1023	1023	1023	1023	1100	1100	1100	1100	0
(16)	QUARRY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_x	R_y	R_T	DRIVING FORCES D_x	$-D_x$	SUMMATION OF MOMENTS RESISTING	DRIVING	FACTOR OF SAFETY
(1)	0	22051	220500	3705	21730	1099	45430	20609	1.39
(2)	0	22051	220500	5850	49847	3388	58955	48279	1.28
(3)	0	27477	196200	16479	74550	29016	57995	45378	1.28
(3)	3.0	27477	210000	14080	74550	21064	83567	53254	1.76
(3)	2.5	27477	202000	10983	74550	9588	69862	54864	1.27
(3)	3.0	27477	45323	9364	74550	7660	80054	66481	1.20
(3)	3.0	27477	53333	8498	74550	5401	98098	68058	1.31
(3)	.0	32351	46403	11038	88800	11895	90289	76902	1.17
(3)	-.0	32231	57809	10648	88800	12129	110786	78671	1.28
(3)	-.0	32231	88701	9723	88800	9443	110755	80353	1.36
(3)	-15.0	32351	1000000	75003	182028	61885	2678214	1804218	2.87
(3)	-19.2	32351	1258419	6000	182028	10781	4276755	1982276	2.83
(3)	-30.0	35224	3268880	12000	259460	45235	6797211	2158836	2.18
(3)	-45.0	39734	5295656	20234	343263	84826	10644243	2834344	1.98
(3)	-60.0	3875578	93000	41643	3538233	3648404	3943202	277436	1.38
(3)	-90.0	3875578	1778000	23096	3538235	1777202	4780318	3038515	1.33
(3)	-75.0	2838855	780000	58416	2698943	2832770	4200981	3119688	1.32
(3)	-75.0	2838855	2210000	50233	2698943	2738698	4355869	4202450	1.32
(3)	-90.0	3043331	880000	19705	2820692	2592244	4720388	3613168	1.31
(3)	-90.0	3043331	2520000	21885	2820692	4198.0	5285384	4129881	1.34

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1460 LB\IN.

GEOSYNTHETIC TENSILE STRENGTH CALCULATIONS

CONTROLLING FAILURE SURFACE NO. = (3)

T = OVERTURNING - RESISTING

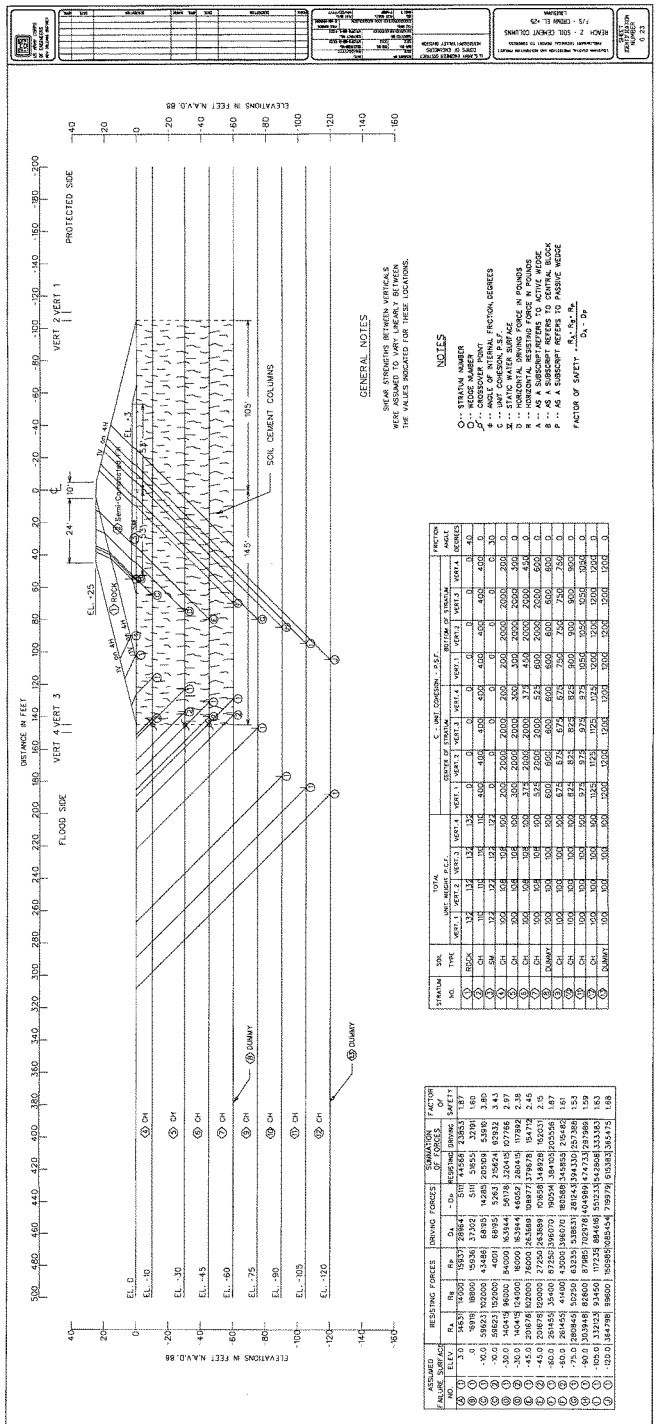
T = (4864413.9 - 4796.0 = 1460LB/FT

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_x	R_y	R_T	DRIVING FORCES D_x	$-D_x$	SUMMATION OF MOMENTS RESISTING	DRIVING	FACTOR OF SAFETY
(1)	-100.0	3290327	187701	85334	2684995	879348	5343921	4102243	1.30
(2)	-105.0	3429232	2691100	98235	3084592	572081	6924537	3923.7	1.35
(3)	-120.0	3577222	160000	154208	3308931	86348	8024801	4447825	1.25
(4)	-106.0	3577223	1803000	139898	3308931	754.72	7961727	4344298	1.38
(5)	-30.0	1953.4	5000	140001	85757	43896	3453.9	2180.4	1.57

NOTES

O -- STRAIN NUMBER
□ -- WEDGE NUMBER
// -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
W -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_x + R_y + R_z}{D_x + D_y}$



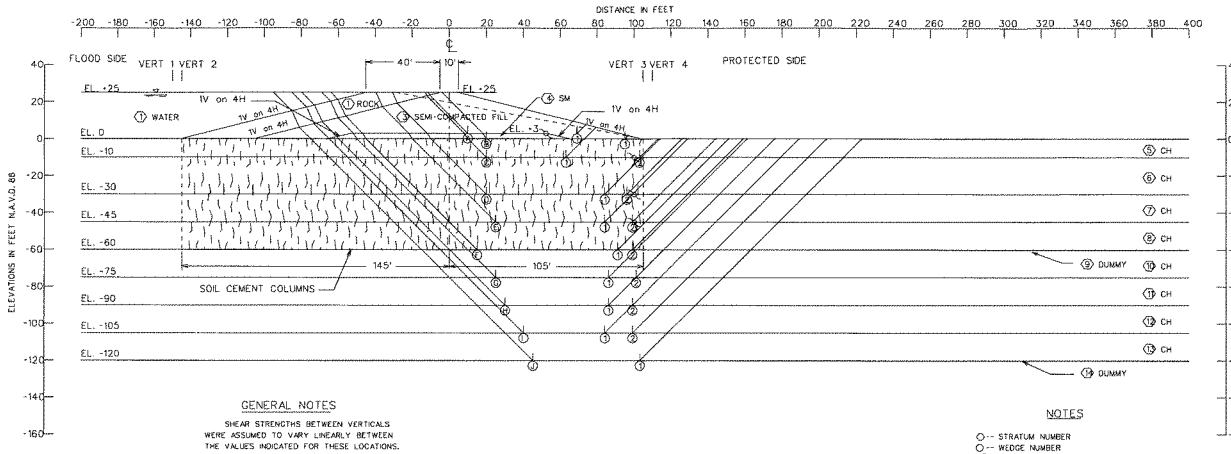
ASSURED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES Rx	Ry	DRIVING FORCES Dx	Dy	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
18 17 1	3.0	1483	140001	15937	28964	5111	43588	13853	1.87
18 17 2	0.0	59384	140001	31222	28964	5111	43588	13853	1.85
18 17 3	-10.0	59823	152000	43486	28935	14285	26503	53910	3.80
18 17 4	-30.0	59823	152000	4001	68935	52687	25624	182932	3.43
18 17 5	-30.0	140419	96000	86344	56178	32049	107766	2.97
18 17 6	-30.0	140419	124000	86000	86344	46057	26041	107892	2.38
18 17 7	-45.0	140419	152000	86000	86344	46057	26041	107892	2.45
18 17 8	-45.0	209878	152000	71858	34492	8	26503	2.45
18 17 9	-60.0	261455	354001	87250	396070	190254	384105	2055561	1.87
18 17 10	-60.0	261455	414001	43000	396070	1805861	3458582	2164821	1.61
18 17 11	-75.0	261455	502901	61235	538531	2816743	3943302	2573881	1.53
18 17 12	-90.0	261455	502901	78000	680444	3827623	5243302	3239061	1.39
18 17 13	-105.0	132125	934501	117235	884496	5513335	8426821	5313951	1.51
18 17 14	-120.0	384798	89600	150985	1085404	7199791	6153821	3654751	1.68

STRATUM NO.	SOIL TYPE	UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	490	490	490	490	490	490	490	490	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	104	104	104	104	1000	1000	1000	1000	2000	2000	2000	2000	0
5	CH	104	104	104	104	1000	1000	1000	1000	2000	2000	2000	2000	0
6	CH	104	104	104	104	375	2000	2000	2000	375	450	2000	2000	0
7	CH	110	104	104	104	525	2000	2000	2000	525	600	2000	2000	0
8	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
9	CH	110	100	100	100	675	675	675	675	750	750	750	750	0
10	CH	110	100	100	100	675	675	675	675	750	750	750	750	0
11	CH	110	100	100	100	975	975	975	975	1050	1050	1050	1050	0
12	CH	110	100	100	100	975	975	975	975	1050	1050	1050	1050	0
13	CH	110	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
14	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

NOTES

Q - - STRATUM NUMBER
14 - - NEXT NUMBER
O - - CROSSOVER POINT
Φ - - ANGLE OF INTERNAL FRICTION, DEGREES
C - - UNIT COHESION, P.S.F.
γ_w - - STATIC WATER SURFACE
D - - HORIZONTAL DRIVING FORCE IN POUNDS
R - - HORIZONTAL RESISTING FORCE IN POUNDS
A - - AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B - - AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P - - AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_B}$



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
7	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
8	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
9	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
10	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
11	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
12	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
13	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
14	DUMMY	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
(A)	(1)	3.0	16582	23544	3839	29381	1583	43965	24798	1.77
(B)	(1)	.0	20213	33383	1599	30836	274	55195	30562	1.81
(C)	(1)	-10.0	59291	86000	45119	64612	18394	190410	46218	4.12
(C)	(2)	-10.0	59291	165966	4000	64612	5055	229257	59557	3.85
(D)	(1)	-30.0	136685	128000	84000	165363	54258	348685	111105	3.14
(D)	(2)	-30.0	136685	152000	16004	165363	46112	304689	119251	2.56
(E)	(1)	-45.0	195668	118000	76000	267046	111806	389668	155240	2.51
(E)	(2)	-45.0	195668	148000	27250	267046	101741	370918	165305	2.24
(F)	(1)	-60.0	252528	45600	87250	400221	188987	385378	211234	1.82
(F)	(2)	-60.0	252528	50400	43000	400221	180489	345928	219732	1.57
(G)	(1)	-75.0	271845	45750	63235	545889	293401	380830	252488	1.51
(G)	(2)	-75.0	271845	57000	63235	545889	281461	392080	264428	1.48
(H)	(1)	-90.0	294703	50400	87985	713640	417149	433088	296491	1.46
(H)	(2)	-90.0	294703	62350	87985	713640	405483	444788	308157	1.44
(I)	(1)	-105.0	323074	46200	117235	899223	564494	486509	334729	1.45
(I)	(2)	-105.0	323074	61950	117235	899223	551728	502259	347495	1.45
(J)	(1)	-120.0	355223	69600	150985	1108451	720034	575808	388417	1.48

NOTES

- ○-- STRATUM NUMBER
- ⬡-- WEDGE NUMBER
- ⊿-- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

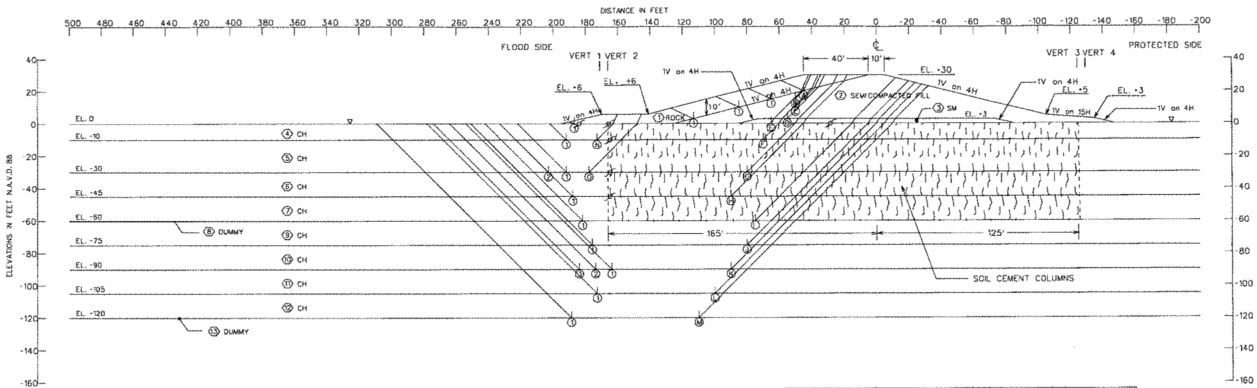
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 2 - SOIL CEMENT COLUMNS
PVS - CROSS CLOSURE ELEVS

SHEET IDENTIFICATION NUMBER C 24



ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
(A)	(1)	20.0	5163	0	15460	5597	4294	20623	2303	8.96
(B)	(1)	15.0	9163	6000	15461	14227	4294	30624	9933	3.08
(C)	(1)	10.0	12919	14000	15461	24859	4294	42380	20565	2.06
(D)	(1)	3.0	18501	23200	15457	43373	4294	57158	39079	1.46
(E)	(1)	.0	24483	99814	578	48817	161	124875	48656	2.57
(F)	(1)	-10.0	64520	196280	4000	86864	5142	264800	81722	3.24
(G)	(1)	-30.0	145297	178931	16000	192562	45141	340228	147421	2.31
(H)	(1)	-45.0	206058	161107	27250	294868	102023	394415	192845	2.05
(I)	(1)	-60.0	266610	64200	43000	441936	182671	373810	259265	1.44
(J)	(1)	-75.0	285145	72000	63235	593900	286959	420380	306941	1.37
(K)	(1)	-90.0	309095	66600	87985	763427	419637	463680	343790	1.35
(K)	(2)	-90.0	309095	75600	87985	763427	411972	472680	351455	1.34
(K)	(3)	-90.0	309095	84600	87985	763427	406905	481680	356522	1.35
(L)	(1)	-105.0	337498	76650	117235	952083	558900	531383	393183	1.35
(M)	(1)	-120.0	370173	94800	150985	1159799	720551	615958	439248	1.40
(N)	(1)	-10.0	5859	3800	4000	15152	5142	13659	10010	1.36
(O)	(2)	-30.0	53860	7500	16000	70518	44999	77360	25519	3.03

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

- ○ -- STRATUM NUMBER
- ⬡ -- WEDGE NUMBER
- ⌀ -- CROSSOVER POINT
- Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
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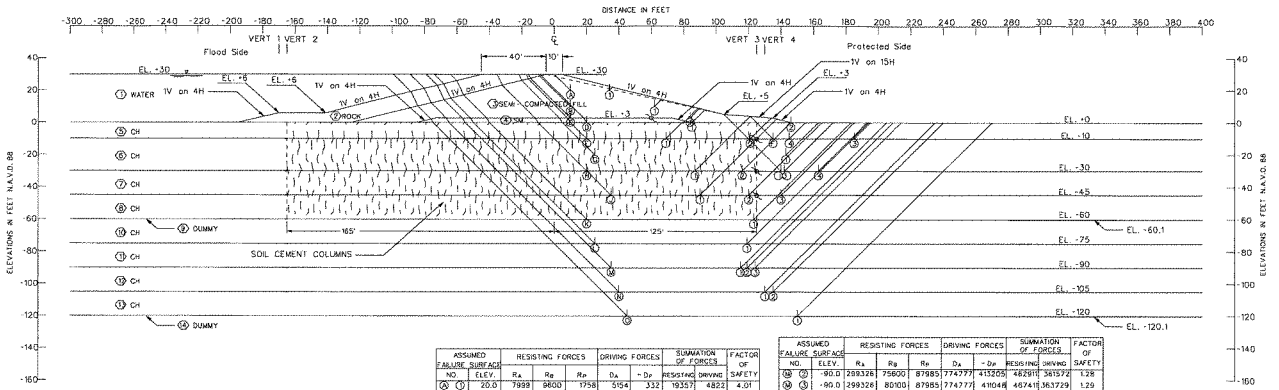
FACTOR OF SAFETY $= \frac{R_A + R_B + R_P}{D_A - D_P}$

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
(1)	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
(2)	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
(3)	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
(4)	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
(5)	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
(6)	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
(7)	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
(8)	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
(9)	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
(10)	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
(11)	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
(12)	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
(13)	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 2 - SOIL CEMENT COLUMNS
F/S - CROWN EL +30
LOUISIANA

SHEET IDENTIFICATION NUMBER
G 25



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				BOTTOM OF STRATUM				FRICTION ANGLE
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	DEGREES
①	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
②	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
③	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
④	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⑤	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑥	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
⑦	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
⑧	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
⑨	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
⑩	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
⑪	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
⑫	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
⑬	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
⑭	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ	①	20.0	7998	9600	1758	5154	332	19357	4822	4.01
Ⓑ	①	10.0	15227	20800	3679	21708	1454	39706	20254	1.96
Ⓒ	①	3.0	19837	28359	4639	40065	2311	52835	37754	1.40
Ⓓ	①	.0	23869	36325	6399	45991	4398	66393	41593	1.60
Ⓔ	②	.0	23869	44388	160	45991	3	68217	45988	1.48
Ⓕ	①	-10.0	62526	98000	47561	85484	25373	208087	60111	3.46
Ⓖ	②	-10.0	62526	202060	6449	85484	9525	270975	75959	3.57
Ⓗ	③	-10.0	62526	222880	4000	85484	5000	289406	80484	3.60
Ⓘ	③	-10.0	7161	2000	4000	9473	5055	13161	4418	2.98
Ⓙ	①	-20.0	10986	206230	10000	13135	20219	318216	112916	2.82
Ⓚ	①	-30.0	140249	134000	123166	197673	69951	397415	127722	3.11
Ⓛ	②	-30.0	140249	182000	16162	197673	54585	348411	143088	2.43
Ⓜ	③	-30.0	140249	219931	16000	197673	45342	372180	152331	2.44
Ⓝ	③	-30.0	56000	6900	16000	58551	44999	78900	13582	5.82
Ⓞ	①	-45.0	200249	160000	145978	289178	153917	456165	185259	2.76
Ⓟ	①	-45.0	200249	170000	27250	289178	139017	397499	190159	2.09
Ⓠ	③	-45.0	200249	187507	27250	289178	101920	415006	197258	2.10
Ⓡ	①	-60.0	256988	61800	43003	446582	186471	361788	260111	1.39
Ⓢ	①	-75.0	275499	70500	63235	501602	288459	409234	312143	1.31
Ⓣ	①	-90.0	299326	72000	87985	774777	422259	459315	352518	1.30

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓤ	②	-90.0	299326	75600	87985	774777	413505	462816	361372	1.28
Ⓥ	①	-90.0	299326	80100	87985	774777	411048	467416	363729	1.29
Ⓦ	①	-105.0	328729	94500	117235	971201	554955	538464	416246	1.29
Ⓧ	②	-105.0	328729	98750	117235	971201	553214	543714	417987	1.30
Ⓨ	①	-120.0	358594	126000	150985	1188372	719980	635579	468392	1.36

NOTES

○-- STRATUM NUMBER
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𝒪-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
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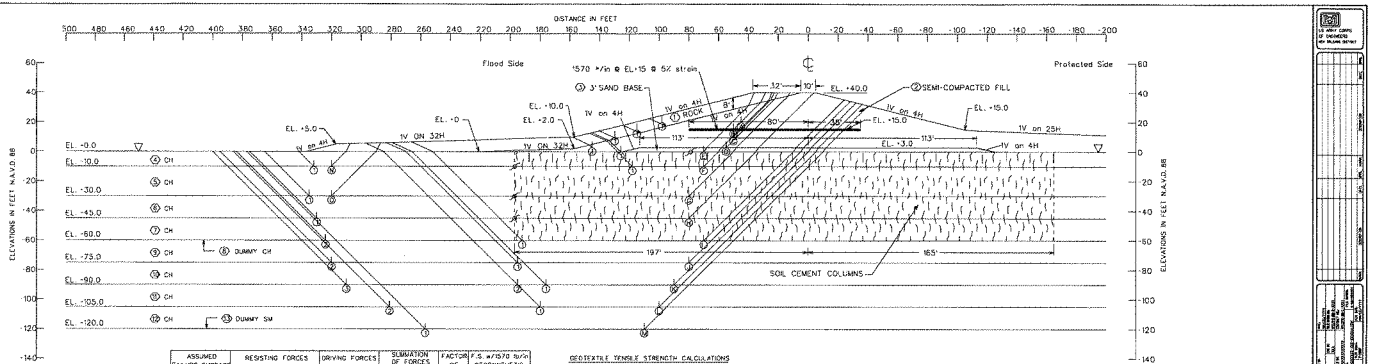
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.



ASSUMED FAILURE SURFACE		RESISTING FORCES		DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1570 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_W$	RESISTING	DRIVING	
Ⓐ	Ⓒ 20.0	129651	212201	7814	23872	468	419774	234104	1.79	
Ⓑ	Ⓒ 15.0	16980	28000	8294	35681	911	51254	34770	1.47	
Ⓒ	Ⓒ 10.0	21290	32000	9085	31449	1761	62385	49686	1.26	1.63
Ⓓ	Ⓒ 3.0	27101	39000	11986	76194	4620	74287	75746	1.04	1.30
Ⓔ	Ⓒ .0	33388	94123	16135	78091	15166	143648	92925	2.28	
Ⓕ	Ⓒ -10.0	74418	98000	55815	129330	41905	228233	88024	2.57	
Ⓖ	Ⓒ -30.0	156341	275520	18000	254328	45065	447886	209264	2.14	
Ⓗ	Ⓒ -45.0	218428	293058	27250	362764	102555	538635	280109	1.83	
Ⓘ	Ⓒ -60.0	274355	73200	54002	536074	245872	401557	290202	1.38	
Ⓙ	Ⓒ -60.0	274355	152400	43090	536074	182782	469705	553282	1.33	
Ⓚ	Ⓒ -75.0	293760	86250	72981	699434	359233	452571	340221	1.33	
Ⓛ	Ⓒ -75.0	293760	182000	83250	699434	288026	557010	413449	1.35	
Ⓜ	Ⓒ -90.0	317710	77400	97674	882061	509274	492784	372847	1.32	
Ⓝ	Ⓒ -90.0	317710	94500	98009	882061	494980	508219	387161	1.31	
Ⓞ	Ⓒ -90.0	317710	198000	88000	882061	416304	603710	465757	1.30	
Ⓟ	Ⓒ -105.0	346113	84000	125259	1083826	659972	555772	417987	1.33	
Ⓠ	Ⓒ -105.0	346113	190950	117250	1083826	584257	653413	496582	1.31	
Ⓡ	Ⓒ -120.0	378788	176400	151000	1304671	773592	706188	531079	1.33	
Ⓢ	Ⓒ -10.0	5426	2400	4000	13372	5412	11826	7980	1.49	
Ⓣ	Ⓒ -30.0	17789	3060	18000	58432	43867	38789	22825	1.63	

NOTES

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-- CROSSOVER POINT
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FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_W}$

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. -15.0 CONTROLLING FAILURE WEDGE (Ⓒ - Ⓒ)

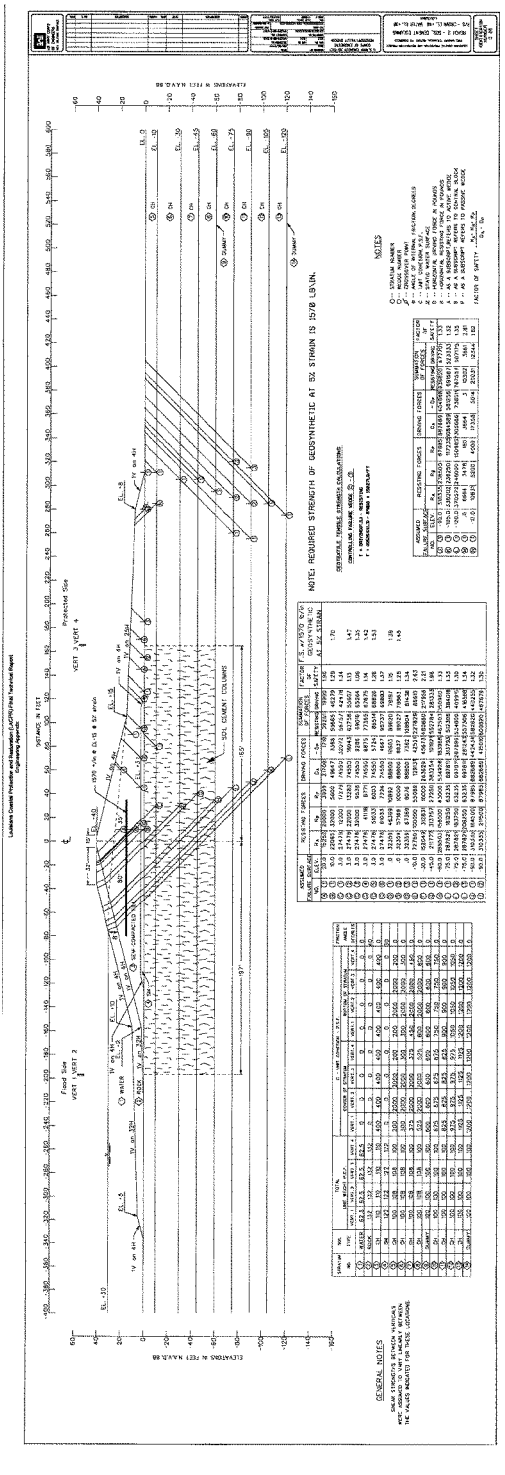
T = DAVIM(01,5) - RESISTING
T = (75754.1.3) - 74297 = 18750 LB/FT

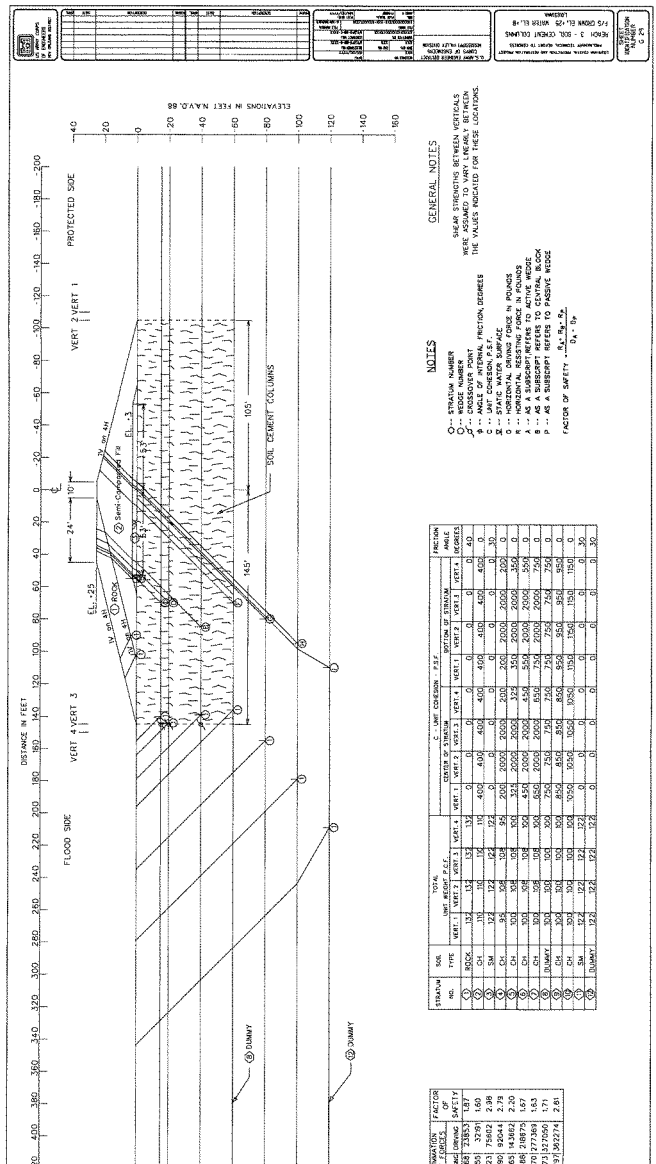
STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						COVER OF STRATUM				BOTTOM OF STRATUM				
		VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	100	100	100	100	2000	2000	2000	2000	2000	2000	2000	2000	0
⑤	CH	100	100	100	100	300	2000	2000	300	300	2000	2000	300	0
⑥	CH	100	100	100	100	375	2000	2000	375	450	2000	2000	450	0
⑦	CH	100	100	100	100	525	2000	2000	525	600	2000	2000	600	0
⑧	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
⑨	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
⑩	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
⑪	CH	100	100	100	100	875	975	875	975	1050	1050	1050	1050	0
⑫	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
⑬	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

GENERAL NOTES

CLASSIFICATION STRATIFICATION, SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

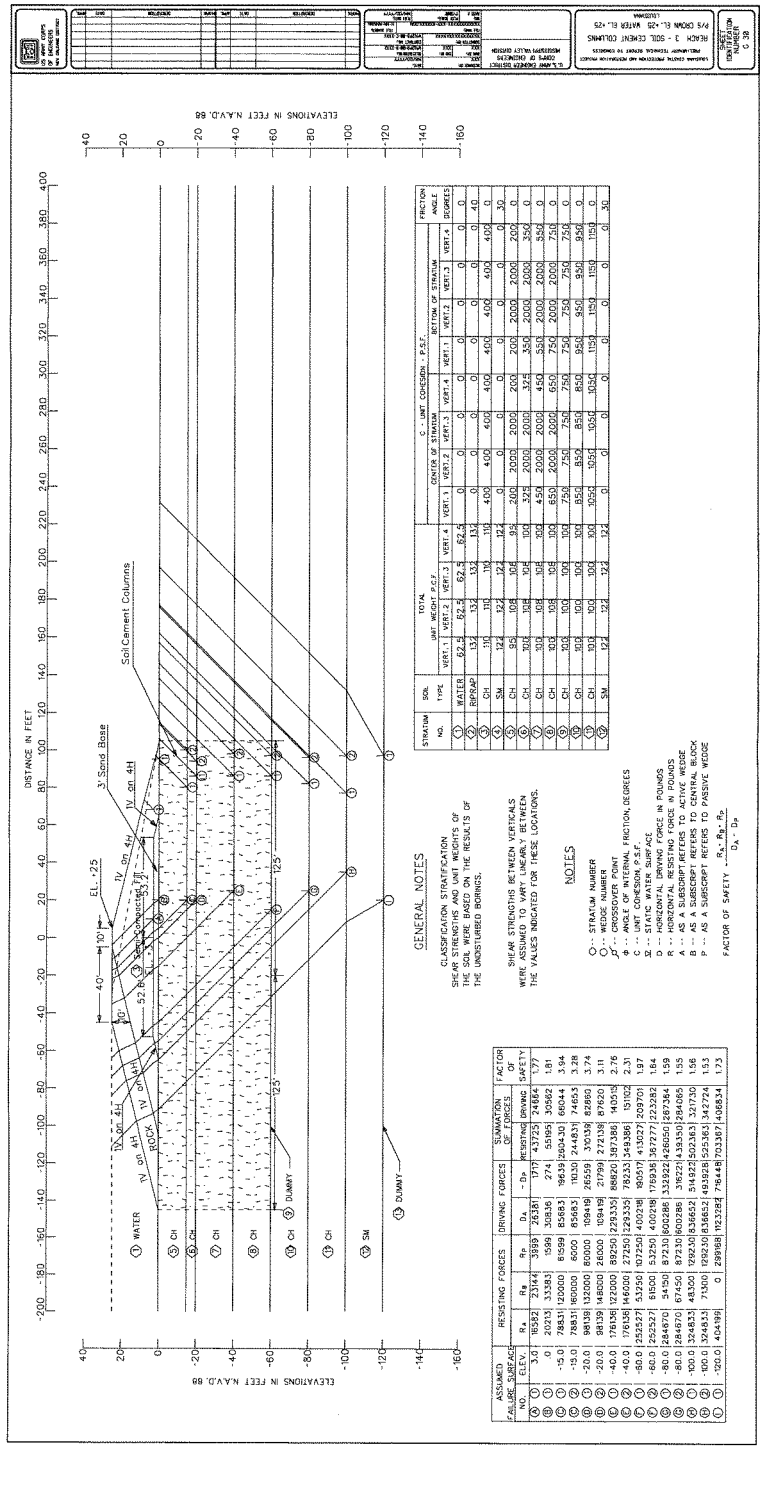
SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.





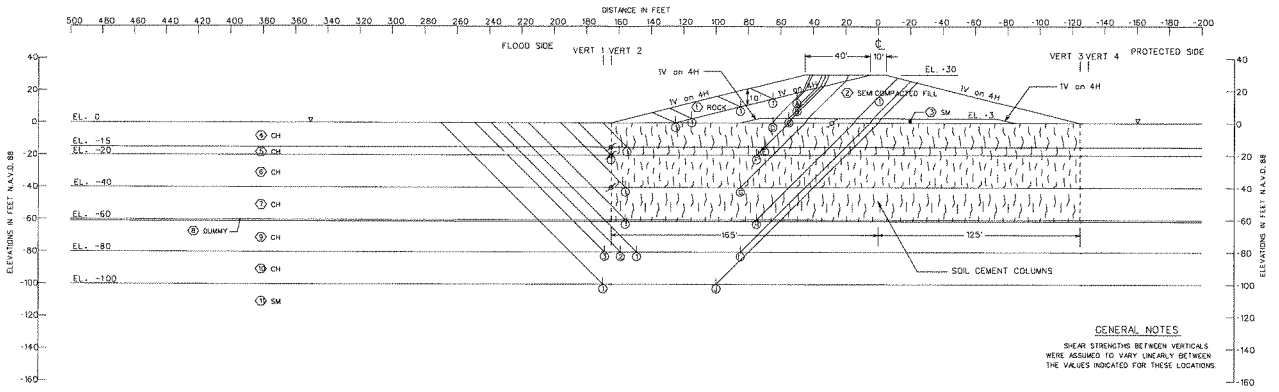
ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUBMERGED WEIGHT OF SOIL	FACTOR OF SAFETY
NO.	ELEV.	Rx	Ry	Rz	Dx	Dy	RESISTING	DRIVING	
(5) (1)	3.0	14631	14000	15937	28964	5111	443568	23853	1.87
(4) (1)	0	18919	18800	15836	37302	5111	57655	32791	1.60
(3) (1)	-15	218123	1400000	6000	88702	1981	2258431	75802	2.38
(2) (1)	-20	87966	140000	5064	41122	1981	1078736	36064	2.33
(1) (1)	-40.0	180418	1080000	27250	78153	31565	1536651	143662	2.20
(1) (1)	-60.0	261495	493000	51233	136070	17339	3641985	218875	1.67
(1) (1)	-80.0	285796	77250	87230	181923	31445	4145321	277369	1.63
(1) (1)	-100.0	334945	98650	122330	182028	43234	5662773	327056	1.71
(1) (1)	-120.0	413674	140000	200000	407824	75346	10703	582274	1.81

STRATUM NO.	SOIL TYPE	UNIT WEIGHT P.C.F.			CENTER OF STRATUM			UNIT COHESION P.S.F.			BOTTOM OF STRATUM			FRICTION ANGLE DEGREES
		VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	
(1)	ADOA	132	132	132	132	0	0	0	0	0	0	0	0	40
(2)	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
(3)	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
(4)	CH	95	95	108	108	195	200	200	200	195	200	200	200	0
(5)	CH	95	95	108	108	195	200	200	200	195	200	200	200	0
(6)	CH	100	108	108	108	392	450	2000	2000	554	550	2000	2000	0
(7)	CH	100	105	108	108	392	450	2000	2000	554	650	2000	2000	0
(8)	DUMMY	300	150	105	300	750	750	750	750	750	750	750	750	0
(9)	CH	300	300	105	300	850	850	850	850	950	950	950	950	0
(10)	CH	300	300	105	300	1000	1000	1000	1000	1050	1050	1050	1050	0
(11)	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
(12)	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
①	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
②	RIPRAP	134	134	134	134	0	0	0	0	0	0	0	0	40
③	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
④	SM	124	122	124	124	0	0	0	0	0	0	0	0	30
⑤	CH	95	104	104	95	2001	2003	2003	2001	2001	2001	2000	2000	0
⑥	CH	103	104	104	103	325	2000	2000	325	350	2000	2000	350	0
⑦	CH	103	103	103	103	550	2000	2000	550	550	2000	2000	550	0
⑧	CH	103	103	103	103	750	2000	2000	750	750	2000	2000	750	0
⑨	CH	103	103	103	103	750	750	750	750	750	750	750	750	0
⑩	CH	103	103	103	103	850	850	850	850	850	850	850	850	0
⑪	CH	103	103	103	103	1050	1050	1050	1050	1150	1150	1150	1150	0
⑫	SM	122	122	122	122	0	0	0	0	0	0	0	0	30

FAILURE SURFACE NO.	ASSUMED WEDGE ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_B$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	3.0	16382	23144	3989	26381	1777	43725	24604	1.77
Ⓑ ①	.0	20213	33383	1599	30836	274	55195	30562	1.81
Ⓒ ①	-15.0	78831	120000	61599	85683	19639	260430	66044	3.94
Ⓓ ①	-13.0	78831	160000	6000	85683	11030	244831	74653	3.28
Ⓔ ①	-20.0	98139	132000	80000	109419	26559	310139	82860	3.74
Ⓕ ①	-20.0	88139	148000	26000	109419	21799	272138	87620	3.11
Ⓖ ①	-40.0	176156	122000	89250	229335	88820	387386	140515	2.76
Ⓗ ①	-40.0	176156	146000	27250	229335	78233	349386	151102	2.31
Ⓘ ①	-60.0	252527	51750	51750	107250	40218	190517	413027	209701	1.97
Ⓙ ①	-60.0	252527	81450	53250	400218	1769.38	387277	1223282	1.84
Ⓚ ①	-80.0	284677	67450	87330	600286	1339.22	4269.60	2287364	1.59
Ⓛ ①	-80.0	284677	67450	87330	600286	1339.22	4269.60	2287364	1.59
Ⓜ ①	-100.0	324653	433.02	12820	856522	5145.22	5023.63	3427.20	1.65
Ⓝ ①	-100.0	324653	433.02	12820	856522	5145.22	5023.63	3427.20	1.65
Ⓞ ①	-100.0	324653	73300	12820	856522	4919.28	5251.63	3427.24	1.53
Ⓟ ①	-120.0	404991	0	28998	1123282	7564.48	7331.87	4508.34	1.73



CENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓐ ①	15.0	9163	6000	1544.2	14228	4286	30605	9837	3.08
Ⓑ ①	10.0	12919	14000	15430	24859	4286	42349	20573	2.08
Ⓒ ①	3.0	18501	24000	15087	43371	3755	57588	39606	1.45
Ⓓ ①	0	24481	24478	15400	48819	4279	114358	44532	2.57
Ⓔ ①	-15.0	84765	170000	60000	111329	13788	314765	97561	3.23
Ⓕ ①	-20.0	104764	178999	9230	134768	18999	294013	115767	2.54
Ⓖ ①	-40.0	186058	142000	27243	260160	77308	355301	182832	1.94
Ⓗ ①	-60.0	266809	60750	53230	441923	172307	380589	269618	1.41
Ⓘ ③	-80.0	298708	60800	87050	647504	323986	446558	323518	1.38
Ⓙ ②	-80.0	298708	70300	87050	647504	305387	456058	341917	1.33
Ⓙ ③	-80.0	298708	79800	87050	647504	304992	465558	342912	1.36
Ⓙ ①	-100.0	339907	80500	128050	884622	478987	549457	405635	1.35

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⊿ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

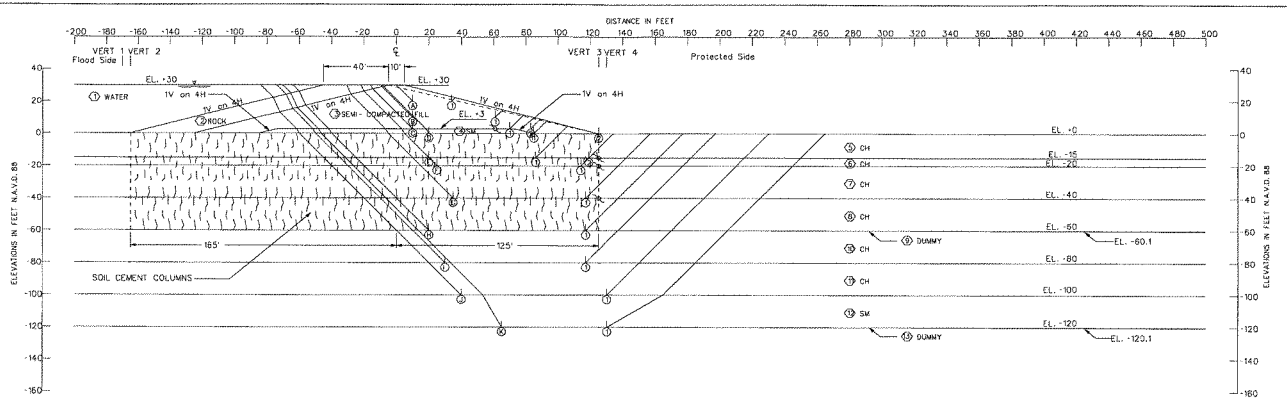
FACTOR OF SAFETY $= \dfrac{R_A + R_B + R_P}{D_A - D_P}$

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	95	108	108	95	325	2000	2000	325	350	2000	2000	350	0
⑥	CH	85	108	108	95	450	2000	2000	450	550	2000	2000	550	0
⑦	CH	95	108	108	95	650	2000	2000	650	750	2000	2000	750	0
⑧	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
⑨	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
⑩	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
⑪	SM	122	122	122	122	0	0	0	0	0	0	0	0	30

U.S. ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

REACH 3 - SOIL CEMENT COLUMNS
I/O CANAL EL. -30 WATER EL. +3

SHEET IDENTIFICATION NUMBER
G 31



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
5	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
7	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
8	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
9	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
10	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
11	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
12	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
13	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

ASSUMED FAILURE SURFACE NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	FACTOR OF SAFETY
A	1	20.0	7999	9600	1758	5154	332	19357	4822	4.01
B	1	10.0	15227	20400	3834	21708	1580	39481	20128	1.96
C	1	3.0	19837	23698	6872	40065	5074	50407	34991	1.44
D	2	3.0	19837	28153	4780	40065	2465	52760	37600	1.40
E	1	.0	23680	36780	6389	45991	4386	66849	41605	1.61
E	2	.0	23680	42787	0	45991	0	66467	45991	1.45
F	1	-15.0	81992	132000	63827	109414	26689	277818	82725	3.36
G	2	-15.0	81992	198000	6000	109414	1166	285992	98248	2.91
H	1	-20.0	101992	178000	26000	133135	21772	305992	111563	2.75
M	1	-40.0	180655	164000	27250	258825	78419	371905	180406	2.06
K	1	-60.0	256988	72750	53250	446583	176917	382986	269666	1.42
L	1	-80.0	289244	82650	87230	656335	315413	459124	340922	1.35
J	1	-100.0	329398	103500	129230	902288	493050	562128	409238	1.37
N	1	-120.0	408150	215774	300098	1150335	759944	923942	444391	2.08

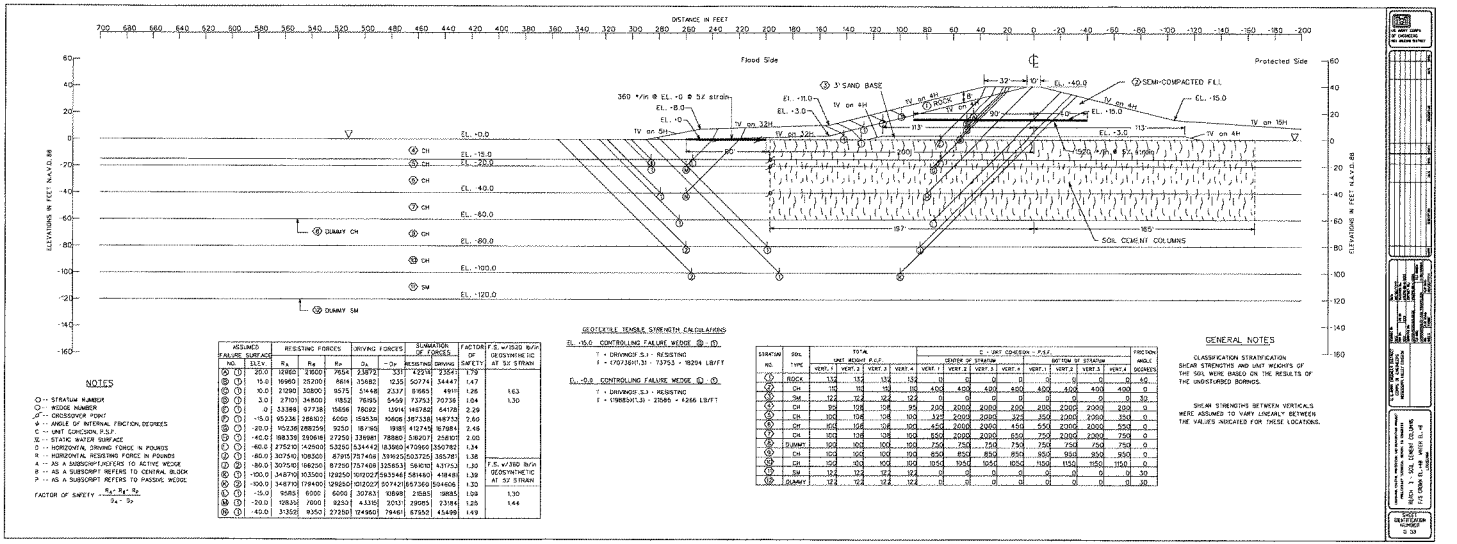
GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

O -- STRATUM NUMBER
O -- WEDGE NUMBER
-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
-- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$



NOTES

○ -- STRATUM NUMBER
◇ -- WEDGE NUMBER
-x- -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
W -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
R -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSING WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_R + R_P}{D_A - D_P}$

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/S3D Rein GEOSYNTHETIC AT 5% STRAIN
NO.	STRATUM	ELEV.	R_A	R_R	R_P	D_A	$-D_P$	RESISTING	DRIVING	SAFETY	
Ⓐ	①	20.0	12960	21600	7654	23872	331	42214	23541	1.79	
Ⓑ	①	15.0	16965	25200	8814	33682	1235	50774	34447	1.47	
Ⓒ	①	10.0	21290	30800	9575	51448	2337	81665	49111	1.66	1.63
Ⓓ	①	3.0	27101	34800	11852	76935	5458	73753	70735	1.04	1.30
Ⓔ	①	.0	33388	37738	15856	78092	13914	146782	64178	2.29	
Ⓕ	①	-15.0	45236	286103	6000	159536	10606	387338	148733	2.60	
Ⓖ	①	-29.0	102236	288256	9250	187905	19181	412745	187984	2.19	
Ⓗ	①	-40.0	188336	290618	27250	339981	78880	518207	258107	2.00	
Ⓒ	①	-60.0	275210	429500	53250	534442	183960	470960	350782	1.34	
Ⓓ	②	-80.0	307540	108500	87975	757406	334925	503725	365781	1.38	
Ⓔ	②	-80.0	307510	166200	87250	757408	325853	561010	431753	1.30	F.S. w/3D Rein GEOSYNTHETIC AT 5% STRAIN
Ⓕ	②	-100.0	348700	103500	129250	1012027	563946	581480	448481	1.30	
Ⓖ	②	-100.0	348700	178400	129250	1012027	557421	657360	594606	1.11	
Ⓒ	①	-15.0	9585	6000	6000	30763	10898	21585	19865	1.09	1.30
Ⓘ	①	-20.0	12833	7000	9250	43315	20131	29085	23184	1.25	1.44
Ⓙ	①	-40.0	31350	9350	27250	124980	79481	67550	45499	1.49	

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL -15.0 CONTROLLING FAILURE WEDGE Ⓑ - Ⓘ

T = DRIVING F.S. = RESISTING
T = 470736#1.31 = 73753 = 18204 LB/FT

EL. -0.0 CONTROLLING FAILURE WEDGE Ⓒ - Ⓘ

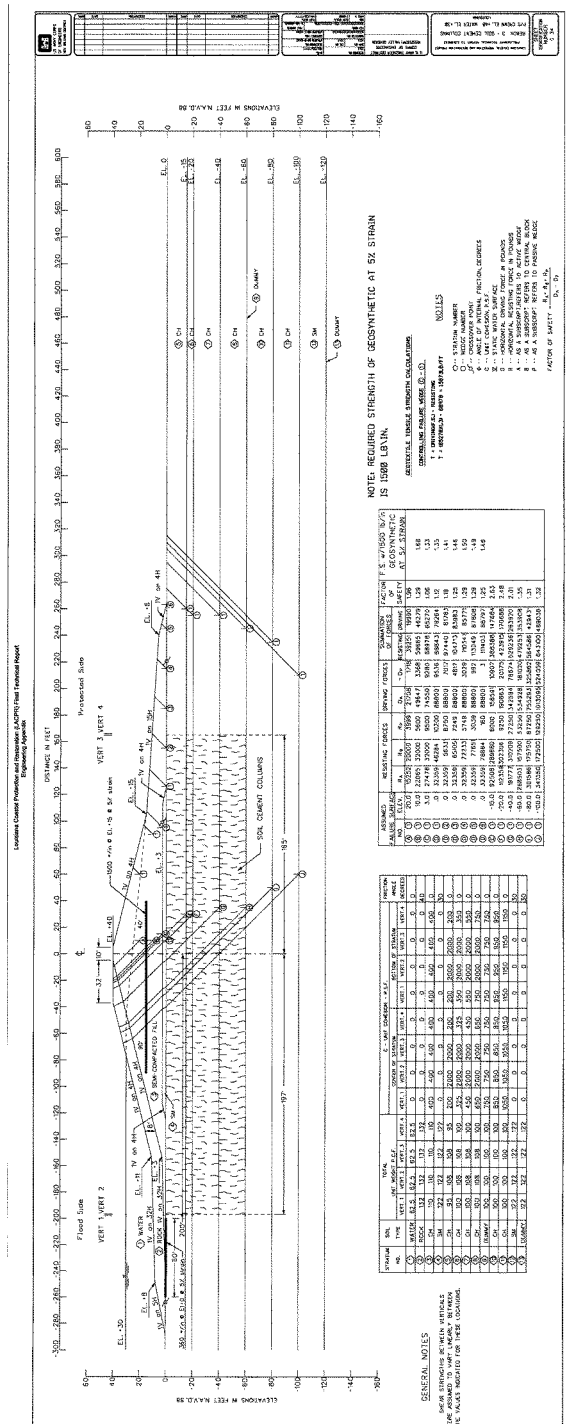
T = DRIVING F.S. = RESISTING
T = 1788341.31 = 27885 = 4266 LB/FT

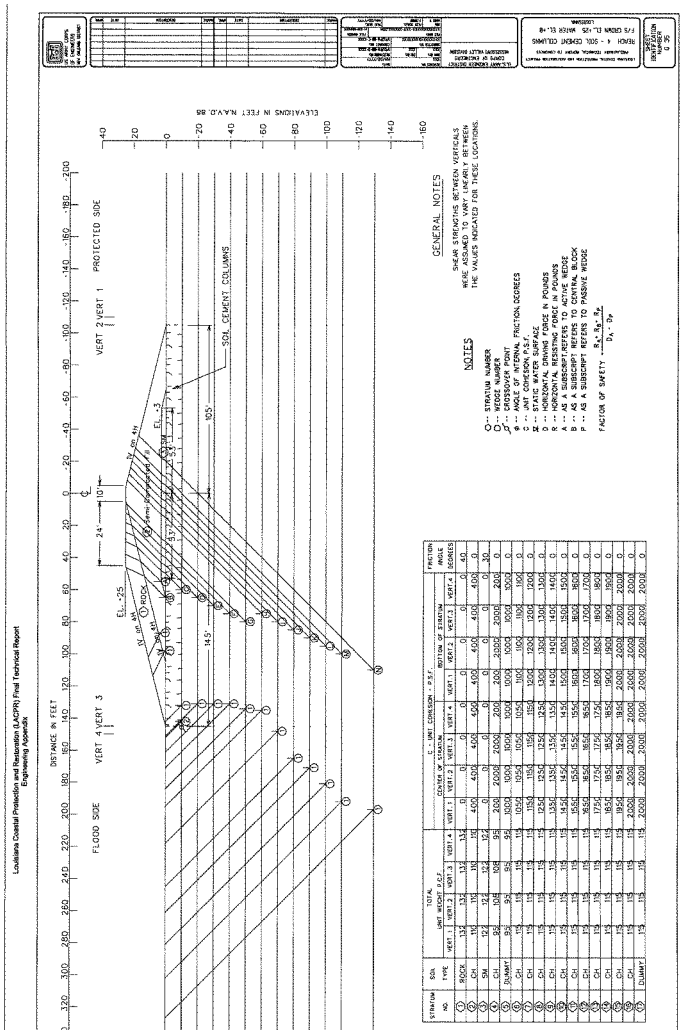
STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	DEGREES
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	45
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	CH	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	90	108	108	90	2000	2000	2000	2000	2000	2000	2000	2000	0
⑤	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
⑥	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
⑦	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
⑧	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
⑨	CH	100	100	100	100	850	850	850	850	900	900	900	900	0
⑩	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
⑪	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⑫	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

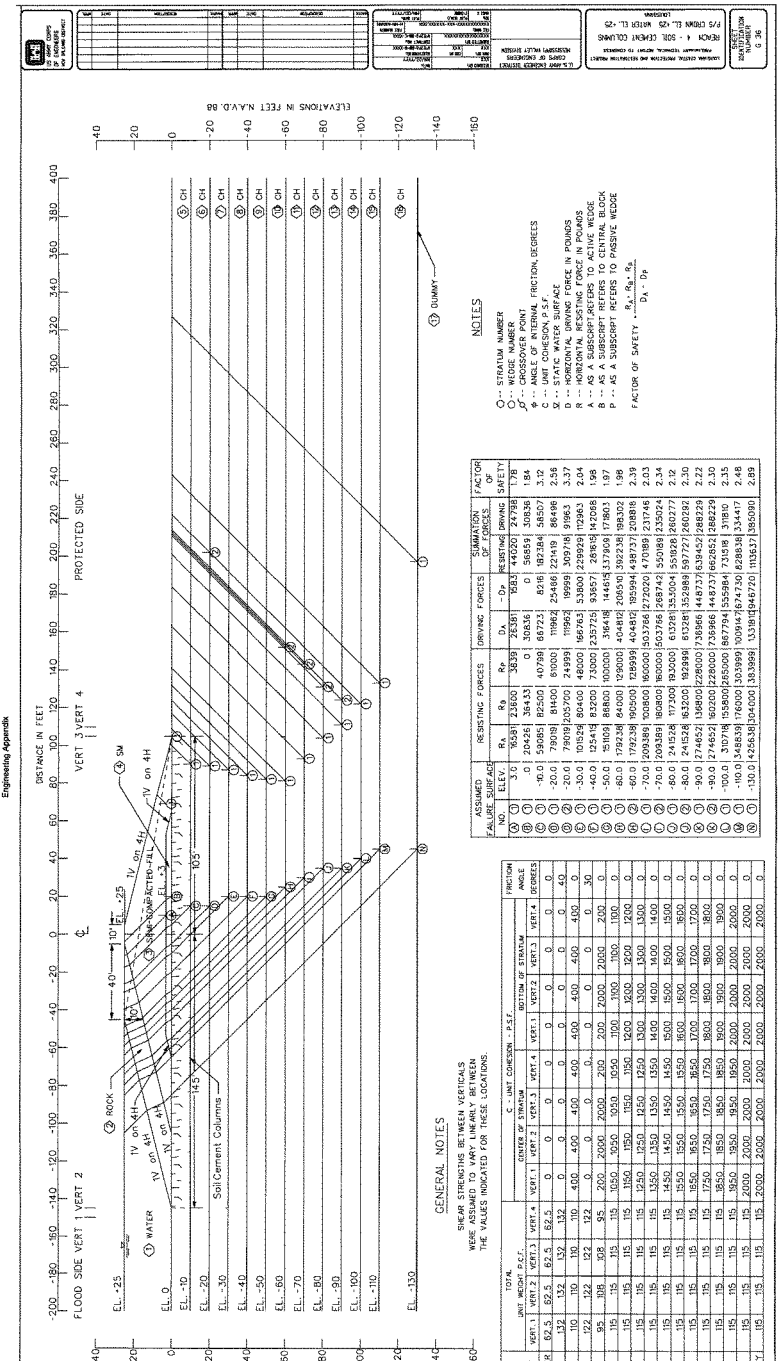
GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

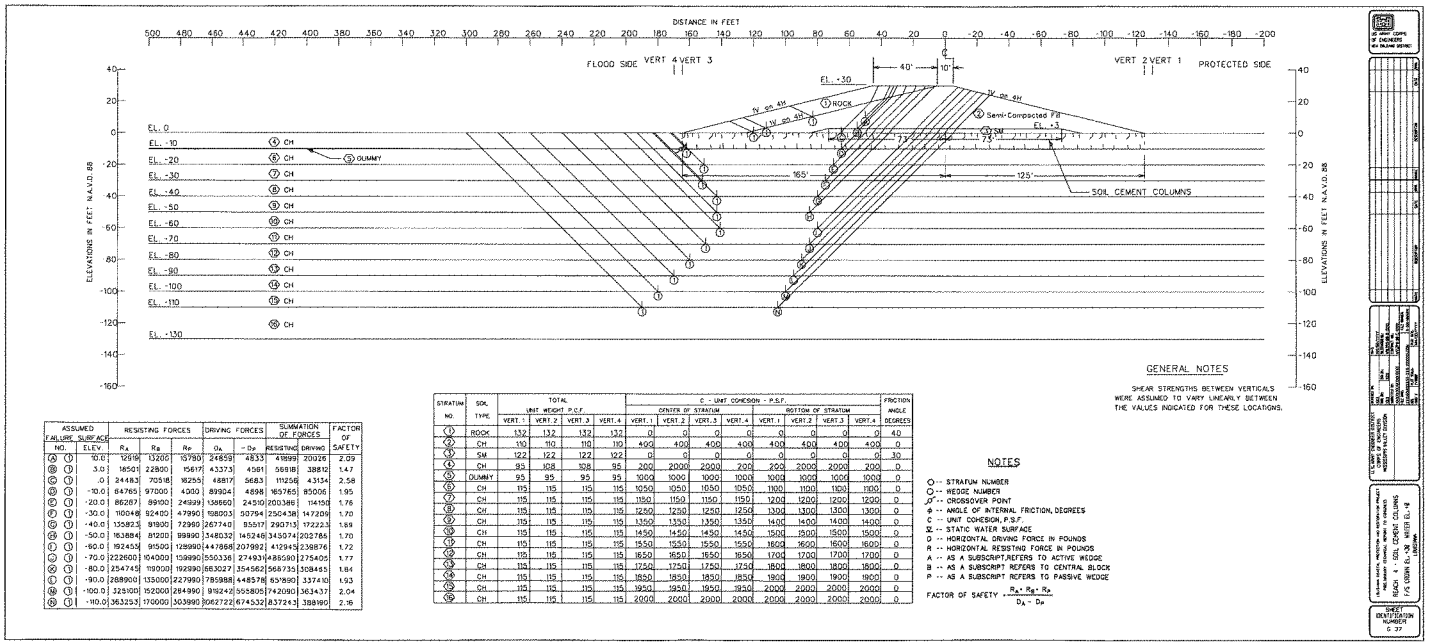

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
DISTANCE IN FEET
ELEVATIONS IN FEET NAVD 88
Protected Side
Flood Side
VERT 1
VERT 2
VERT 3
VERT 4
WATER
SEMI-COMPACTED FILL
SOIL CEMENT COLUMNS
GEOSYNTHETIC
GENERAL NOTES
NOTES
NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN
IS 1598 LB/IN.


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
DISTANCE IN FEET
ELEVATIONS IN FEET NAVD 88
FLOOD SIDE
PROTECTED SIDE
VERT 1
VERT 2
VERT 3
VERT 4
EL. 0
EL. -10
EL. -20
EL. -30
EL. -40
EL. -50
EL. -60
EL. -70
EL. -80
EL. -90
EL. -100
EL. -110
EL. -120
EL. -130
DUMMY
CH
SOIL CEMENT COLUMNS
ROCK
GENERAL NOTES
NOTES
SHEAR STRENGTHS BETWEEN VERTICALS
WERE ASSIGNED TO VARIATION LINES
THE VALUES INDICATED FOR THESE LOCATIONS
0 -- STRATUM NUMBER
C -- COHESION INTERCEPT
D -- DISPERSION POINT
φ -- ANGLE OF INTERNAL FRICTION, DEGREES
U -- UNIT CONDITION, P.S.F.
Z -- UNIT WEIGHT, P.C.F.
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE
FACTOR OF SAFETY = Ra + Rb + Rc
Da + Dp
REACH 4 - SOIL CEMENT COLUMNS
F/S CANAL DL-25 WATER EL.+8
LOUISIANA



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
①	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
②	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
③	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
④	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⑤	CH	95	108	95	108	200	200	200	200	2000	2000	2000	2000	0
⑥	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
⑦	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
⑧	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
⑨	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
⑩	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
⑪	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
⑫	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
⑬	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
⑭	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
⑮	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
⑯	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0
⑰	DUMMY	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES Ra	Rb	Rp	DRIVING FORCES Da	- Dp	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
⑯ ①	3.0	165381	238600	38319	263381	9083	442020	247298	1.78
⑯ ①	0	204261	384133	0	308536	0	588594	308536	1.84
⑯ ①	-10.0	590851	822500	407981	867723	8216	1821384	585507	3.12
⑯ ①	-20.0	790198	814500	810000	1118942	254616	2214698	864326	2.58
⑯ ①	-30.0	1000000	742000	742000	1378963	490000	2484000	888963	2.79
⑯ ①	-40.0	1015128	804600	482600	1667163	538000	2299328	1129163	2.04
⑯ ①	-40.0	1254418	832200	733001	2357251	939577	2650153	1417674	1.87
⑯ ①	-50.0	1511091	888800	1030001	3164181	1446151	3179091	1717803	1.85
⑯ ①	-60.0	1792138	841000	1290001	4044181	2065901	3922138	1978302	1.98
⑯ ②	-60.0	1792138	1905001	1289999	4044181	1959994	4387137	2084187	2.39
⑯ ①	-70.0	2093891	1808001	1800000	5037661	2687942	5501891	2349719	2.34
⑯ ①	-80.0	2415281	1173001	1930001	6139281	3530041	5519281	2609240	2.12
⑯ ①	-80.0	2415281	1923999	1930001	6139281	3529899	5977271	2609382	2.30
⑯ ①	-90.0	2744652	1368001	2280001	7369661	4487371	6394521	2882291	2.22
⑯ ②	-90.0	2744652	1902001	2280001	7369661	4487371	6662852	2882291	2.30
⑯ ①	-100.0	3107181	1598001	2650001	8677941	5559841	7310181	3118101	2.35
⑯ ①	-110.0	3488391	1760001	3039991	10094761	6730741	8288181	3364421	2.48
⑯ ①	-130.0	4256381	2040001	3839991	13381891	9467201	11136171	3865091	2.89



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

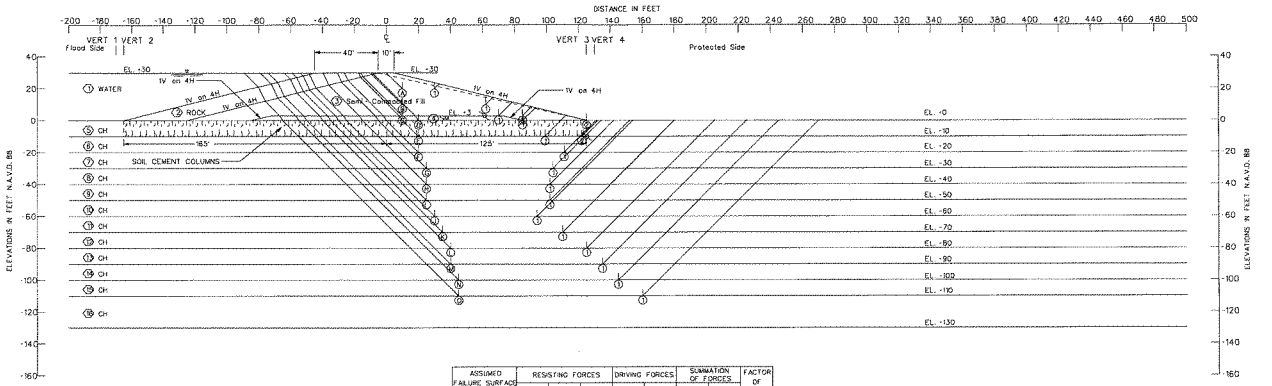
ASSUMED FAILURE SURFACE NO.		ELEV	RESISTING FORCES Rx	Rz	Rw	DRIVING FORCES Ox	-Dw	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A	1	10.0	12916	13200	15780	24859	4833	41896	29026	2.09
B	1	3.0	16501	22800	15617	43373	4561	56918	38812	1.47
C	1	.0	24483	70518	16255	48817	5683	111256	43134	2.58
D	1	-10.0	84765	97000	4000	89904	4898	185765	85006	1.95
E	1	-20.0	86287	89900	24929	136960	24510	200386	114558	1.76
F	1	-30.0	110048	82400	47990	198003	50794	250438	147209	1.70
G	1	-40.0	135823	91800	72990	267740	95517	290715	172223	1.69
H	1	-50.0	163884	81200	99890	348032	145246	343074	202786	1.70
I	1	-60.0	192453	91500	125990	447858	207982	412945	239876	1.72
J	1	-70.0	222600	104000	159990	550338	274934	485590	275405	1.77
K	1	-80.0	254745	119000	192990	663027	354562	566735	308465	1.84
L	1	-90.0	288900	133000	227990	785988	448578	651890	337410	1.93
M	1	-100.0	325100	152000	284990	919242	555805	742090	363437	2.04
N	1	-110.0	363253	170000	303990	1062722	674532	837243	388190	2.16

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	93	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	DUMMY	95	95	95	95	1000	1000	1000	1000	1000	1000	1000	1000	0
6	CH	113	113	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

NOTES

○ -- STRATUM NUMBER
⬡ -- WEDGE NUMBER
⁄ -- CROSSOVER POINT
φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
O -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{O_A - D_W}$



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	DEGREES
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A	1	20.0	7999	8000	2398	5154	818	18397	4536	4.06
B	1	10.0	15227	20800	3679	21708	1454	39706	20254	1.96
C	1	3.0	19837	23701	6879	40065	5083	50417	34982	1.44
C	2	3.0	19837	28927	4479	40065	2155	53243	37910	1.40
D	1	.0	23680	36796	6399	45991	4398	66875	41593	1.61
D	2	.0	23680	42834	0	45991	-0	66514	45991	1.45
E	1	-13.0	82532	86800	42560	85483	11877	191992	72606	2.61
E	2	-10.0	62532	112200	4000	85483	4874	178732	80609	2.22
F	1	-20.0	62279	100100	25011	138360	23984	207390	112366	1.85
G	1	-30.0	104655	94800	48000	185944	55411	247455	140533	1.76
H	1	-40.0	128898	100100	73000	270406	94870	301998	175536	1.72
I	1	-50.0	155196	107800	100000	356598	144618	362996	211979	1.71
J	1	-60.0	183463	96000	129000	449795	213108	408463	236689	1.73
K	1	-70.0	213714	120000	160000	553557	274436	493714	279121	1.77
L	1	-80.0	245988	144500	193000	667891	352992	583466	314899	1.85
M	1	-90.0	279264	171000	228000	797240	448738	678264	348501	1.95
N	1	-100.0	315349	190000	265000	933053	555887	770334	377066	2.04
O	1	-110.0	352494	230000	304001	1083879	674734	886495	409137	2.17

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
// -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
W -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

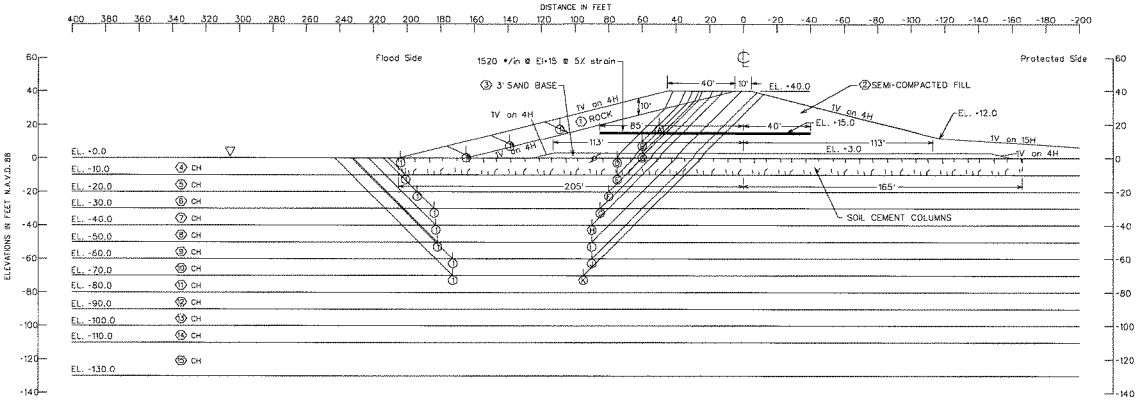
FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

SHEET IDENTIFICATION NUMBER C-28



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1520 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
④	① 20.0	12918	23600	11520	24859	-437	48138	25318	1.90	
⑱	① 10.0	20919	31800	13219	51805	1138	65738	50667	1.30	
⑯	① 3.0	26858	42000	13536	78703	1524	82194	77179	1.06	1.30
⑬	① 0	34133	147614	10	82290	3	181757	82287	2.21	
⑤	① -10.0	74415	138600	4001	124377	5014	217016	129363	1.68	
⑦	① -20.0	85946	125400	25000	192778	23286	236346	169492	1.45	
⑨	① -30.0	119764	118800	48000	261805	56624	286564	205181	1.40	
⑪	① -40.0	145868	120900	73000	341465	95582	339769	245883	1.38	
①	① -50.0	174480	128800	100000	437120	146073	403280	291047	1.38	
③	① -60.0	202775	124500	128000	543807	216787	456275	327020	1.40	
⑰	① -70.0	232820	124800	160000	656059	289535	517720	366524	1.41	

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	VERT.1	VERT.2	VERT.3	VERT.4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	105	105	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1101	1100	0
⑥	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
⑦	CH	115	115	115	115	1250	1254	1250	1250	1300	1300	1302	1302	0
⑧	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
⑨	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
⑩	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
⑪	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
⑫	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
⑬	CH	115	115	115	115	1850	1854	1850	1850	1900	1900	1900	1900	0
⑭	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
⑮	CH	119	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

NOTES

◇ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⊗ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. +15.0 CONTROLLING FAILURE WEDGE ⑮ - ①

T = DRIVING(F.S.) - RESISTING
T = (77179)(1.3) - 82194 = 18140 LB/FT

GENERAL NOTES

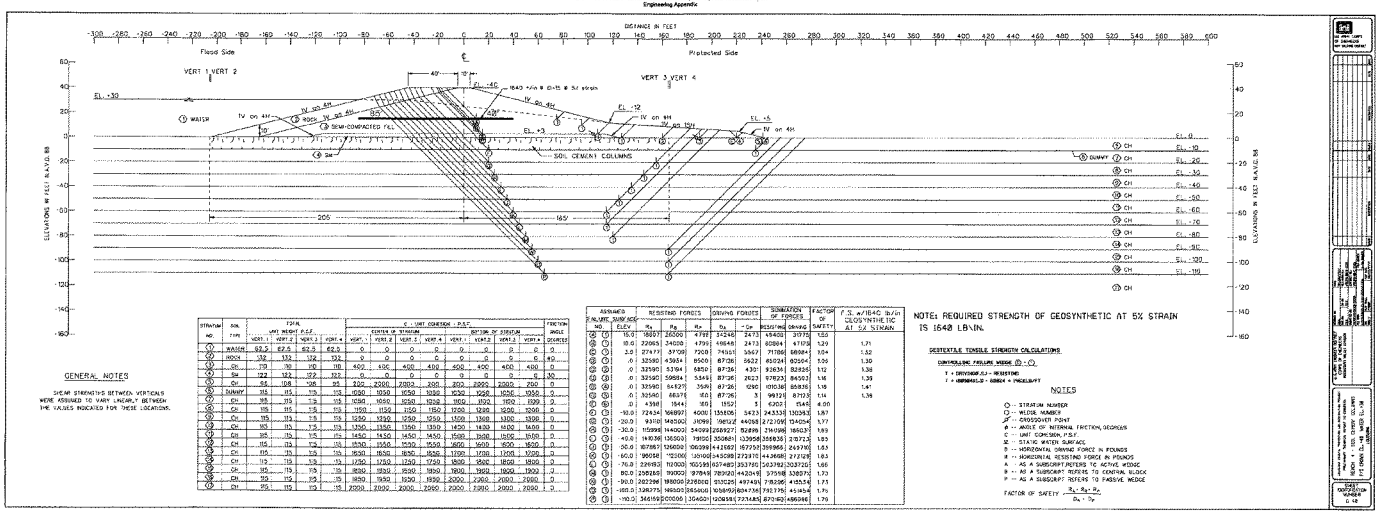
CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
F/S CROWN EL. +15 - WATER EL. +8

SHEET IDENTIFICATION NUMBER
C 39



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSIGNED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE (DEGREES)
1	WASH	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	118	118	118	118	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	85	108	108	85	220	2500	2000	200	200	2500	2000	200	0
6	SANDY	115	115	115	115	1050	1050	1050	1050	1050	1050	1050	1050	0
7	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
8	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
9	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
10	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
11	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
12	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
13	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
14	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
15	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
16	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
17	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSIGNED SLIP SURFACE NO.		RESISTING FORCES ELEV	R_A	R_B	R_C	R_R	DRIVING FORCES - D_R	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	P.S. #/1640 lb/in GEOSYNTHETIC AT 5% STRAIN
1	C	15.0	19807	26000	4788	34240	24743	134601	31970	1.33	
2	C	13.6	22065	24020	4739	49648	24743	60864	41178	1.29	1.71
3	C	2.0	274771	51728	7308	74161	3847	71786	68984	1.04	1.32
4	C	.0	32590	43934	8508	87126	5622	65024	60504	1.08	1.30
5	C	.0	32590	53784	6830	87126	4301	92634	82836	1.12	1.36
6	C	.0	32590	59884	5346	87126	2621	97823	86903	1.13	1.39
7	C	.0	32590	64127	3691	87126	1290	101036	88836	1.14	1.41
8	C	.0	32590	66371	1761	87126	3	99129	87123	1.14	1.39
9	C	.0	4398	1944	188	1152	5	6203	1345	4.61	
10	C	-10.0	72434	168897	4000	134105	5423	243334	130343	1.87	
11	C	-20.0	93118	148362	18986	98162	44058	272109	134054	1.77	
12	C	-30.0	115998	114003	34099	2269277	82899	2141086	186235	1.89	
13	C	+0.0	19036	136503	19100	350861	13295	356835	215715	1.83	
14	C	00.0	107857	126038	08098	1412921	19737	3989951	2169790	1.83	
15	C	-60.0	99058	-10100	-101008	3430918	2723170	4426818	2727128	1.63	
16	C	-70.0	239852	112900	102598	1027482	353790	1403782	2037250	1.66	
17	C	80.0	258260	119002	97949	783920	442049	975598	1328270	1.73	
18	C	-90.0	282298	118000	226000	3132251	497439	7182916	4155354	1.73	
19	C	-100.0	1382775	1496000	2856000	9368926	1834736	7183776	4154854	1.74	
20	C	-110.0	346159	2000006	1064001	1206588	7237485	8731482	4866986	1.79	

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1640 LBS/IN.

DESIRTABLE TENSILE STRENGTH CALCULATIONS

DIMENSIONLESS FAILURE WEDGE (D - C)

T = DRIVE/ANGLE - RESISTING
T = HORIZONTAL - RANKIN + PASSIVE/LIFT

NOTES

O - STRATUM NUMBER
O - WEDGE NUMBER
- CROSSOVER POINT
ø - ANGLE OF INTERNAL FRICTION, DEGREES
C - UNIT COHESION, P.S.F.
M - STATIC WATER SURFACE
S - HORIZONTAL DRIVING FORCE IN POUNDS
R - HORIZONTAL RESISTING FORCE IN POUNDS
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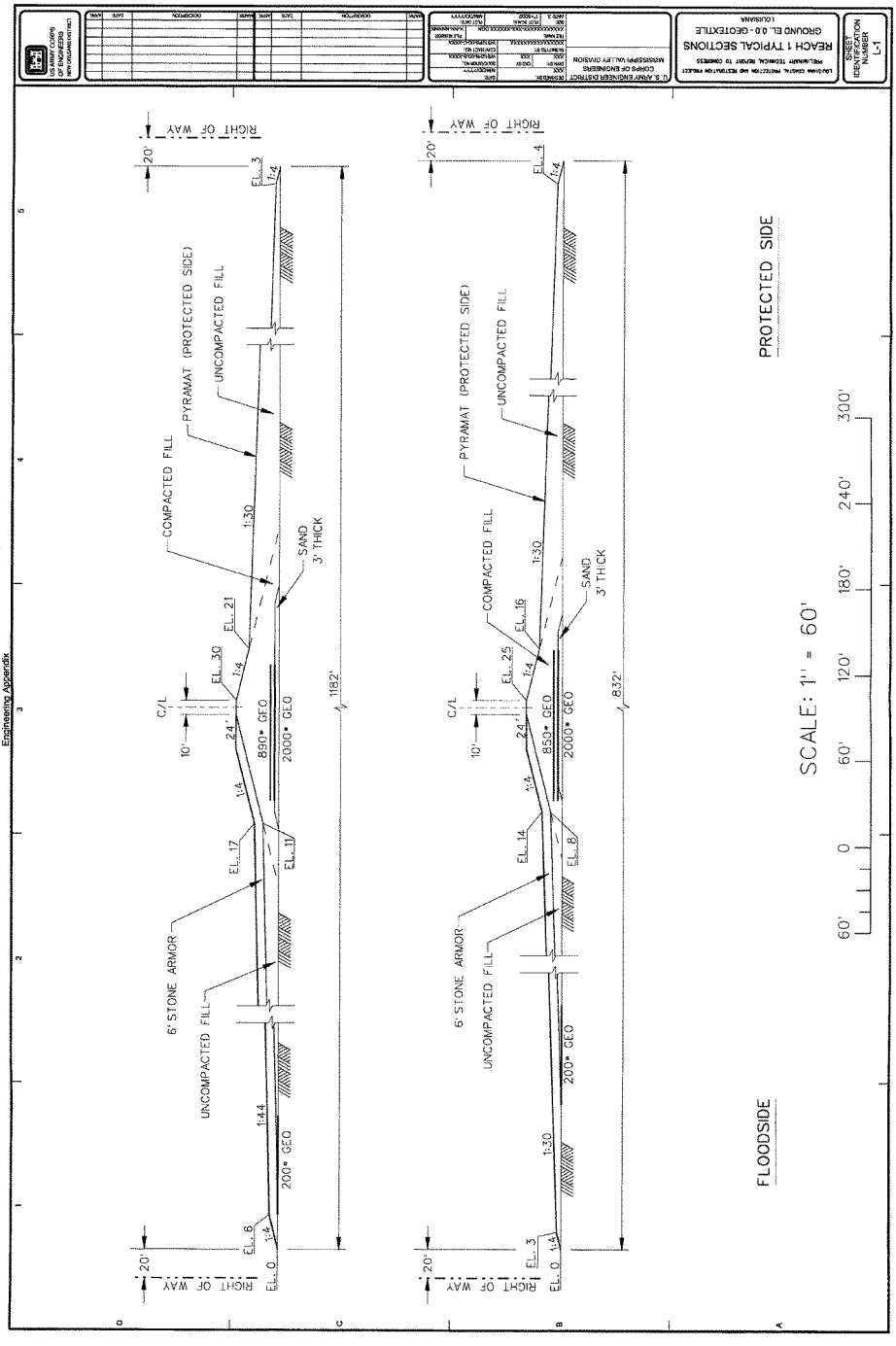
B - AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P - AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_C}{D_A + D_P}$



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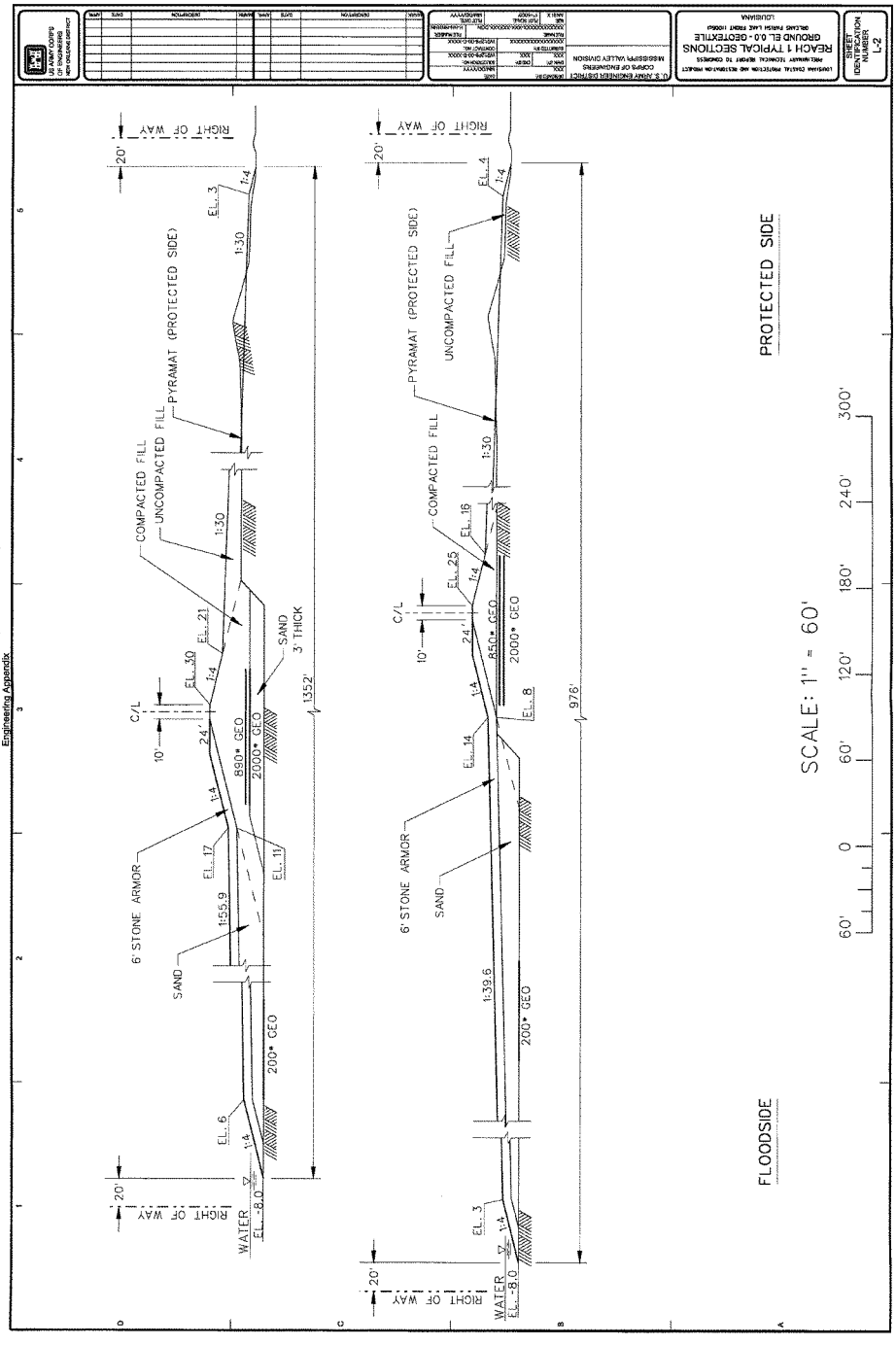

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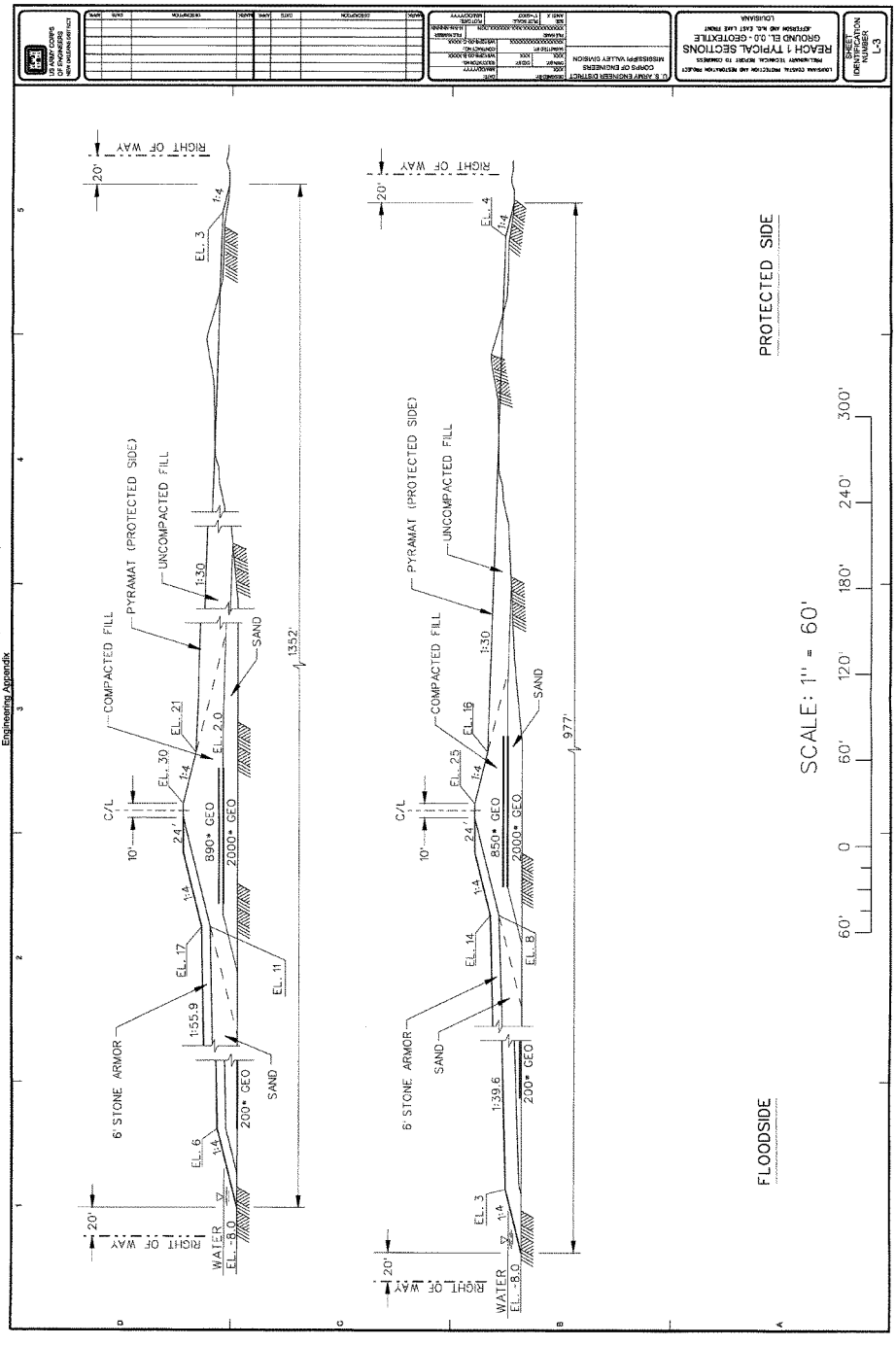
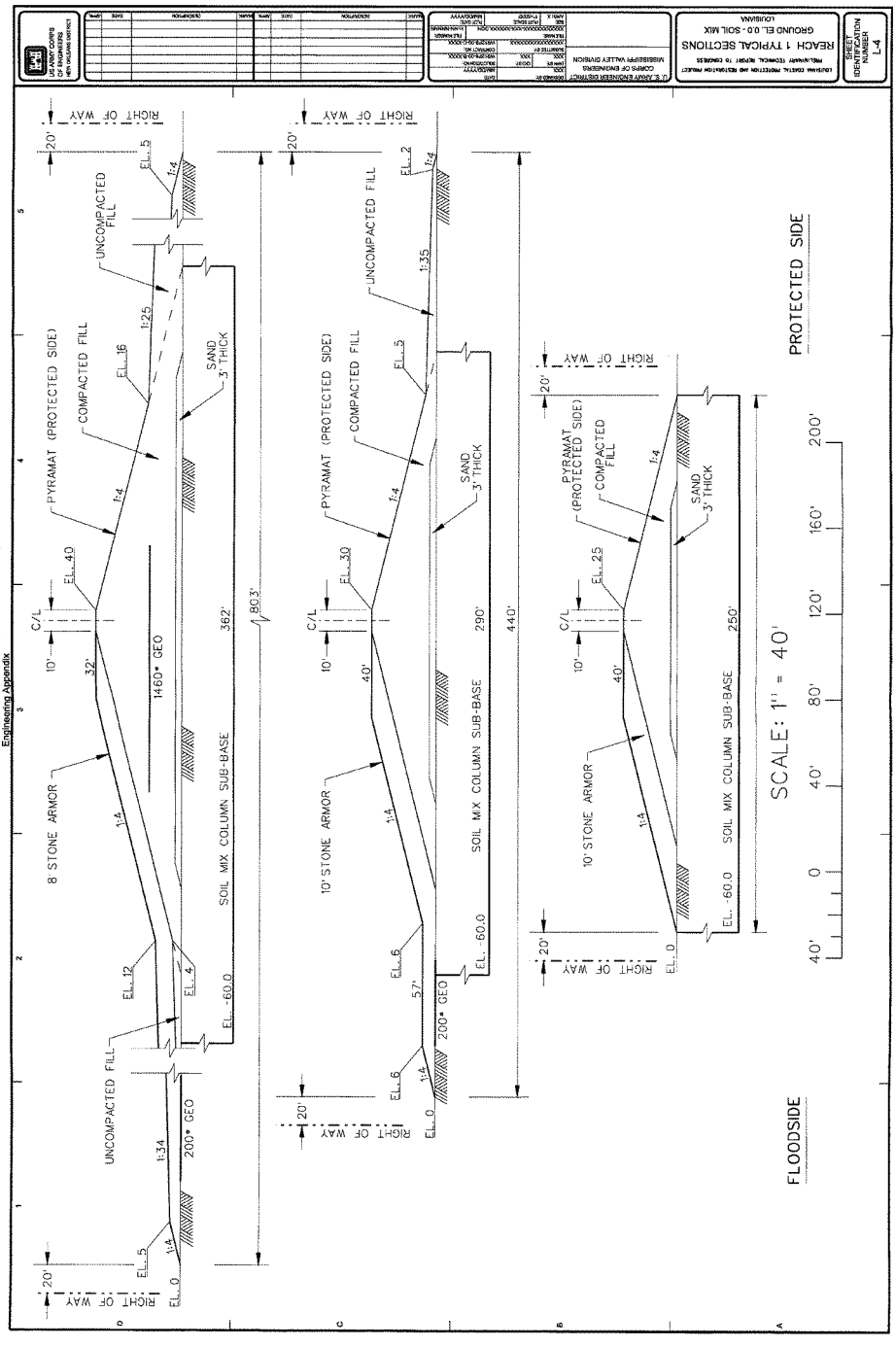

RIGHT OF WAY
20'
EL. 3
1:4
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
1:30
SAND
1352'
C/L
10'
EL. 30
EL. 21
EL. 2.0
24'
1:4
890* GEO
2000* GEO
EL. 17
1:55.9
EL. 11
6' STONE ARMOR
SAND
EL. 6
200* GEO
WATER
EL. -8.0
PROTECTED SIDE
EL. 4
EL. 16
EL. 25
850* GEO
977'
EL. 14
EL. 8
1:39.6
EL. 3
FLOODSIDE
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
REACH 1 TYPICAL SECTIONS
GROUND EL 0.0 - GEOTEXTILE
JEFFERSON AND ST. TAMMANY
LOUISIANA
SHEET IDENTIFICATION NUMBER L-3



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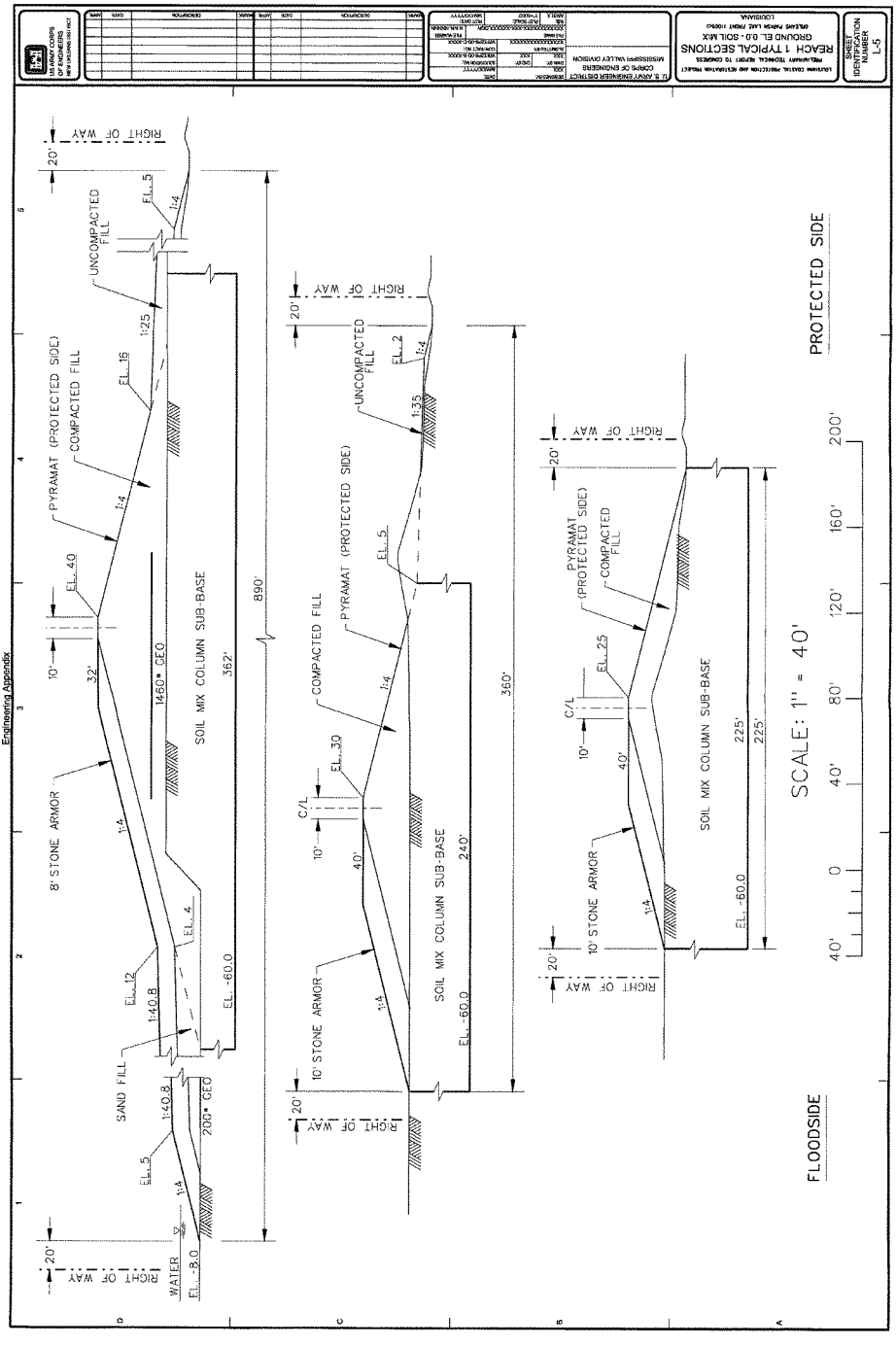

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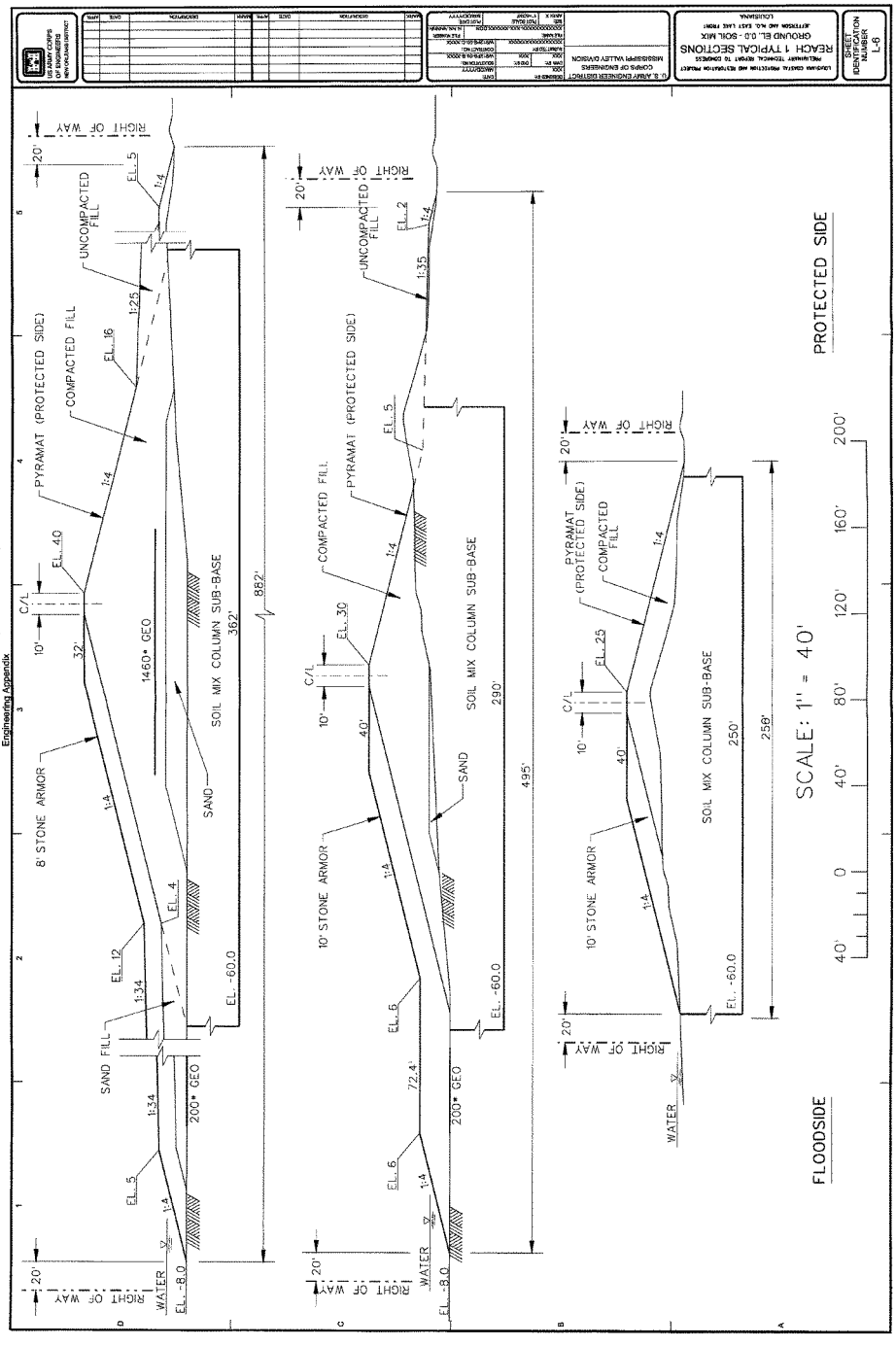
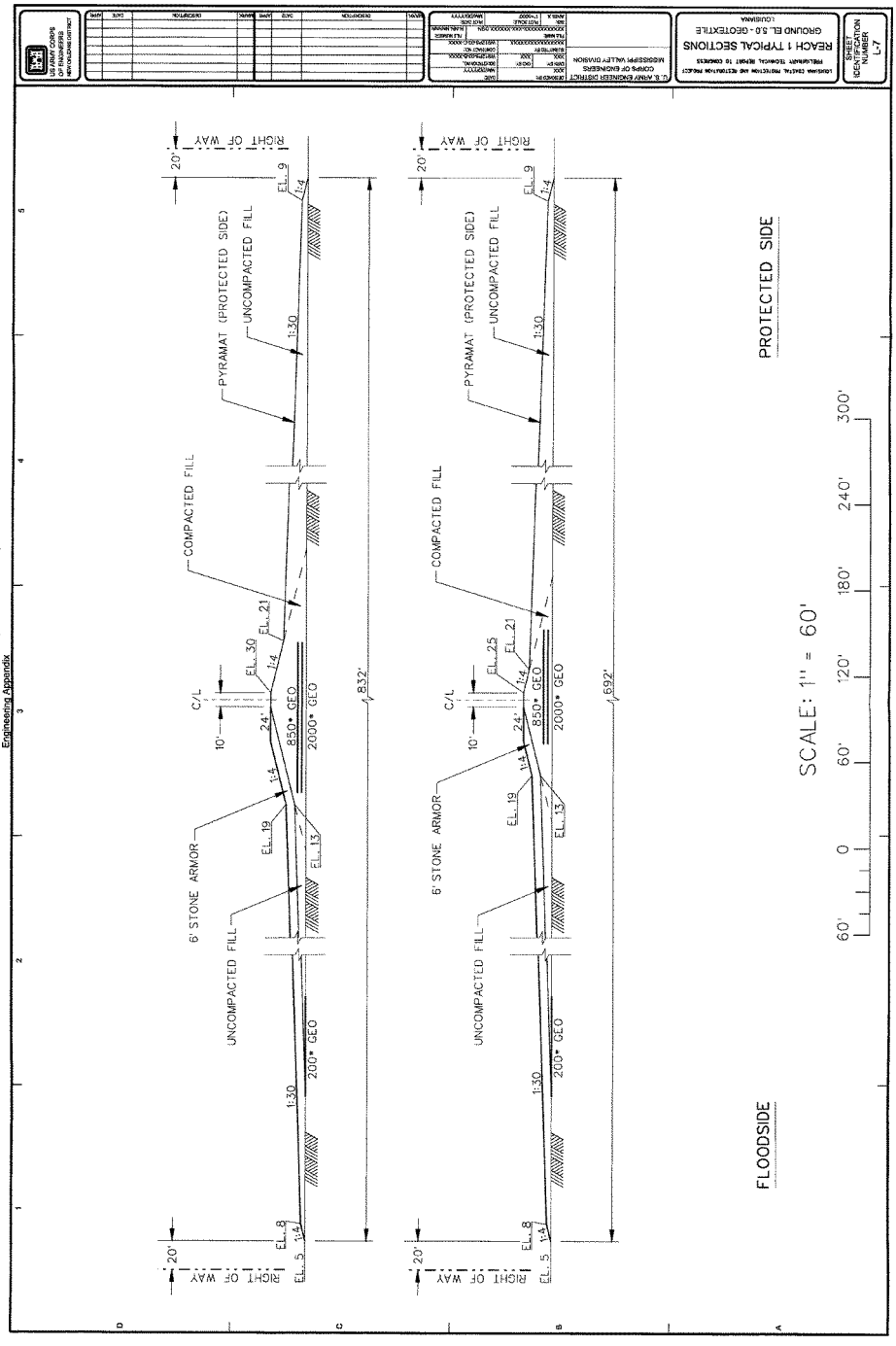
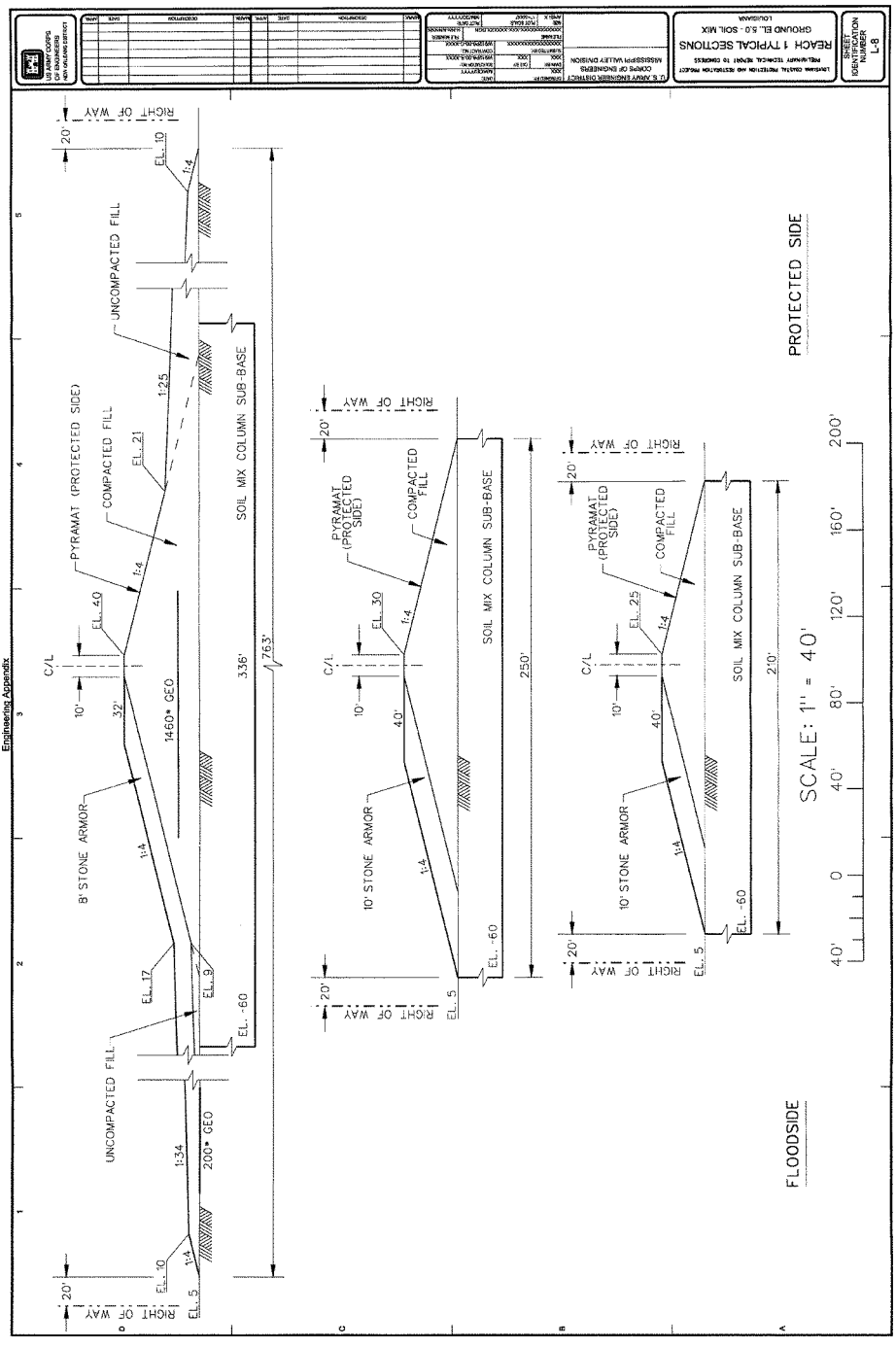

REACH 1 TYPICAL SECTIONS
GROUND EL. 0.0 - SOIL MIX
SCALE: 1" = 40'
PROTECTED SIDE
FLOODSIDE
RIGHT OF WAY
20'
UNCOMPACTED FILL
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
8' STONE ARMOR
10' STONE ARMOR
SAND FILL
SAND
SOIL MIX COLUMN SUB-BASE
1460* GEO
200* GEO
WATER
EL. 40
EL. 16
EL. 12
EL. 5
EL. 4
EL. 2
EL. 30
EL. 25
EL. 6
EL. -8.0
EL. -60.0
882'
362'
495'
290'
258'
250'
72.4'
32'
40'
10'
1:4
1:25
1:34
1:35


REACH 1 TYPICAL SECTIONS
GROUND EL. 5.0 - GEOTEXTILE
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'
PROTECTED SIDE
FLOODSIDE
RIGHT OF WAY
20'
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
COMPACTED FILL
6' STONE ARMOR
C/L
10'
24'
1:4
1:30
EL. 9
EL. 30
EL. 21
EL. 25
EL. 19
EL. 13
EL. 8
EL. 5
850# GEO
2000# GEO
200# GEO
832'
692'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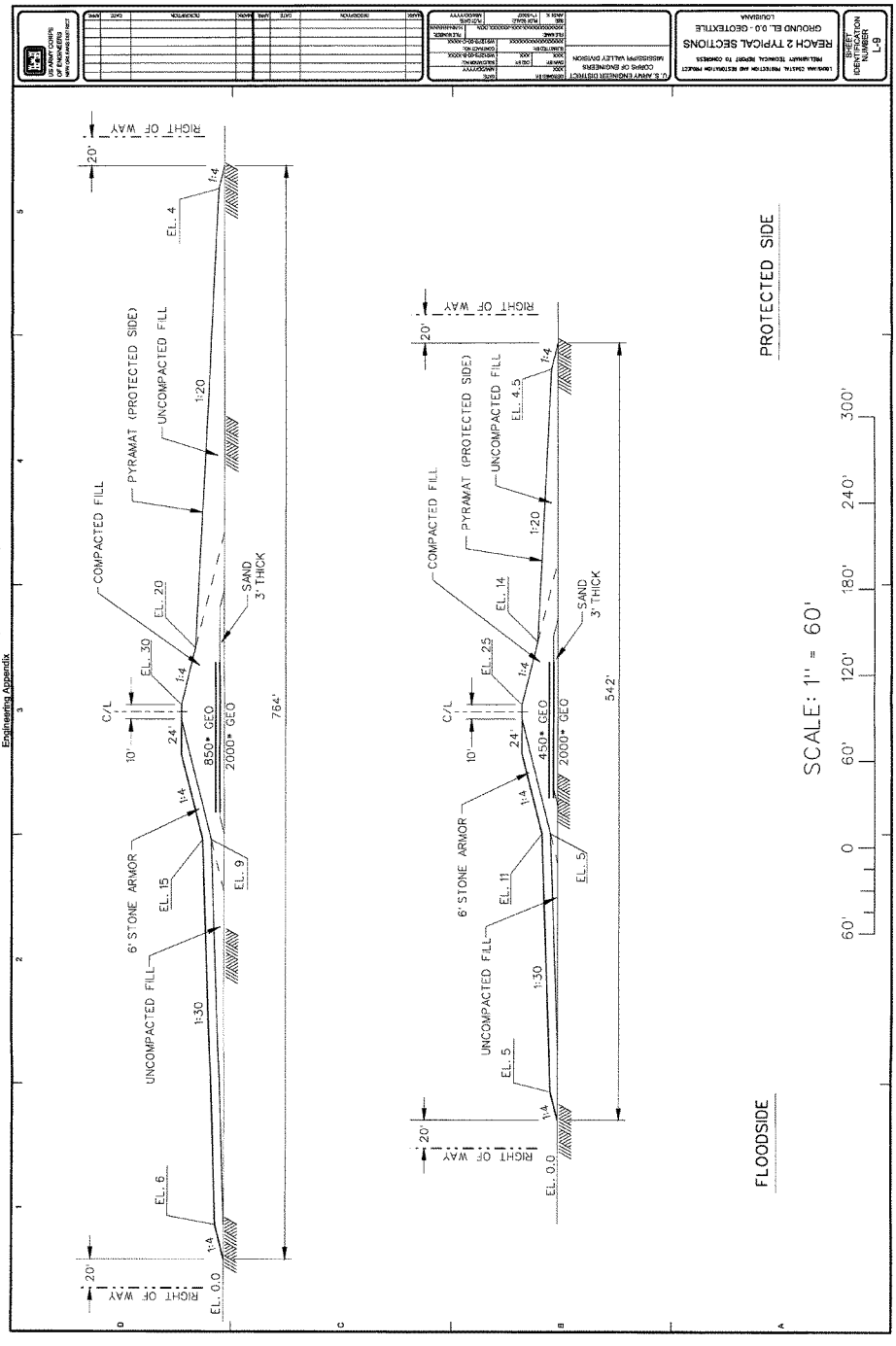
REACH 1 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER
L-8
RIGHT OF WAY
20'
EL. 10
1:4
UNCOMPACTED FILL
1:25
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 21
EL. 40
10'
32'
1460* GEO
8' STONE ARMOR
1:4
EL. 17
EL. 9
EL. -60
336'
763'
SOIL MIX COLUMN SUB-BASE
1:34
200* GEO
EL. 5
FLOODSIDE
PROTECTED SIDE
C/L
40'
10' STONE ARMOR
EL. 30
250'
EL. 25
210'
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'



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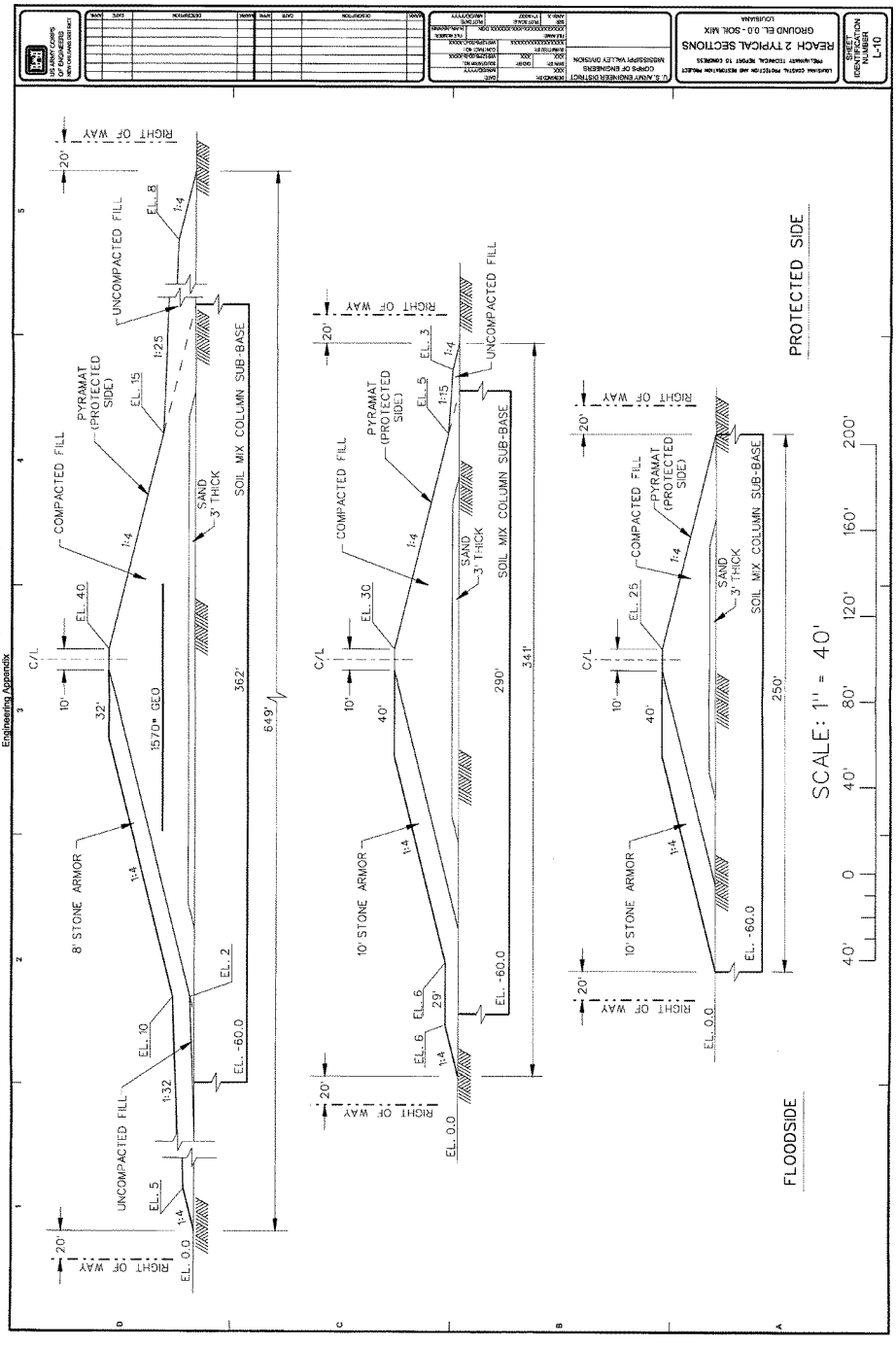
REACH 2 TYPICAL SECTIONS
GROUND EL. 0.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-10
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
RIGHT OF WAY
20'
UNCOMPACTED FILL
EL. 8
1:4
EL. 15
1:25
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 40
1:4
SAND 3' THICK
SOIL MIX COLUMN SUB-BASE
C/L
10'
32'
1570# GEO
362'
649'
8' STONE ARMOR
1:4
EL. 10
1:32
EL. 5
1:4
EL. 2
EL. -60.0
EL. 0.0
UNCOMPACTED FILL
EL. 3
1:4
1:15
EL. 5
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 30
1:4
SAND 3' THICK
SOIL MIX COLUMN SUB-BASE
40'
290'
341'
10' STONE ARMOR
1:4
EL. 6
29'
1:4
EL. 6
EL. -60.0
EL. 0.0
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 25
1:4
SAND 3' THICK
SOIL MIX COLUMN SUB-BASE
250'
10' STONE ARMOR
1:4
EL. -60.0
EL. 0.0
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'
Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
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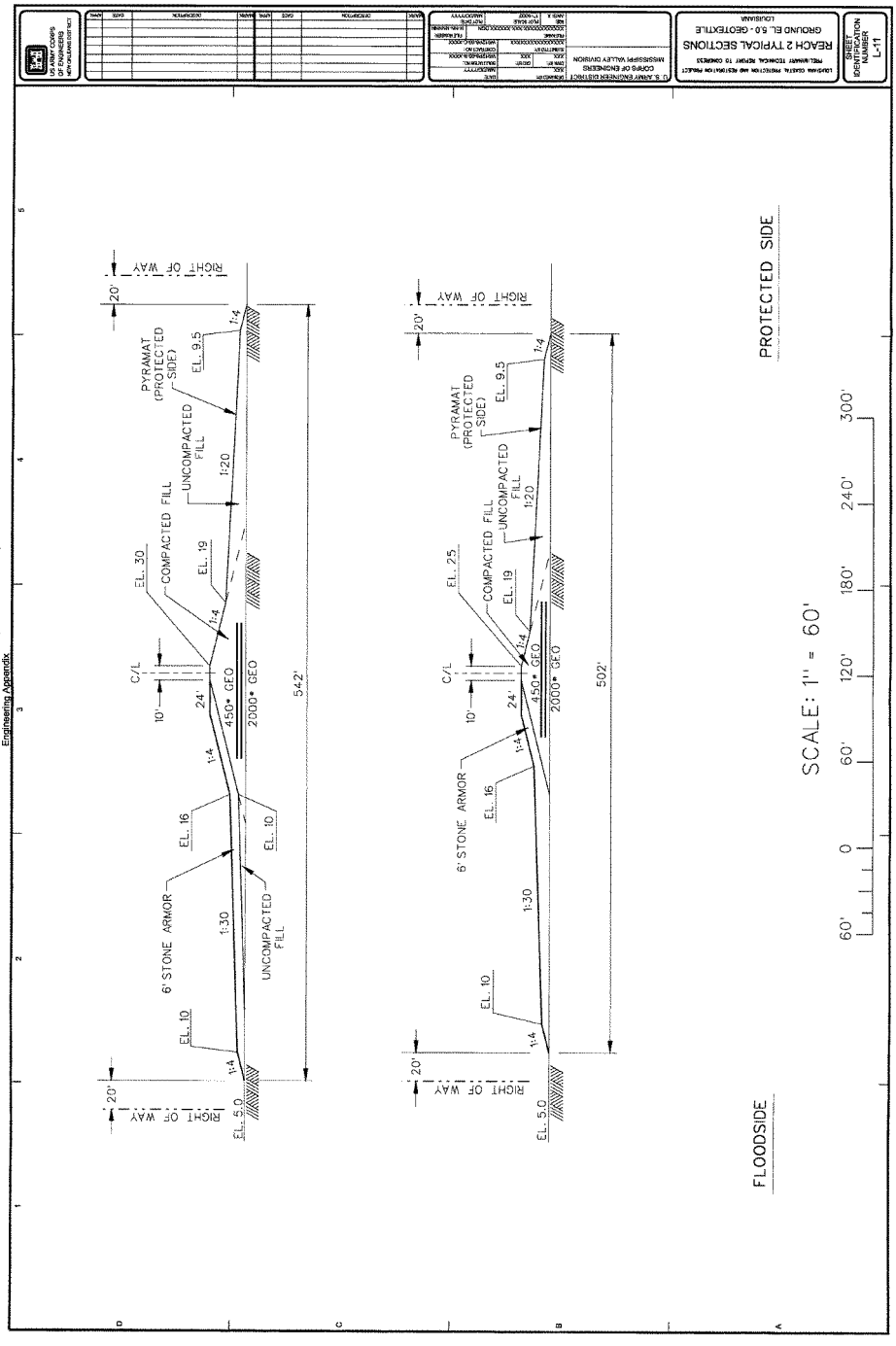
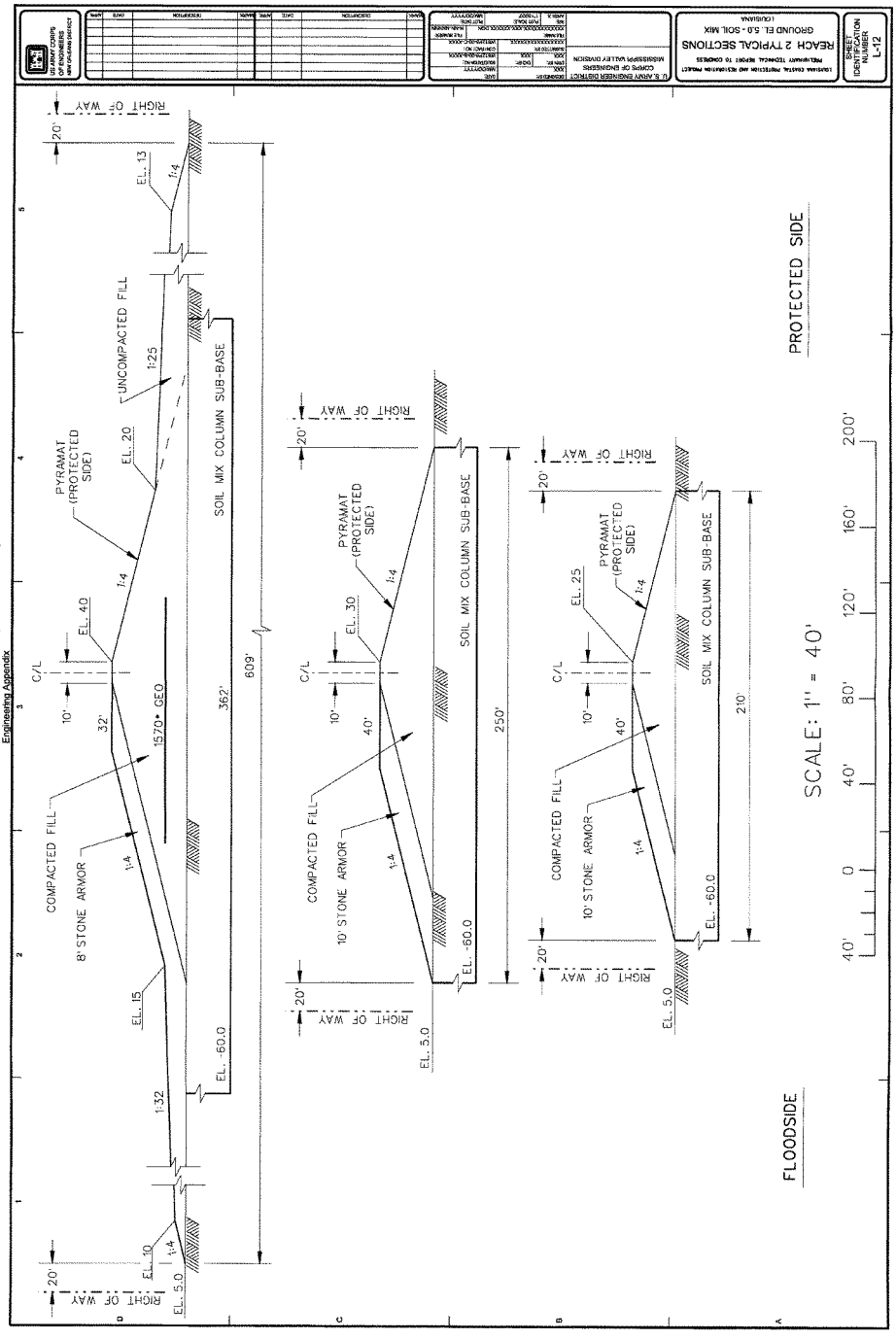
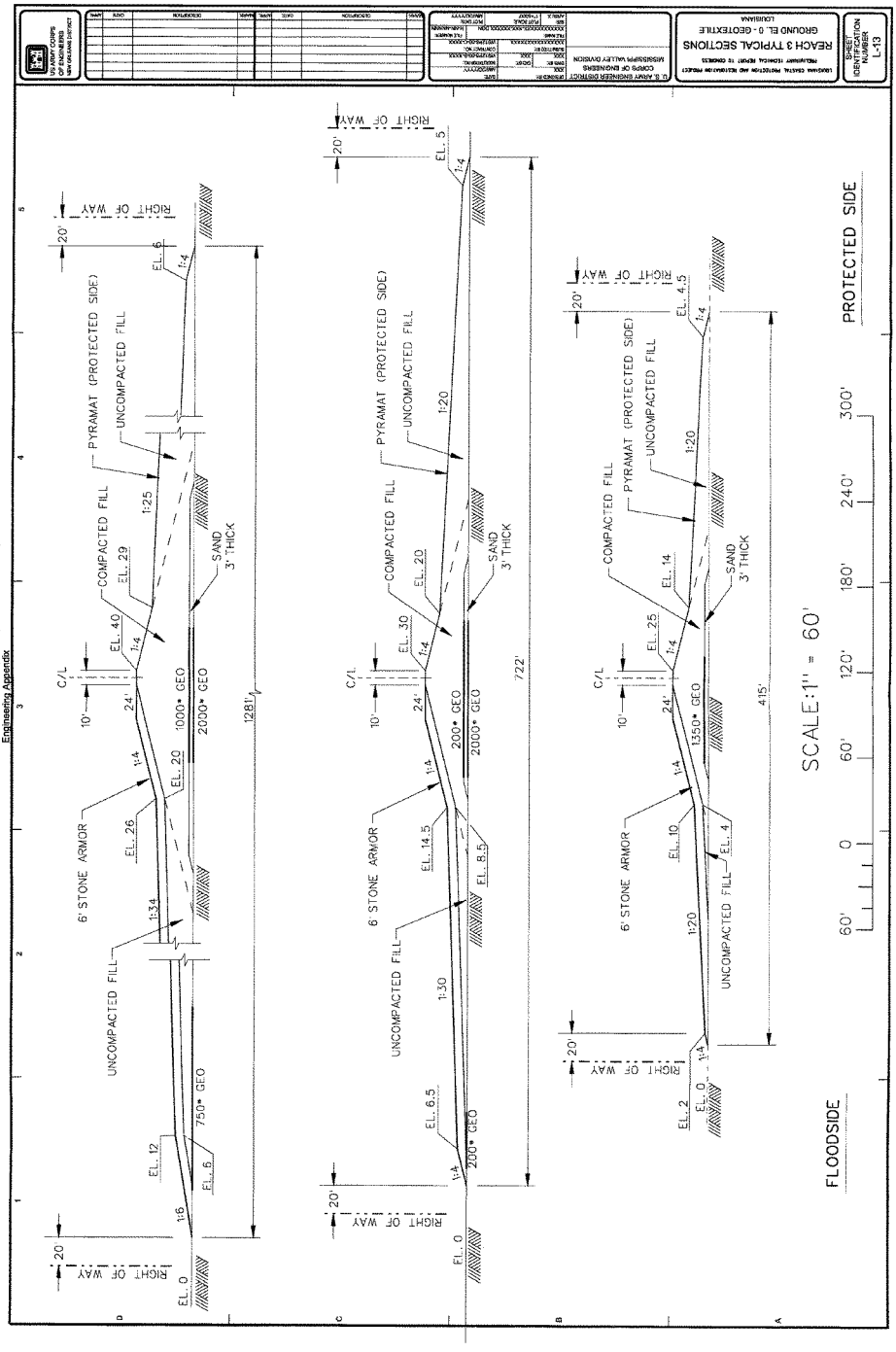

REACH 2 TYPICAL SECTIONS
GROUND EL. 5.0 - GEOTEXTILE
LOUISIANA
RIGHT OF WAY
20'
PYRAMAT (PROTECTED SIDE)
EL. 9.5
1:4
1:20
UNCOMPACTED FILL
COMPACTED FILL
EL. 30
EL. 19
C/L
10'
24'
450# GEO
2000# GEO
542'
6' STONE ARMOR
EL. 16
1:30
EL. 10
EL. 5.0
EL. 25
502'
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'


REACH 2 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-12
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
RIGHT OF WAY
20'
EL. 13
1:4
UNCOMPACTED FILL
1:25
EL. 20
PYRAMAT (PROTECTED SIDE)
1:4
SOIL MIX COLUMN SUB-BASE
C/L
EL. 40
10'
32'
1570# GEO
362'
609'
COMPACTED FILL
8' STONE ARMOR
1:4
EL. 15
EL. -60.0
1:32
1:4
EL. 10
EL. 5.0
RIGHT OF WAY
20'
PYRAMAT (PROTECTED SIDE)
EL. 30
1:4
SOIL MIX COLUMN SUB-BASE
C/L
10'
40'
COMPACTED FILL
10' STONE ARMOR
1:4
EL. -60.0
250'
RIGHT OF WAY
20'
EL. 5.0
RIGHT OF WAY
20'
EL. 25
PYRAMAT (PROTECTED SIDE)
1:4
C/L
10'
40'
COMPACTED FILL
10' STONE ARMOR
1:4
SOIL MIX COLUMN SUB-BASE
EL. -60.0
210'
RIGHT OF WAY
20'
EL. 5.0
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'



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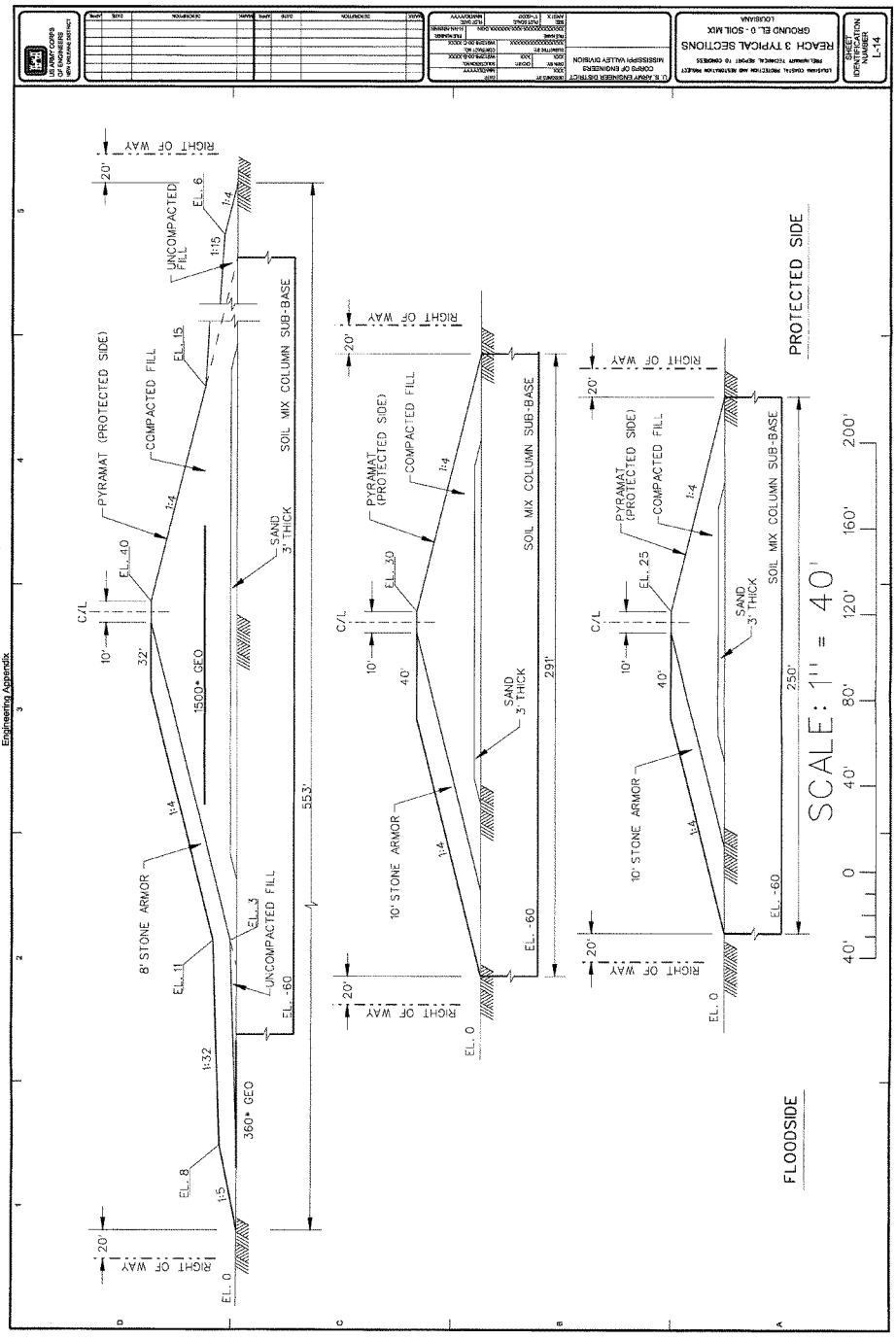

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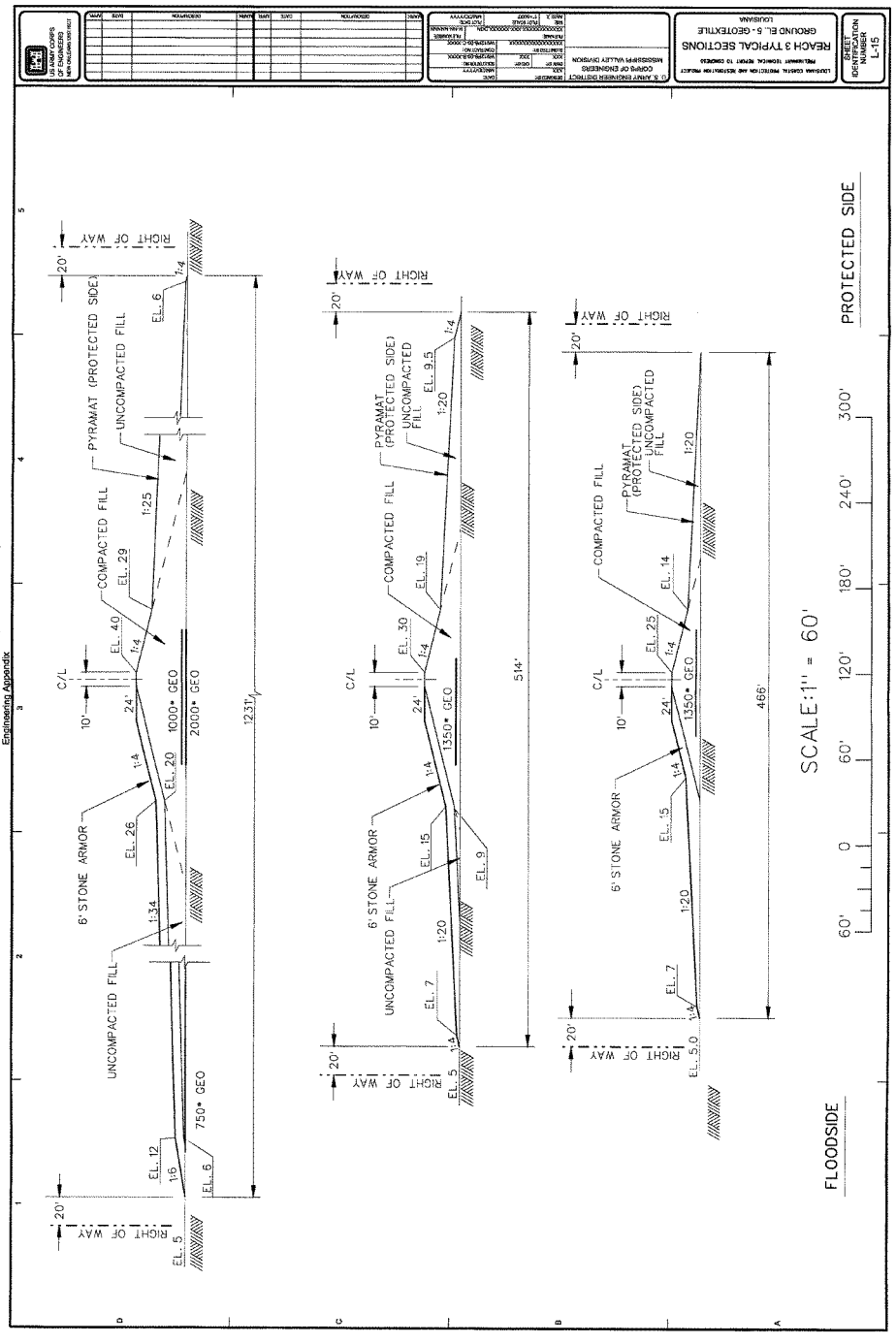
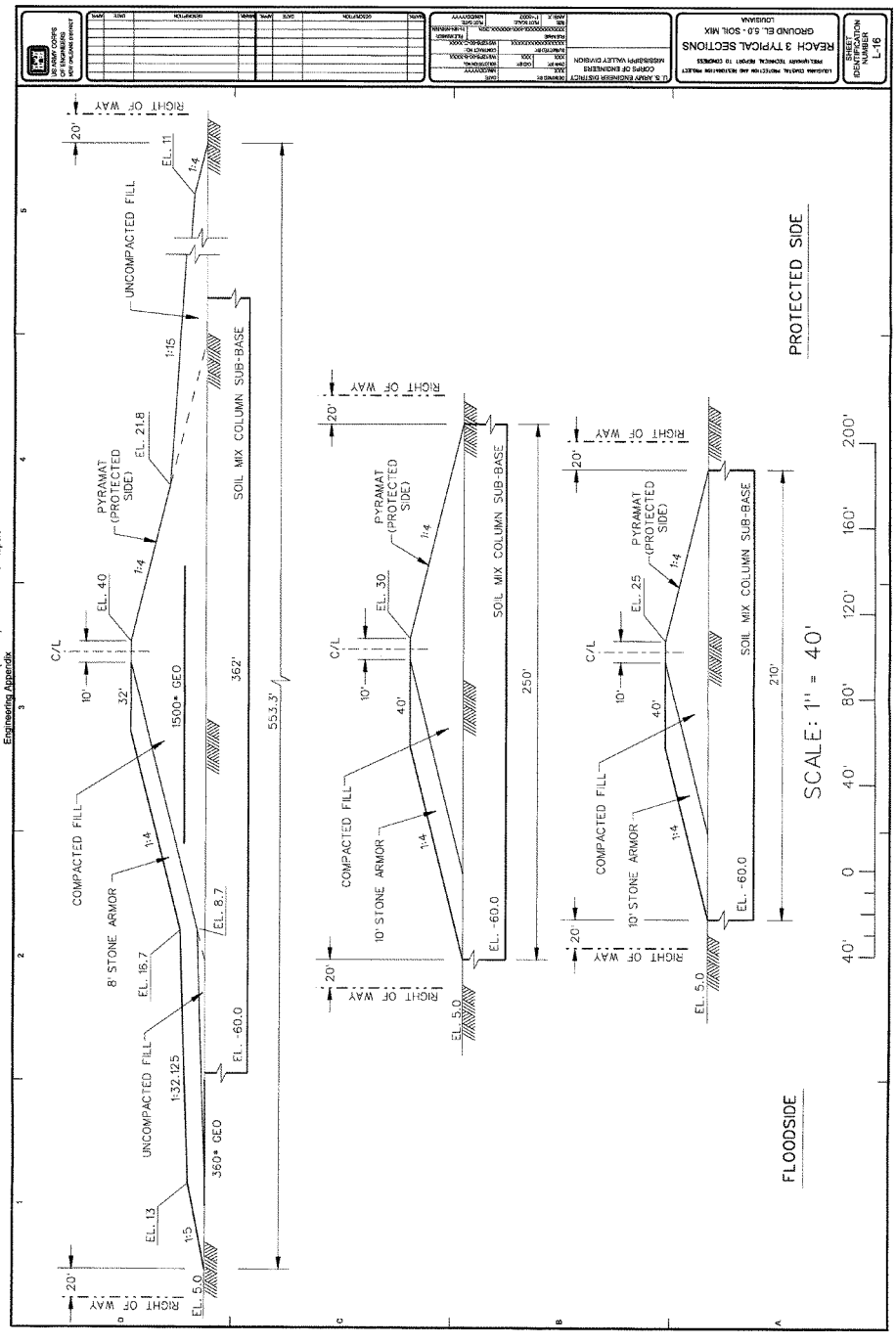
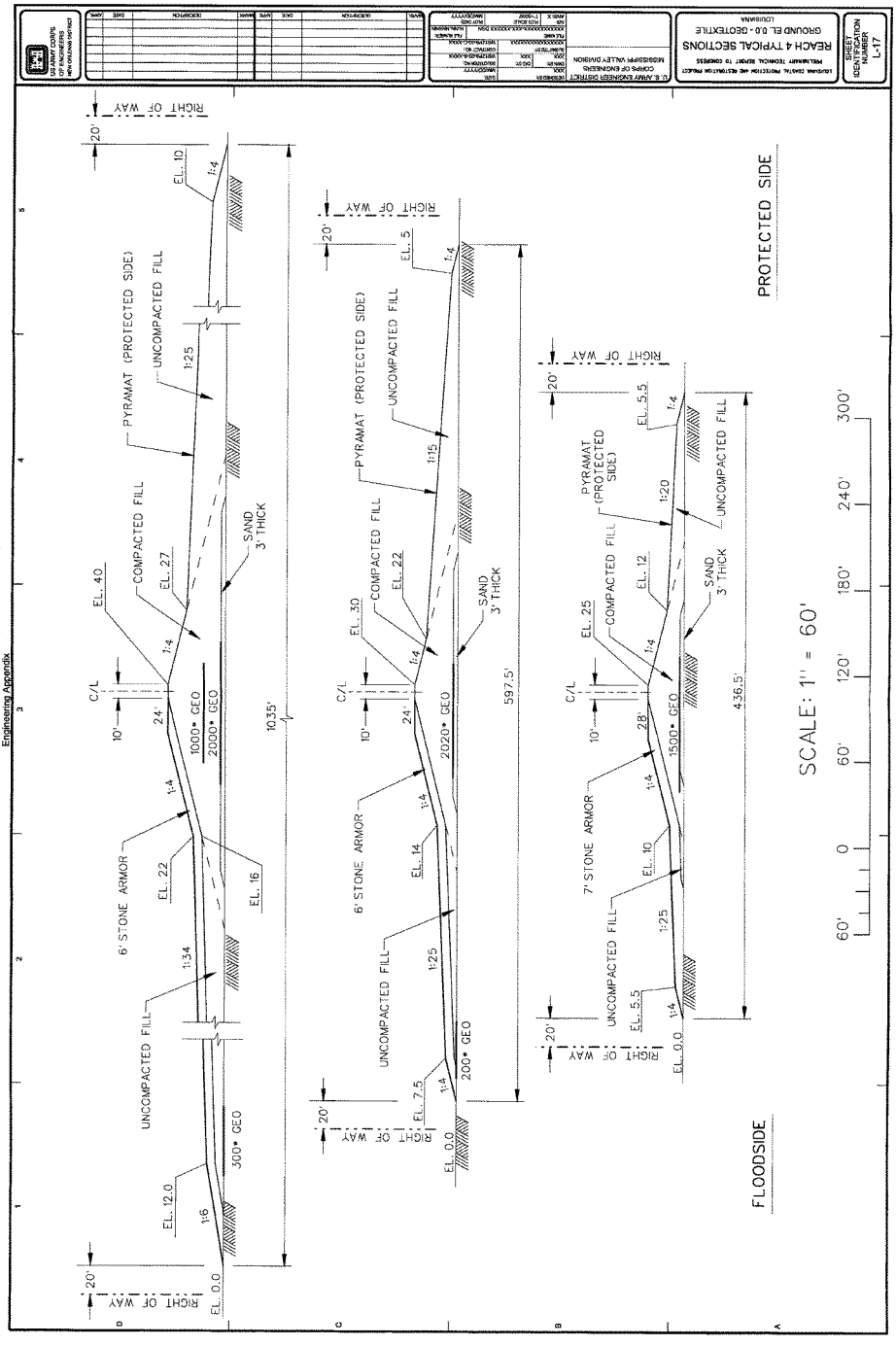
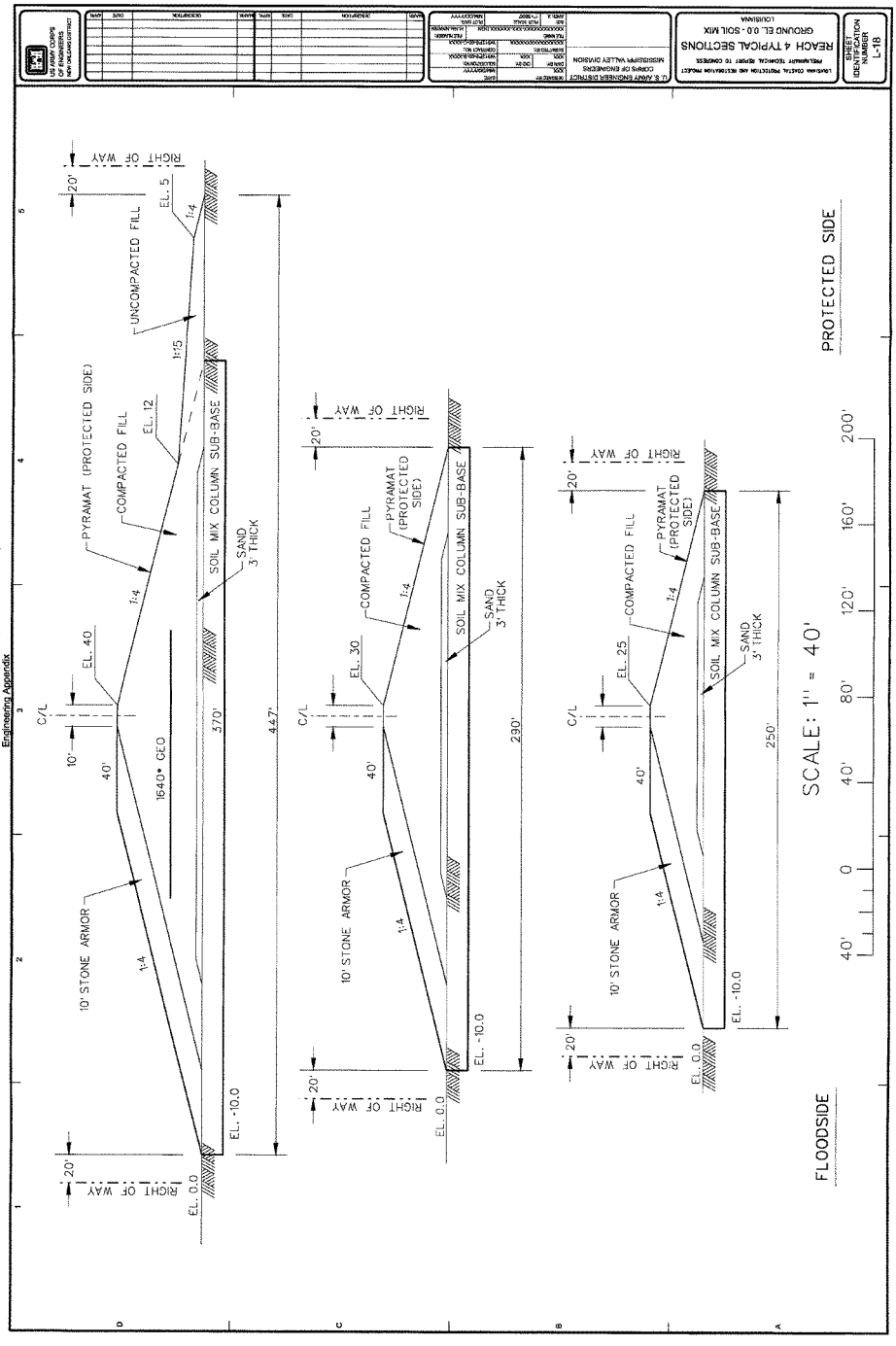
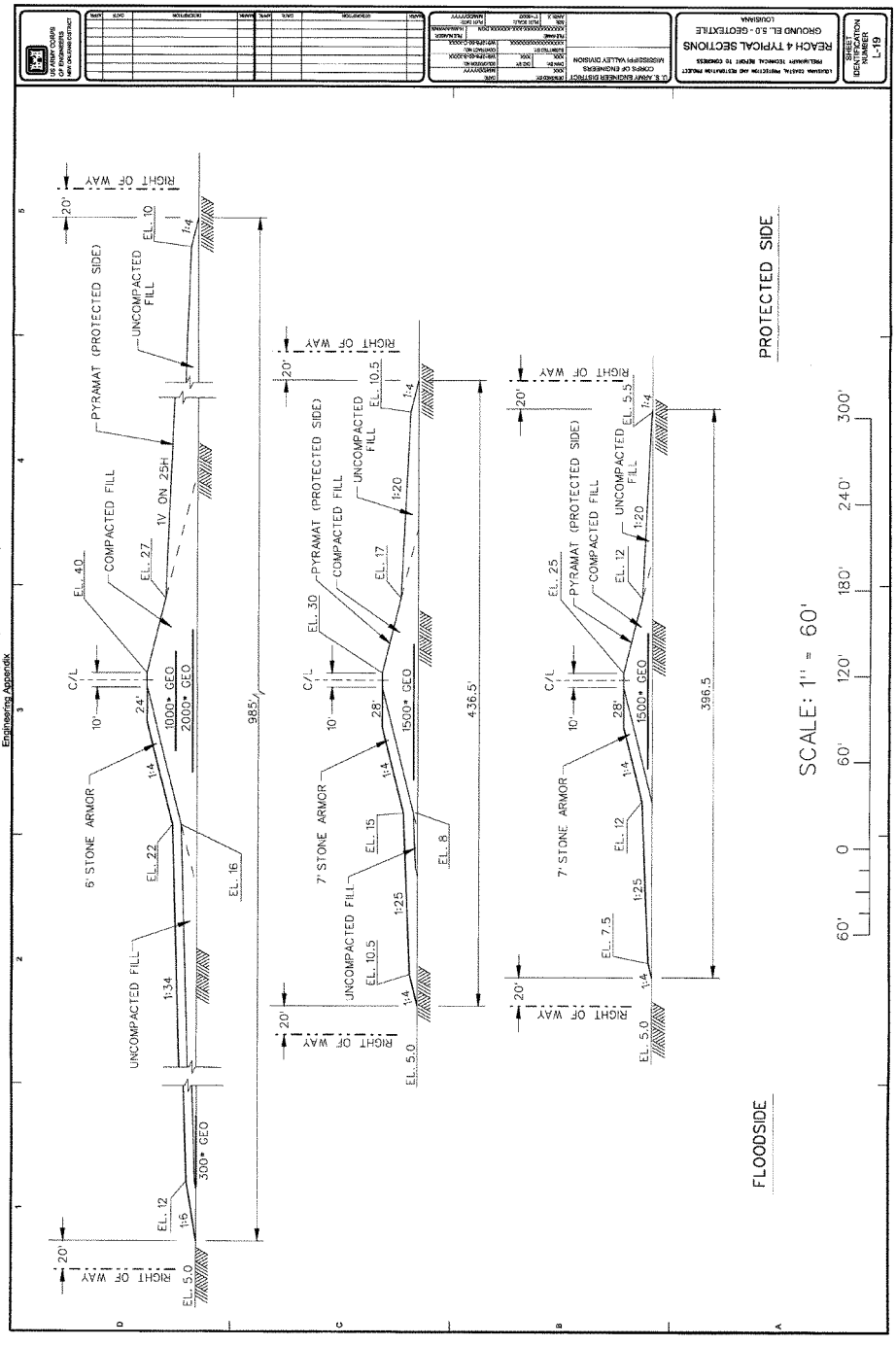
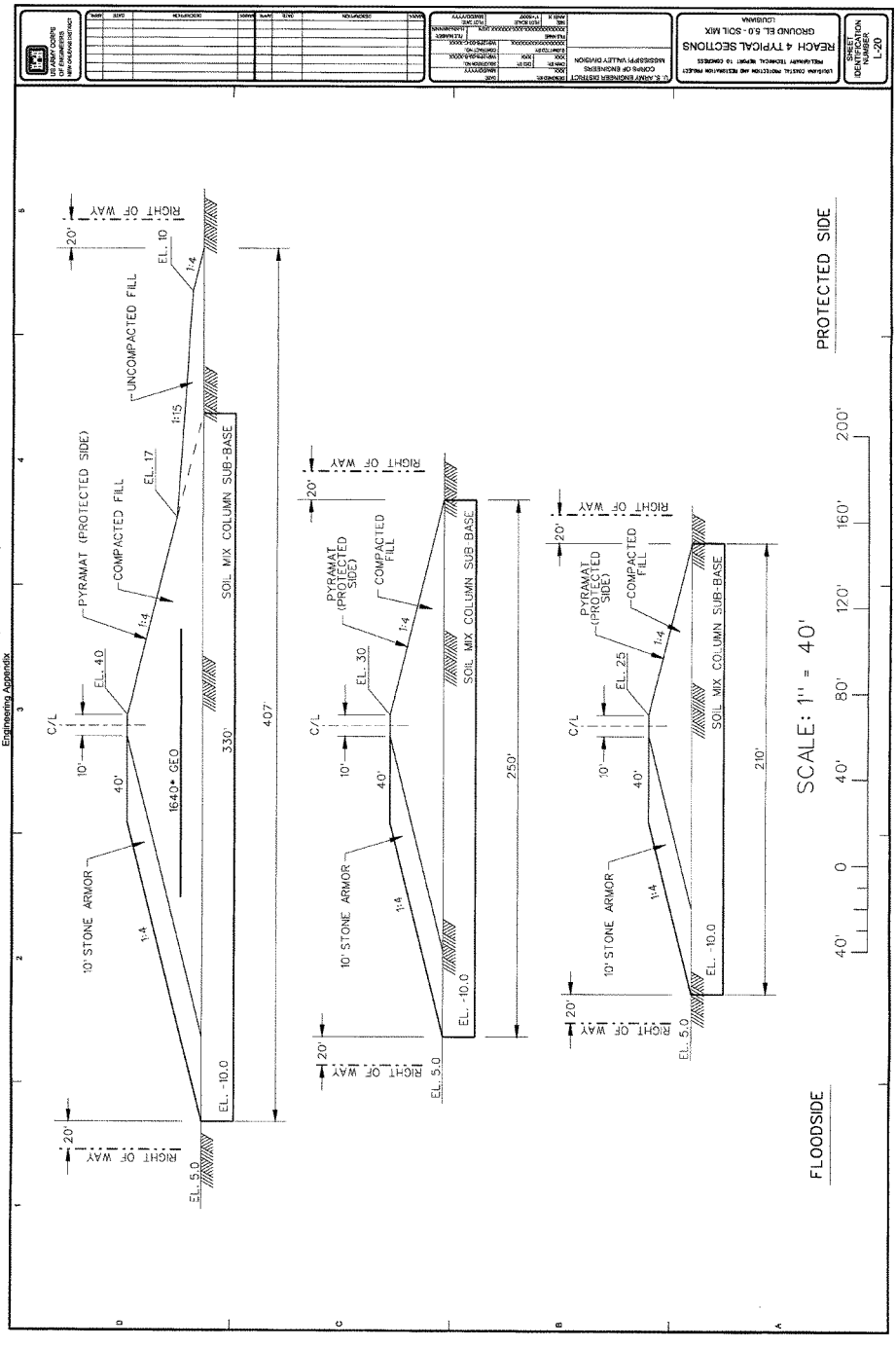

REACH 3 TYPICAL SECTIONS
GEOTEXTILE
SCALE: 1" = 60'
PROTECTED SIDE
FLOODSIDE
RIGHT OF WAY
20'
C/L
10'
24'
6' STONE ARMOR
UNCOMPACTED FILL
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
1000# GEO
2000# GEO
750# GEO
1350# GEO
1:4
1:20
1:25
1:34
1:6
EL. 40
EL. 29
EL. 26
EL. 20
EL. 19
EL. 15
EL. 14
EL. 12
EL. 9.5
EL. 9
EL. 7
EL. 6
EL. 5
EL. 5.0
1231'
514'
466'
60'
0
60'
120'
180'
240'
300'


REACH 3 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-16
RIGHT OF WAY
20'
EL. 11
1:4
UNCOMPACTED FILL
1:15
PYRAMAT (PROTECTED SIDE)
EL. 21.8
EL. 40
C/L
10'
32'
1500# GEO
362'
553.3'
SOIL MIX COLUMN SUB-BASE
COMPACTED FILL
8' STONE ARMOR
EL. 16.7
EL. 8.7
EL. -60.0
UNCOMPACTED FILL
1:32.125
360# GEO
EL. 13
1:5
EL. 5.0
FLOODSIDE
EL. 30
40'
250'
10' STONE ARMOR
EL. 25
210'
PROTECTED SIDE
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'


REACH 4 TYPICAL SECTIONS
GROUND EL. 0.0 - GEOTEXTILE
LOUISIANA
SHEET IDENTIFICATION NUMBER L-17
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 60'
RIGHT OF WAY
20'
EL. 10
1:4
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
1:25
COMPACTED FILL
EL. 27
EL. 40
SAND 3" THICK
C/L
10'
24'
1000# GEO
2000# GEO
10.35'
6' STONE ARMOR
EL. 22
EL. 16
1:34
UNCOMPACTED FILL
EL. 12.0
1:6
300# GEO
EL. 0.0
EL. 5
EL. 30
EL. 22
2020# GEO
597.5'
EL. 14
EL. 7.5
200# GEO
EL. 25
EL. 12
28'
1500# GEO
436.5'
EL. 5.5
1:20
7' STONE ARMOR
EL. 10
60' 0 60' 120' 180' 240' 300'


REACH 4 TYPICAL SECTIONS
GROUND EL. 0.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-18
RIGHT OF WAY
20'
EL. 5
1:4
UNCOMPACTED FILL
1:5
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 12
1:4
SOIL MIX COLUMN SUB-BASE
SAND 3' THICK
C/L
EL. 40
10'
40'
1640# GEO
370'
447'
10' STONE ARMOR
1:4
EL. -10.0
EL. 0.0
EL. 30
290'
EL. 25
250'
FLOODSIDE
PROTECTED SIDE
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'


REACH 4 TYPICAL SECTIONS
GROUND EL. 5.0 - GEOTEXTILE
LOUISIANA
SHEET IDENTIFICATION NUMBER L-19
RIGHT OF WAY
20'
EL. 40
COMPACTED FILL
EL. 27
1V ON 25H
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
EL. 10
1:4
C/L
10'
24'
1000# GEO
2000# GEO
985'
6' STONE ARMOR
EL. 22
1:4
EL. 16
UNCOMPACTED FILL
1:34
300# GEO
EL. 12
1:6
EL. 5.0
EL. 30
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
EL. 17
UNCOMPACTED FILL
EL. 10.5
1:20
1:4
28'
1500# GEO
436.5'
7' STONE ARMOR
EL. 15
EL. 8
UNCOMPACTED FILL
EL. 10.5
1:25
EL. 25
EL. 12
EL. 5.5
396.5'
EL. 7.5
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'
PROTECTED SIDE
FLOODSIDE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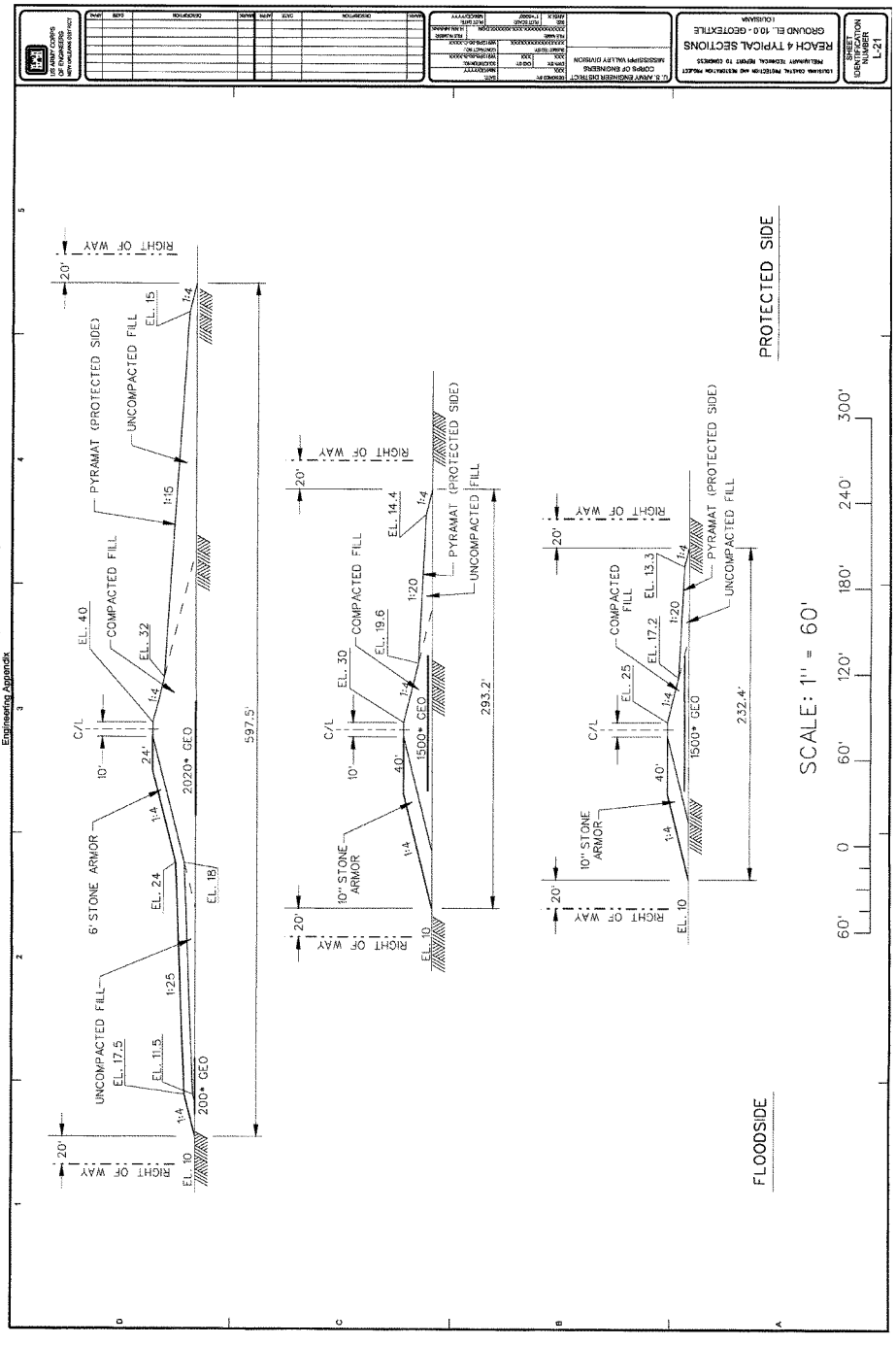
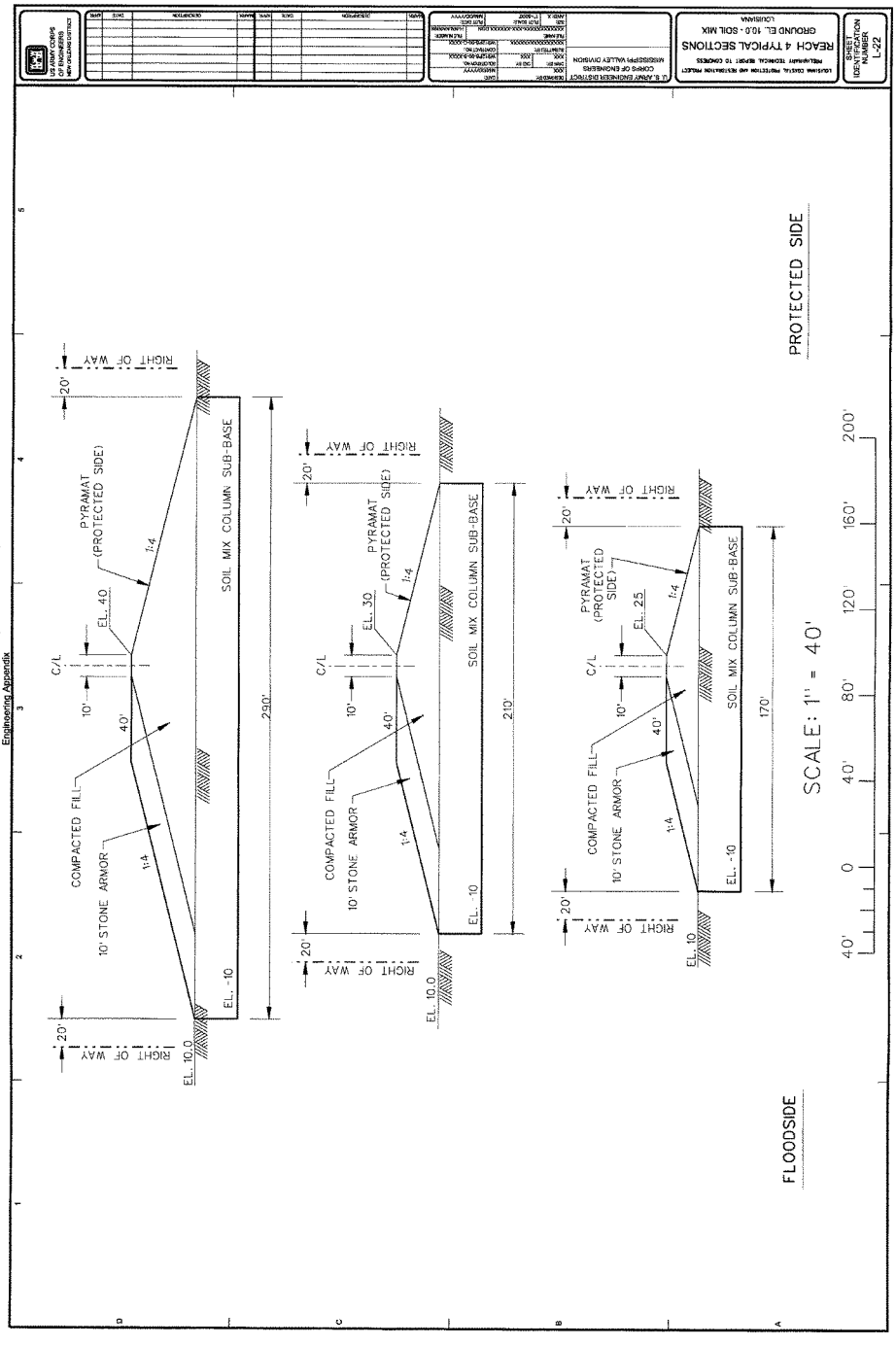
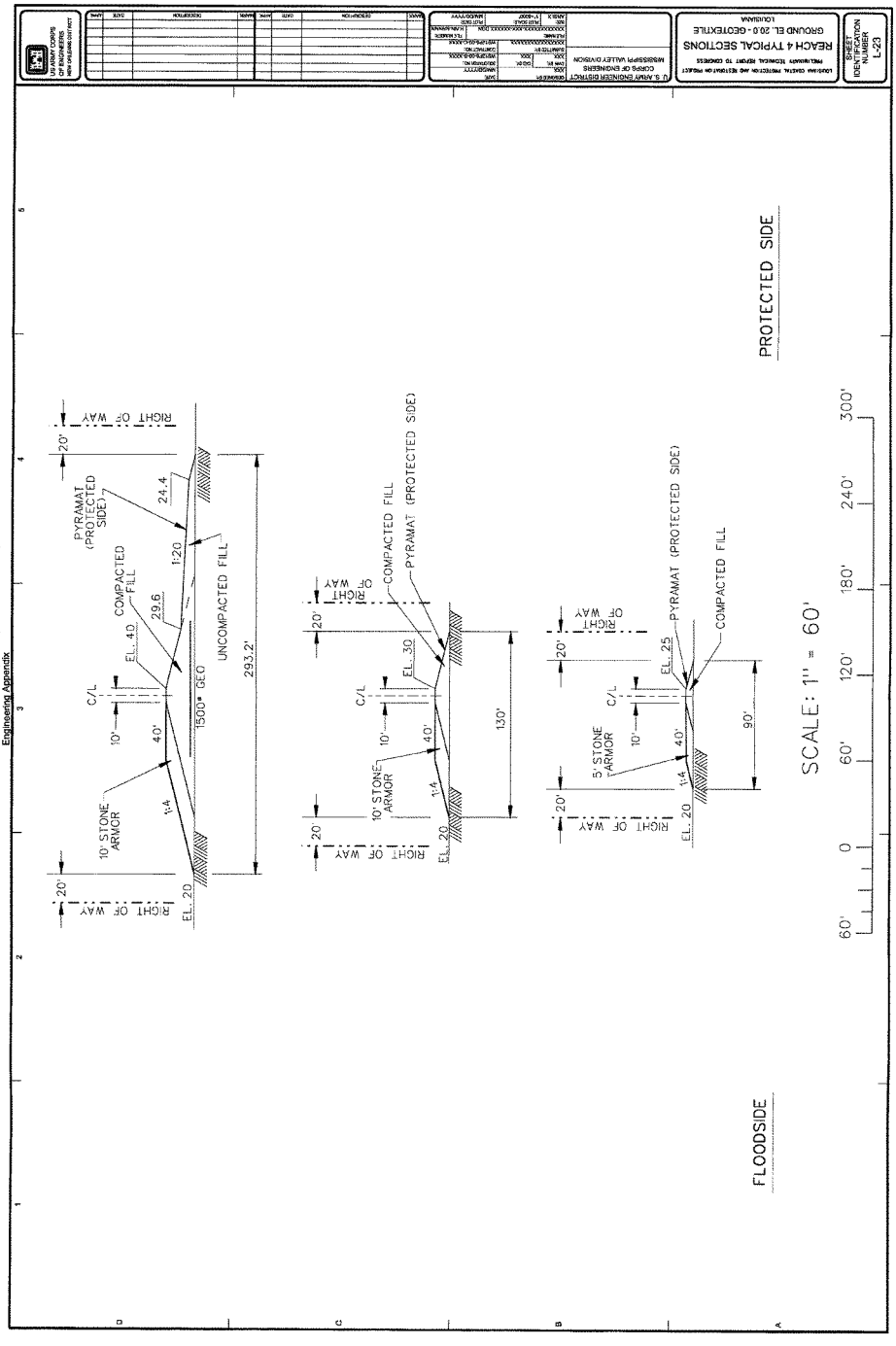
REACH 4 TYPICAL SECTIONS
GROUND EL. 5.0 - SOL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-20
SCALE: 1" = 40'
40' 0 40' 80' 120' 160' 200'
FLOODSIDE
PROTECTED SIDE
10' STONE ARMOR
1:4
40'
C/L
10'
EL. 40
PYRAMAT (PROTECTED SIDE)
COMPACTED FILL
EL. 17
1:4
UNCOMPACTED FILL
1:15
EL. 10
1:4
SOIL MIX COLUMN SUB-BASE
330'
407'
1640 • GEO
EL. -10.0
EL. 5.0
20'
RIGHT OF WAY
EL. 30
EL. 25
250'
210'


REACH 4 TYPICAL SECTIONS
GROUND EL. 10.0 - GEOTEXTILE
LOUISIANA
SHEET IDENTIFICATION NUMBER L-21
RIGHT OF WAY
20'
EL. 10
EL. 17.5
EL. 11.5
UNCOMPACTED FILL
6' STONE ARMOR
1:4
1:25
EL. 24
EL. 18
200# GEO
2020# GEO
597.5'
EL. 40
COMPACTED FILL
EL. 32
24'
C/L
10'
PYRAMAT (PROTECTED SIDE)
UNCOMPACTED FILL
EL. 15
1:15
FLOODSIDE
EL. 30
EL. 19.6
40'
1500# GEO
10" STONE ARMOR
1:20
EL. 14.4
293.2'
EL. 25
EL. 17.2
EL. 13.3
232.4'
PROTECTED SIDE
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'


REACH 4 TYPICAL SECTIONS
GROUND EL. 10.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-22
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
RIGHT OF WAY
20'
PYRAMAT (PROTECTED SIDE)
1:4
EL. 40
C/L
10'
40'
COMPACTED FILL
10' STONE ARMOR
1:4
SOIL MIX COLUMN SUB-BASE
290'
EL. -10
EL. 10.0
EL. 30
210'
EL. 25
170'
EL. 10
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 40'
40'
0
40'
80'
120'
160'
200'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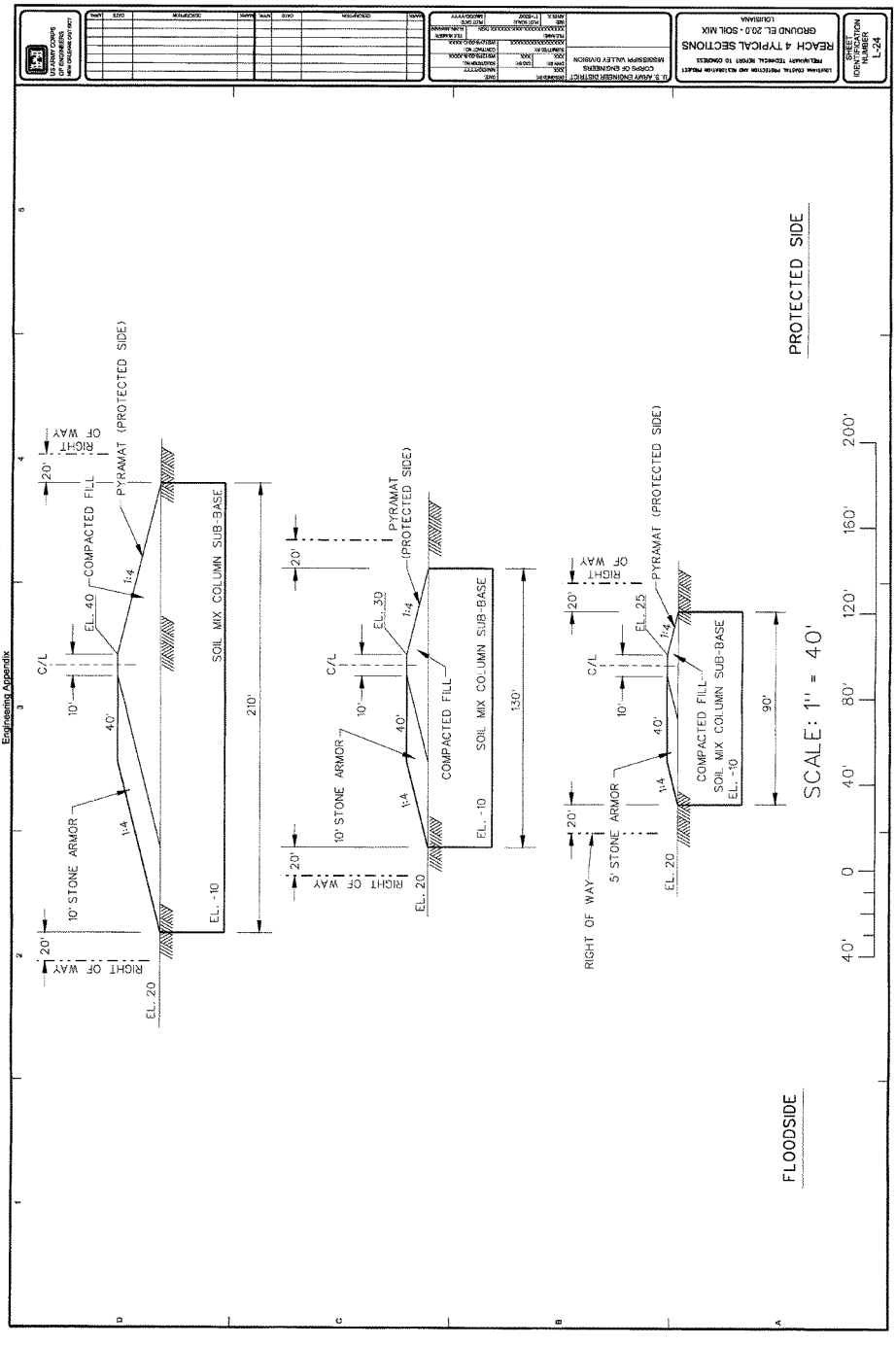
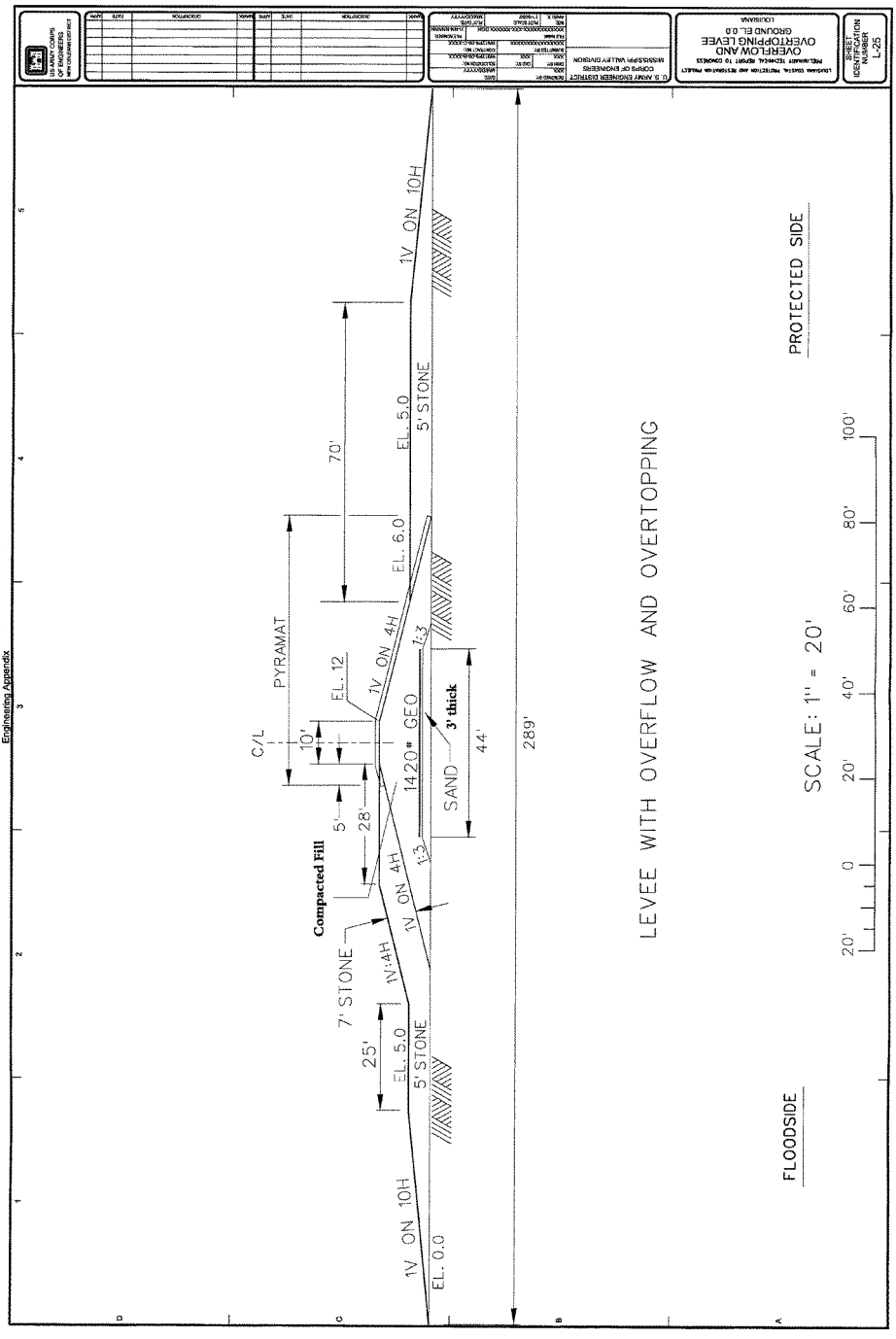
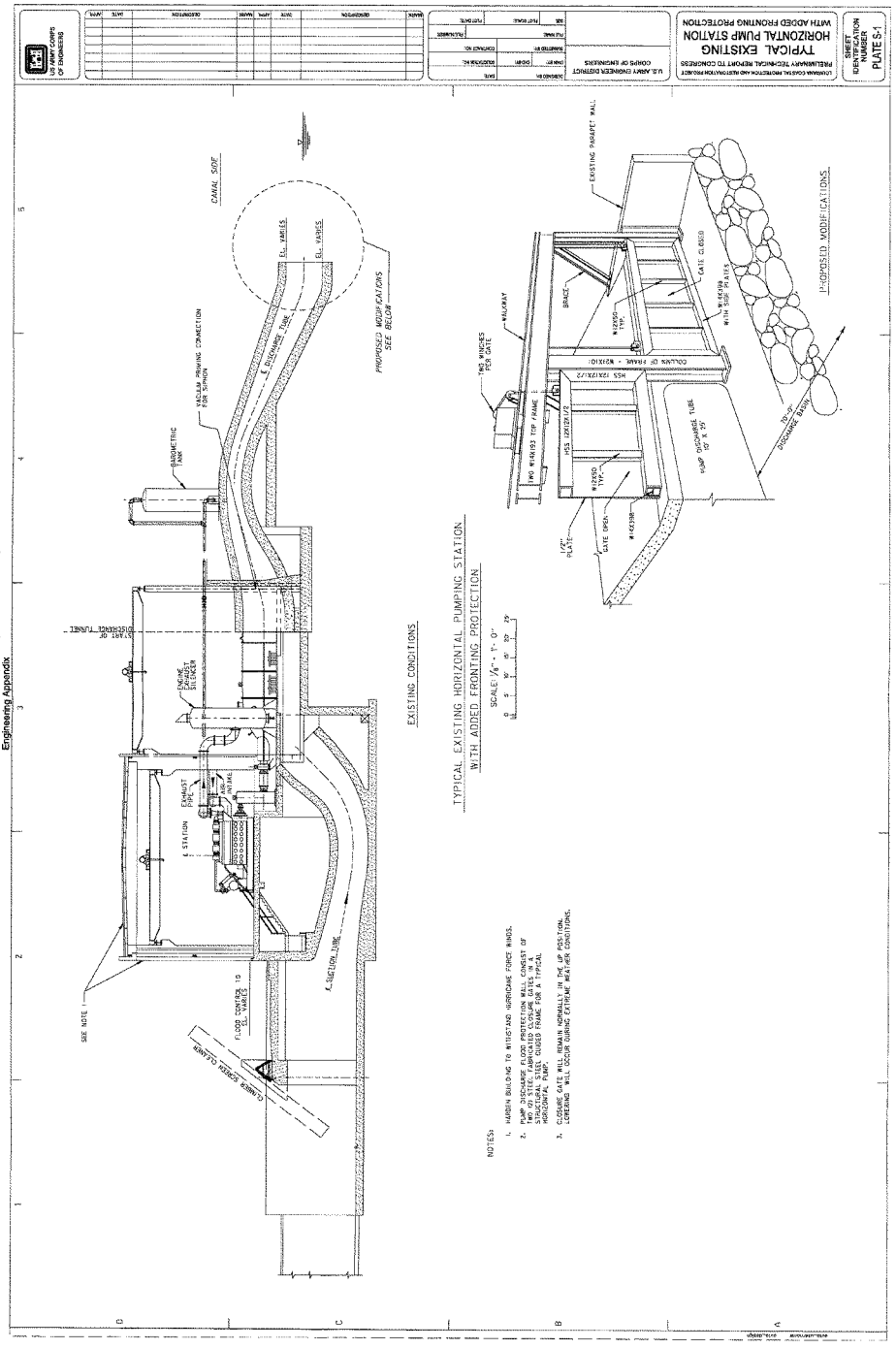
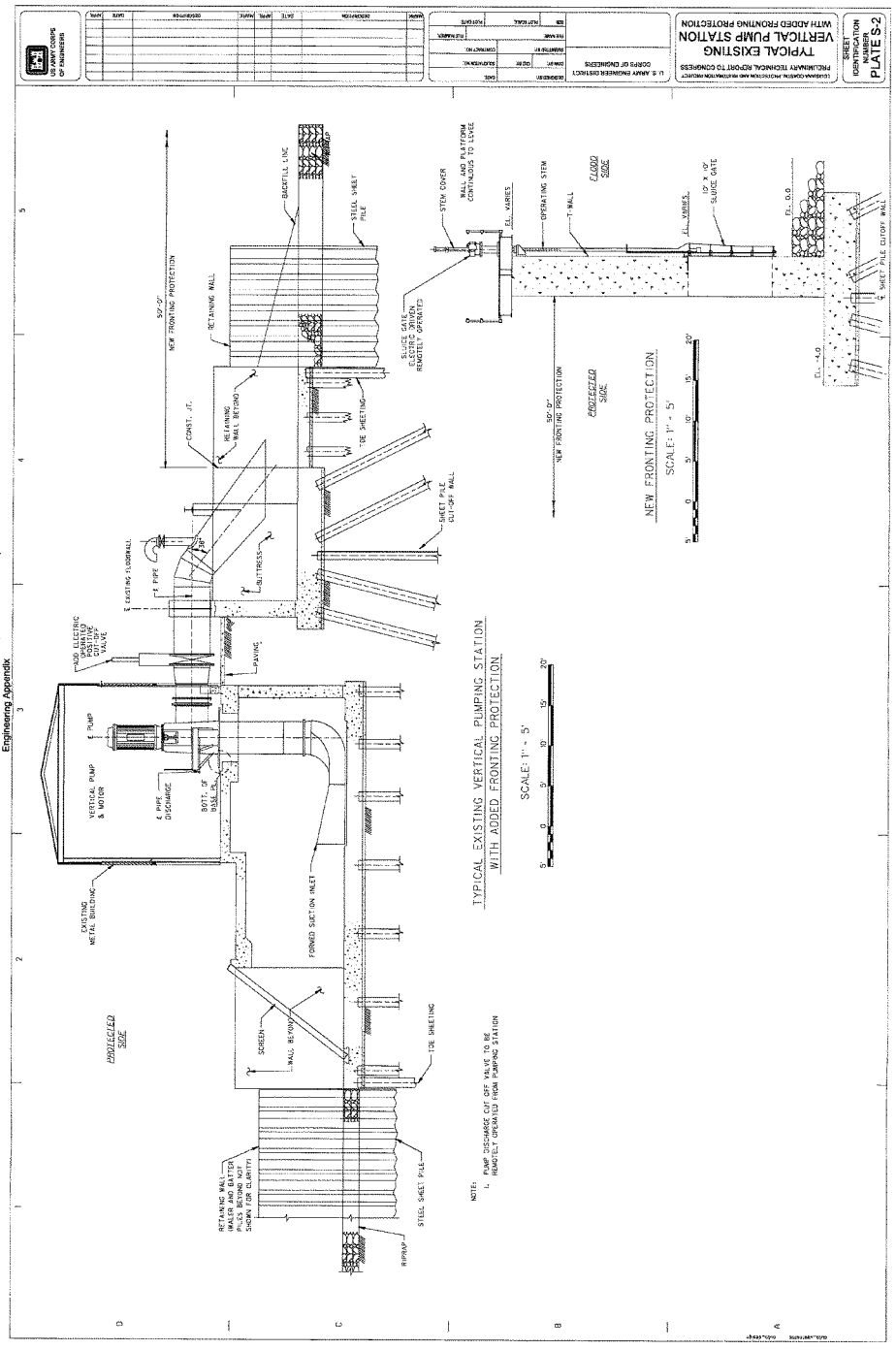
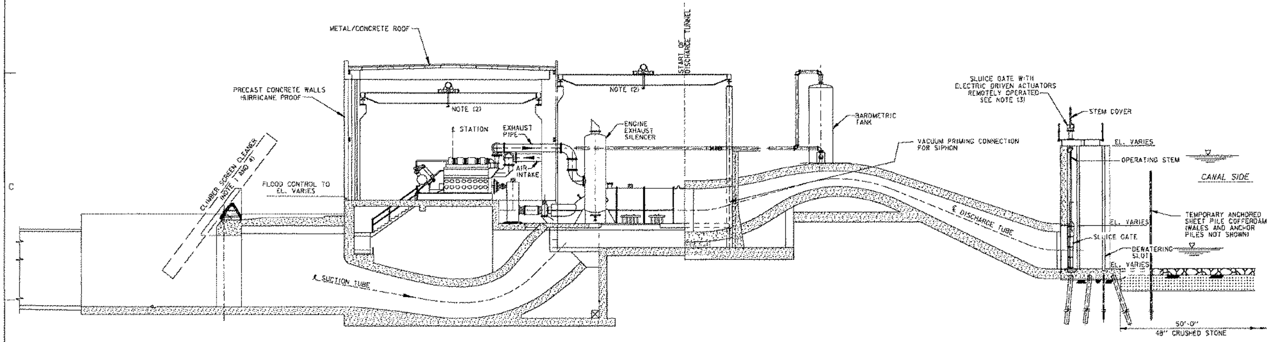
REACH 4 TYPICAL SECTIONS
GROUND EL. 20.0 - GEOTEXTILE
LOUISIANA
SHEET IDENTIFICATION NUMBER L-23
RIGHT OF WAY
20'
PYRAMAT (PROTECTED SIDE)
24.4
1:20
COMPACTED FILL
EL. 40
29.6
C/L
10'
40'
1500# GEO
UNCOMPACTED FILL
293.2'
10' STONE ARMOR
1:4
EL. 20
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
EL. 30
130'
10' STONE ARMOR
EL. 25
90'
5' STONE ARMOR
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 60'
60' 0 60' 120' 180' 240' 300'


REACH 4 TYPICAL SECTIONS
GROUND EL. 20.0 - SOIL MIX
LOUISIANA
SHEET IDENTIFICATION NUMBER L-24
PROTECTED SIDE
FLOODSIDE
SCALE: 1" = 40'
RIGHT OF WAY
10' STONE ARMOR
5' STONE ARMOR
COMPACTED FILL
PYRAMAT (PROTECTED SIDE)
SOIL MIX COLUMN SUB-BASE
EL. -10
EL. 20
EL. 40
EL. 30
EL. 25
1:4
C/L
20'
10'
40'
210'
130'
90'
0
40'
80'
120'
160'
200'


LEVEE WITH OVERFLOW AND OVERTOPPING
SCALE: 1" = 20'
FLOODSIDE
PROTECTED SIDE
C/L
PYRAMAT
10'
EL. 12
5'
28'
Compacted Fill
7' STONE
1V:4H
25'
EL. 5.0
5' STONE
1V ON 10H
EL. 0.0
1V ON 4H
1:3
1420# GEO
1:3
1V ON 4H
EL. 6.0
70'
EL. 5.0
5' STONE
1V ON 10H
SAND
3' thick
44'
289'
20' 0 20' 40' 60' 80' 100'
OVERFLOW AND OVERTOPPING LEVEE
GROUND EL 0.0
LOUISIANA
SHEET IDENTIFICATION NUMBER L-25


TYPICAL EXISTING HORIZONTAL PUMPING STATION
WITH ADDED FRONTING PROTECTION
SCALE: 1/8" = 1'-0"
EXISTING CONDITIONS
PROPOSED MODIFICATIONS SEE BELOW
PROPOSED MODIFICATIONS
NOTES:
1. HARDEN BUILDING TO WITHSTAND HURRICANE FORCE WINDS.
2. PUMP DISCHARGE FLOOR PROTECTION WILL CONSIST OF TWO (2) STEEL FABRICATED CLOSURE GATES, IN A VERTICAL SLIDING GUIDED FRAME FOR A TYPICAL HORIZONTAL PUMP.
3. CLOSURE GATE WILL REMAIN NORMALLY IN THE UP POSITION. LOWERING WILL OCCUR DURING EXTREME WEATHER CONDITIONS.
TYPICAL EXISTING HORIZONTAL PUMP STATION WITH ADDED FRONTING PROTECTION
SHEET IDENTIFICATION NUMBER PLATE E-1


TYPICAL EXISTING VERTICAL PUMPING STATION
WITH ADDED FRONTING PROTECTION
SCALE: 1" = 5'
NEW FRONTING PROTECTION
SCALE: 1" = 5'
NOTE:
1. PUMP DISCHARGE CUT OFF VALVE TO BE REMOTELY OPERATED FROM PUMPING STATION
PROTECTED SIDE
FLOOD SIDE
EXISTING METAL BUILDING
VERTICAL PUMP & MOTOR
FORMED SUCTION INLET
SCREEN
WALL BEYOND
TOE SHEETING
STEEL SHEET PILE
RIPRAP
RETAINING WALL (WALER AND BATTER PILES BEYOND NOT SHOWN FOR CLARITY)
BACKFILL LINE
RETAINING WALL
CONST. JT.
SHEET PILE CUT-OFF WALL
BUTTRESS
PAVING
EXISTING FLOODWALL
STEM COVER
WALL AND PLATFORM CONTINUOUS TO LEVEE
OPERATING STEM
T WALL
SLUICE GATE CLOSED AND REMOTELY OPERATED
10' X 10' SLUICE GATE
50'-0" NEW FRONTING PROTECTION
EL. VARIES
EL. -4.0
SHEET PILE CUTOFF WALL
VERTICAL PUMP STATION
WITH ADDED FRONTING PROTECTION
SHEET IDENTIFICATION NUMBER
PLATE S-2



TYPICAL NEW HORIZONTAL PUMPING STATION
WITH FRONTING PROTECTION

SCALE: 1/8" = 1'-0"

NOTES:

1. SIZE CLIMBER SCREEN CLEANER IN ACCORDANCE WITH DESIGN PUMPING CAPACITY.
2. OVERHEAD CRANE OF SUFFICIENT CAPACITY TO HANDLE LIFTING AND SETTING MAJOR EQUIPMENT.
3. SLUICE GATE ACTUATORS WILL BE ELECTRIC DRIVEN REMOTELY ACTIVATED FROM THE PUMPING STATION CONTROL ROOM.
4. FURNISH SCREEN CLEANERS WITH REMOTE CONTROLS TO ALLOW OPERATION FROM THE PUMPING STATION CONTROL ROOM AND LOCALLY AS WELL.
5. SLUICE GATES WILL BE RUN BETWEEN STEEL RAIL GUIDES IMBEDDED IN CONCRETE WALLS.

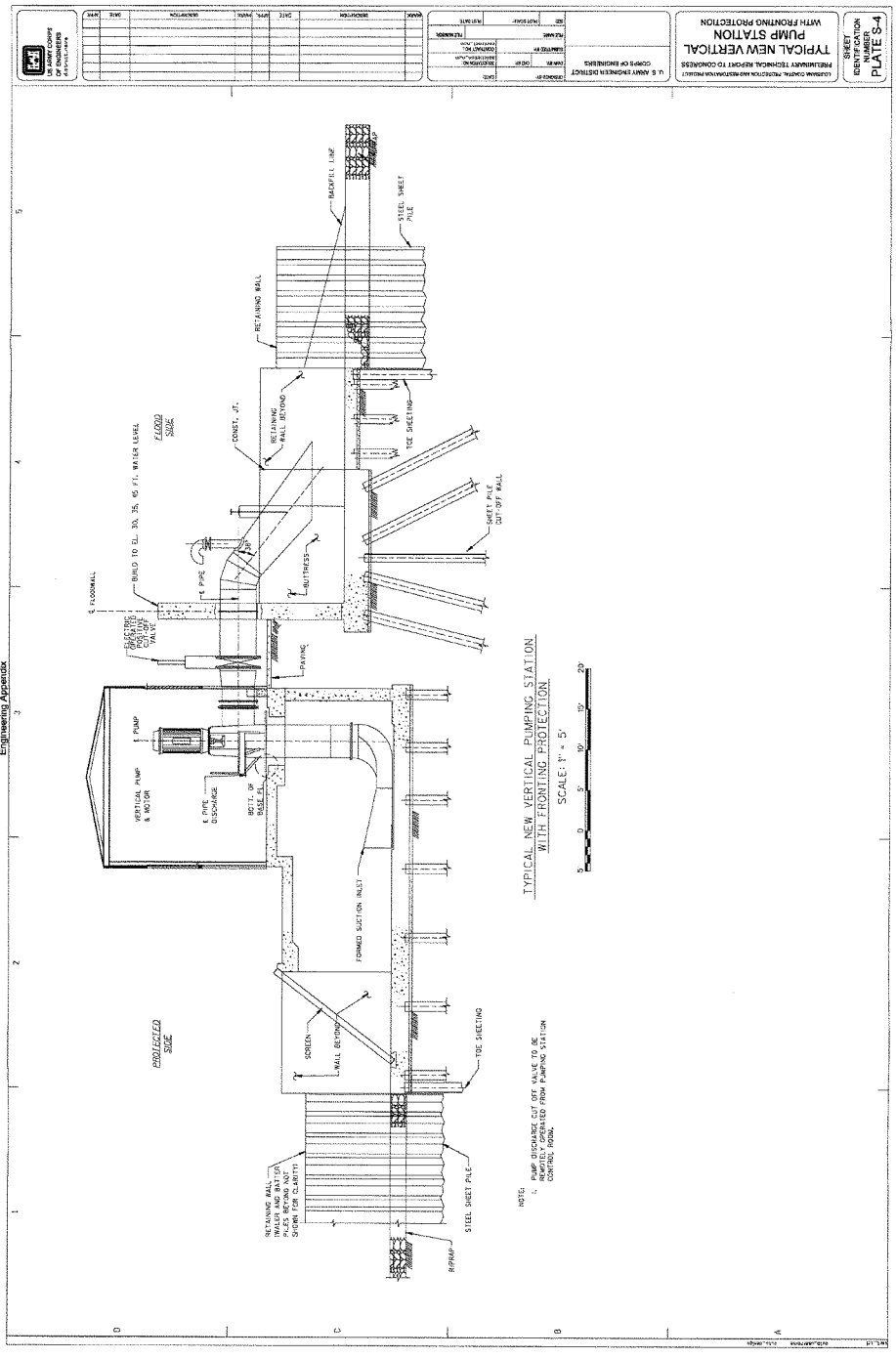
US ARMY CORPS OF ENGINEERS

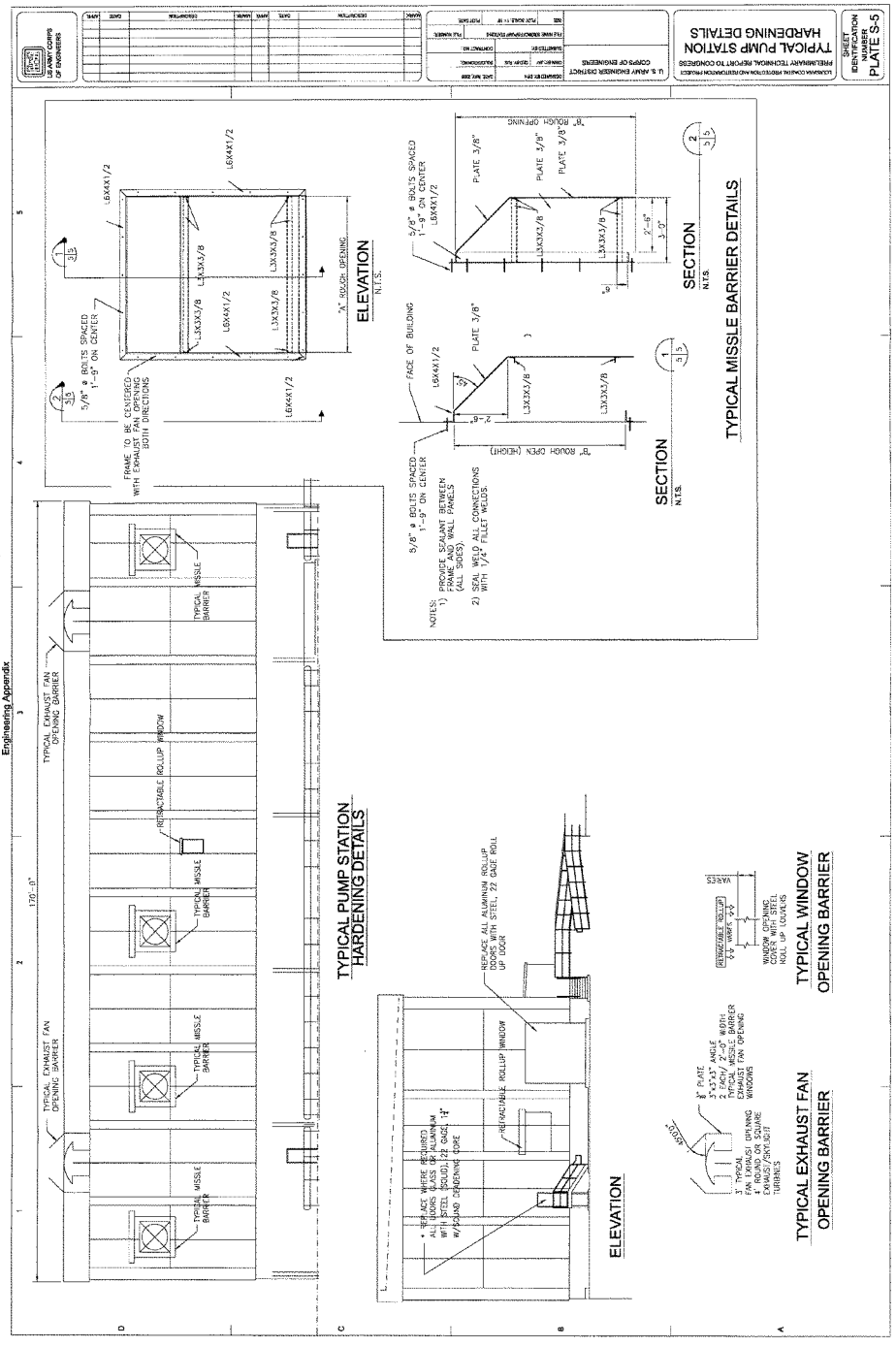
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

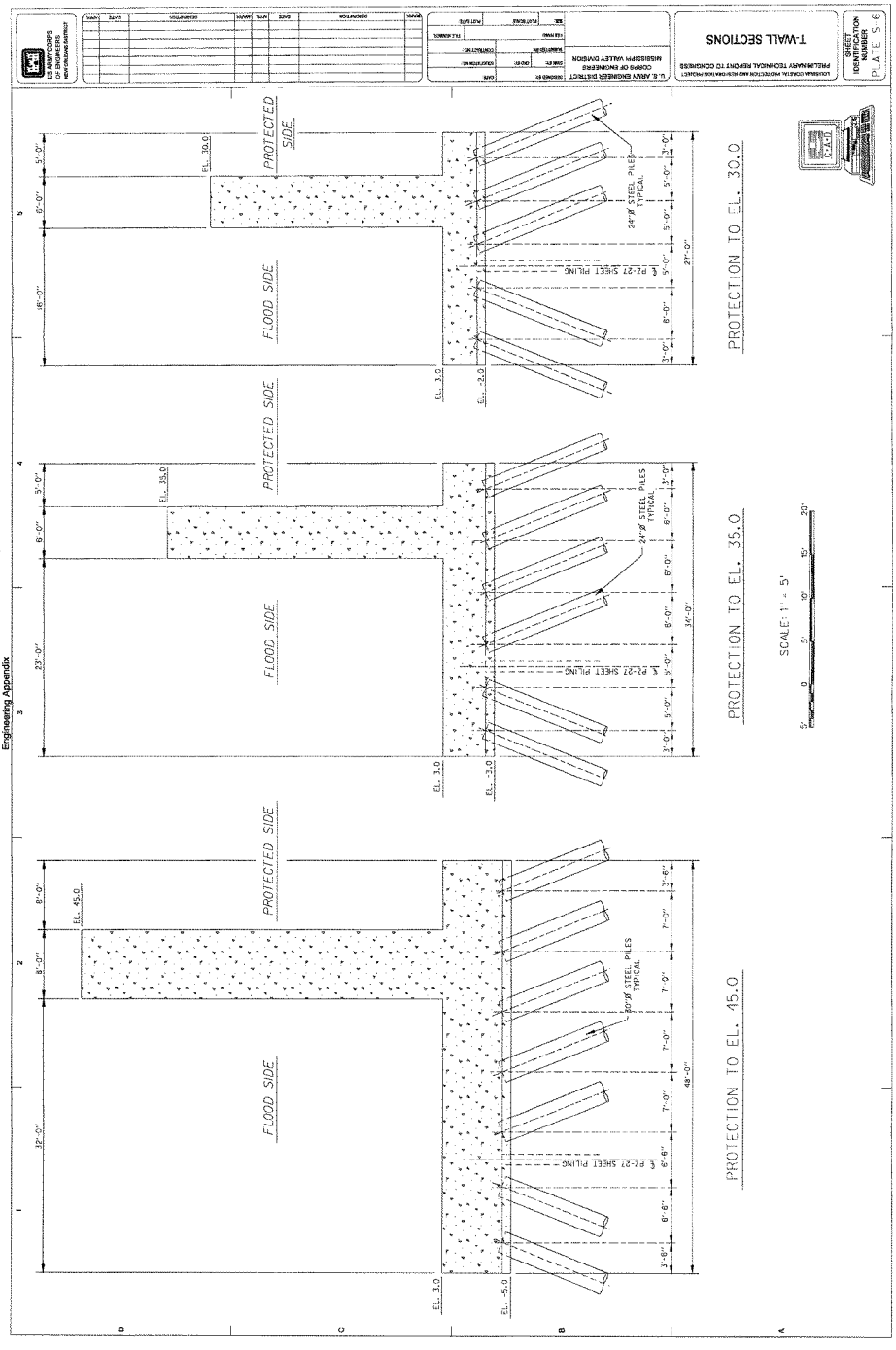
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

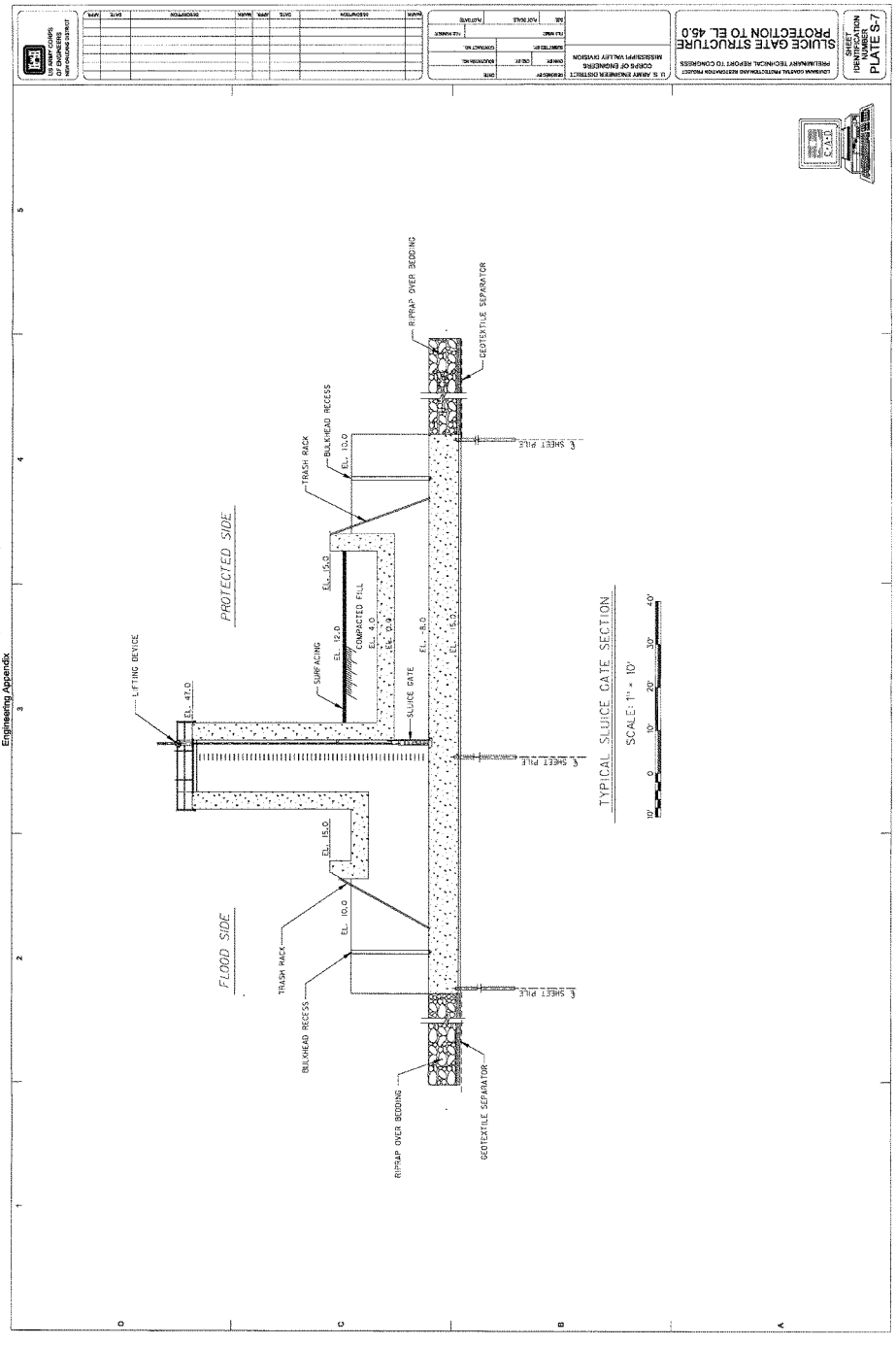
TYPICAL NEW HORIZONTAL
PUMP STATION
WITH FRONTING PROTECTION

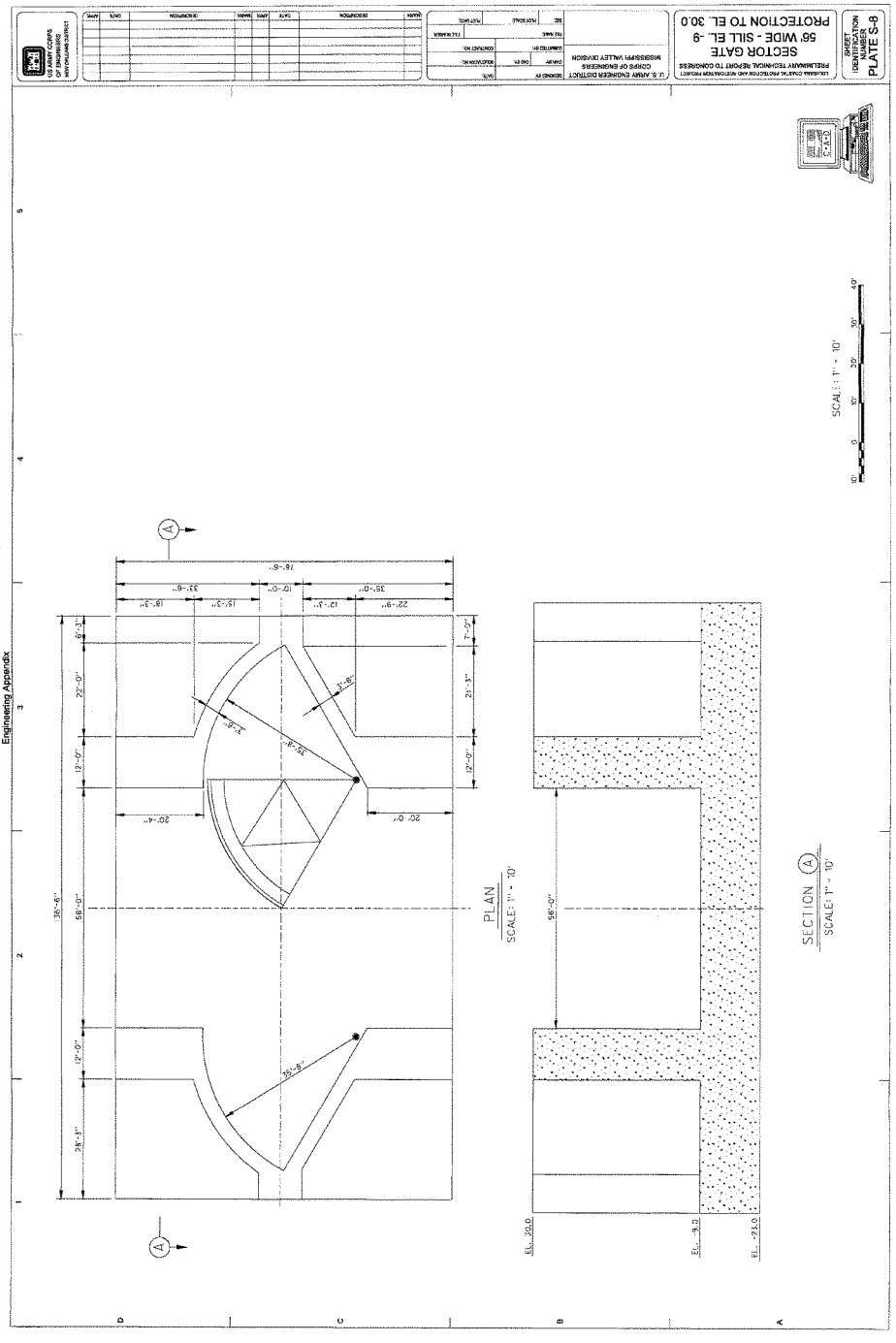
SHEET IDENTIFICATION NUMBER
PLATE S-3

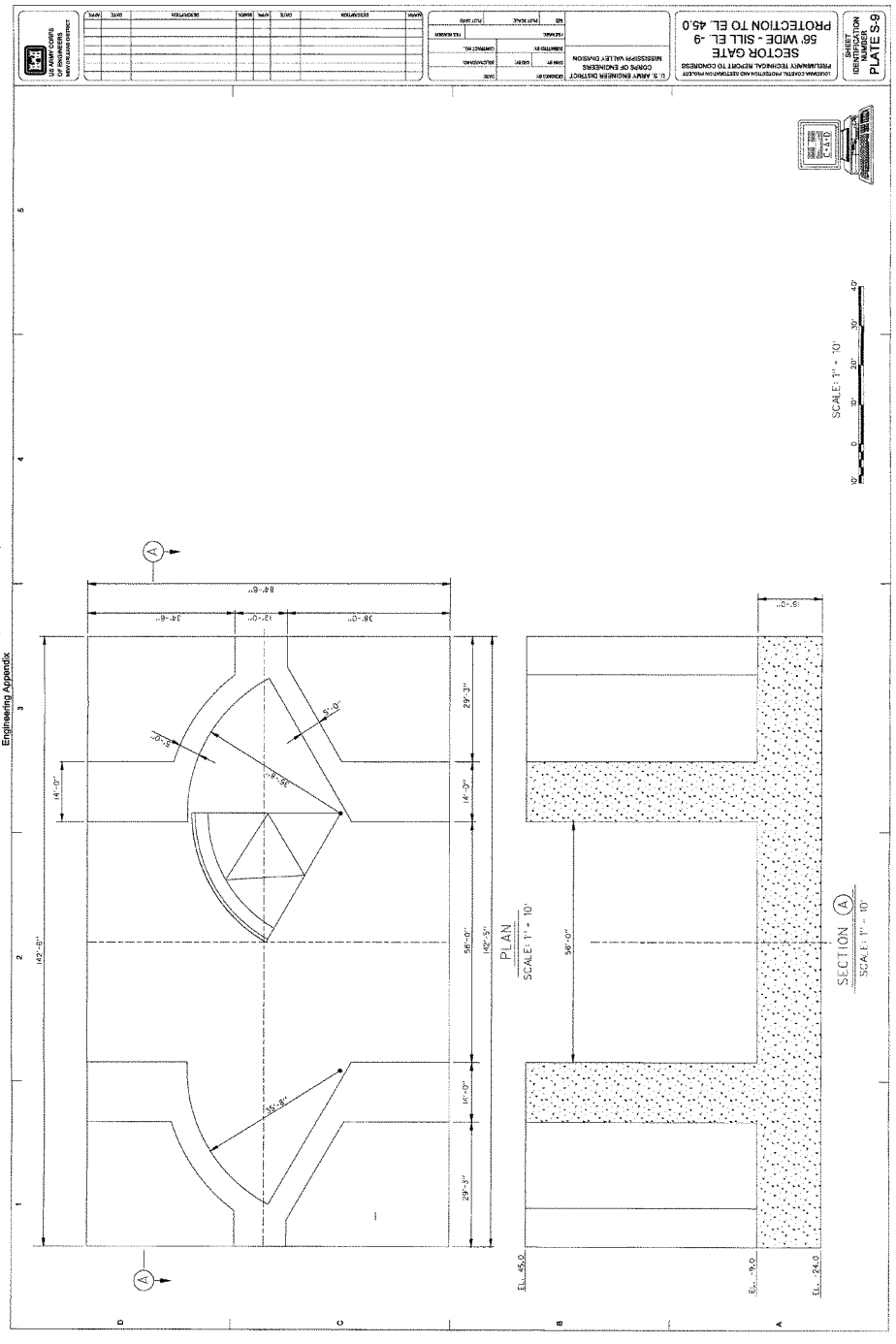

PRELIMINARY TECHNICAL REPORT TO CONGRESS
TYPICAL NEW VERTICAL
PUMP STATION
WITH FRONTING PROTECTION
SHEET IDENTIFICATION NUMBER
PLATE S-4
TYPICAL NEW VERTICAL PUMPING STATION
WITH FRONTING PROTECTION
SCALE: 1" = 5'
PROTECTED SIDE
FLOOD SIDE
BACKFILL LINE
STEEL SHEET PILE
RETAINING WALL
CONST. JT.
RETAINING WALL BEYOND
TOE SHEETING
SHEET PILE CUT-OFF WALL
BUILT TO EL. 30, 35, 45 FT. WATER LEVEL
FLOODWALL
BUTTRESS
PAVING
VERTICAL PUMP & MOTOR
PUMP
FORMED SUCTION INLET
SCREEN
WALL BEYOND
RIPRAP
NOTE:
1. PUMP DISCHARGE CUT OFF VALVE TO BE REMOTELY OPERATED FROM PUMPING STATION CONTROL ROOM.

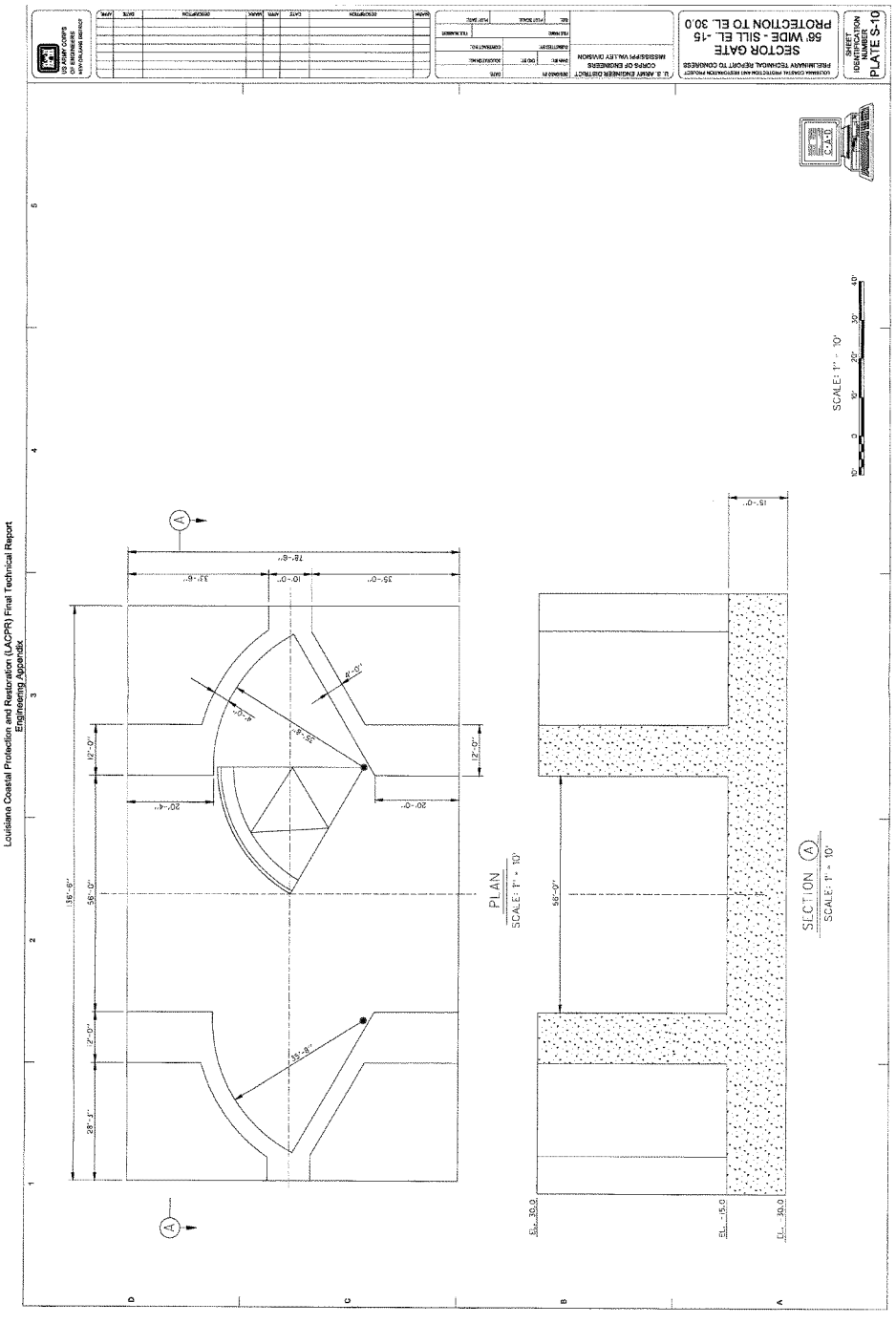

TYPICAL PUMP STATION HARDENING DETAILS
TYPICAL EXHAUST FAN OPENING BARRIER
TYPICAL WINDOW OPENING BARRIER
TYPICAL MISSLE BARRIER DETAILS
ELEVATION
SECTION
TYPICAL PUMP STATION HARDENING DETAILS
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS
SHEET IDENTIFICATION NUMBER PLATE S-5

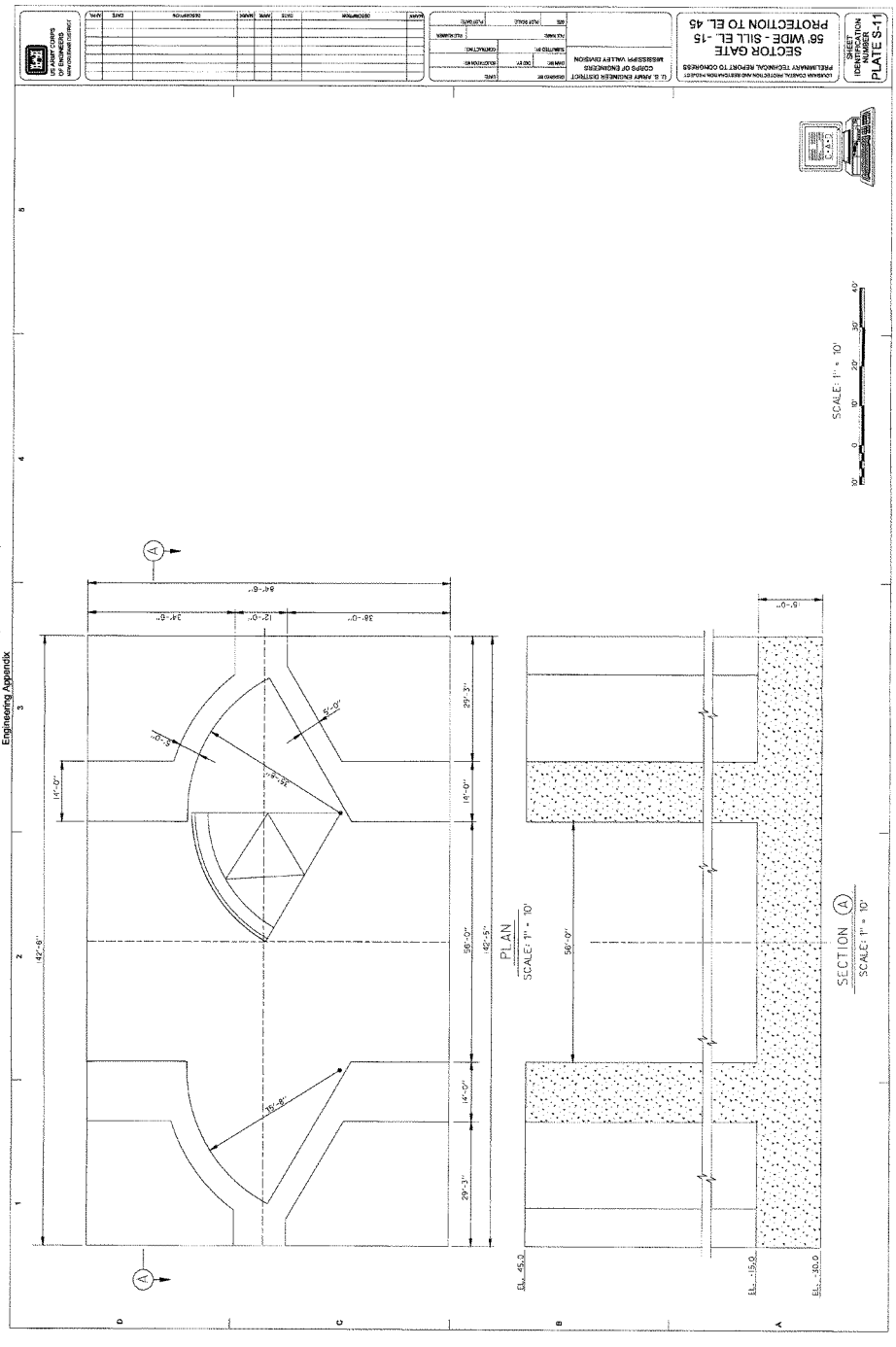

T-WALL SECTIONS
PRELIMINARY TECHNICAL REPORT TO CONGRESS
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
SHEET IDENTIFICATION NUMBER
PLATE 6
PROTECTED SIDE
FLOOD SIDE
PROTECTION TO EL. 30.0
PROTECTION TO EL. 35.0
PROTECTION TO EL. 45.0
24"Ø STEEL PILES TYPICAL
20"Ø STEEL PILES TYPICAL
PZ-27 SHEET PILING
SCALE: 1" = 5'

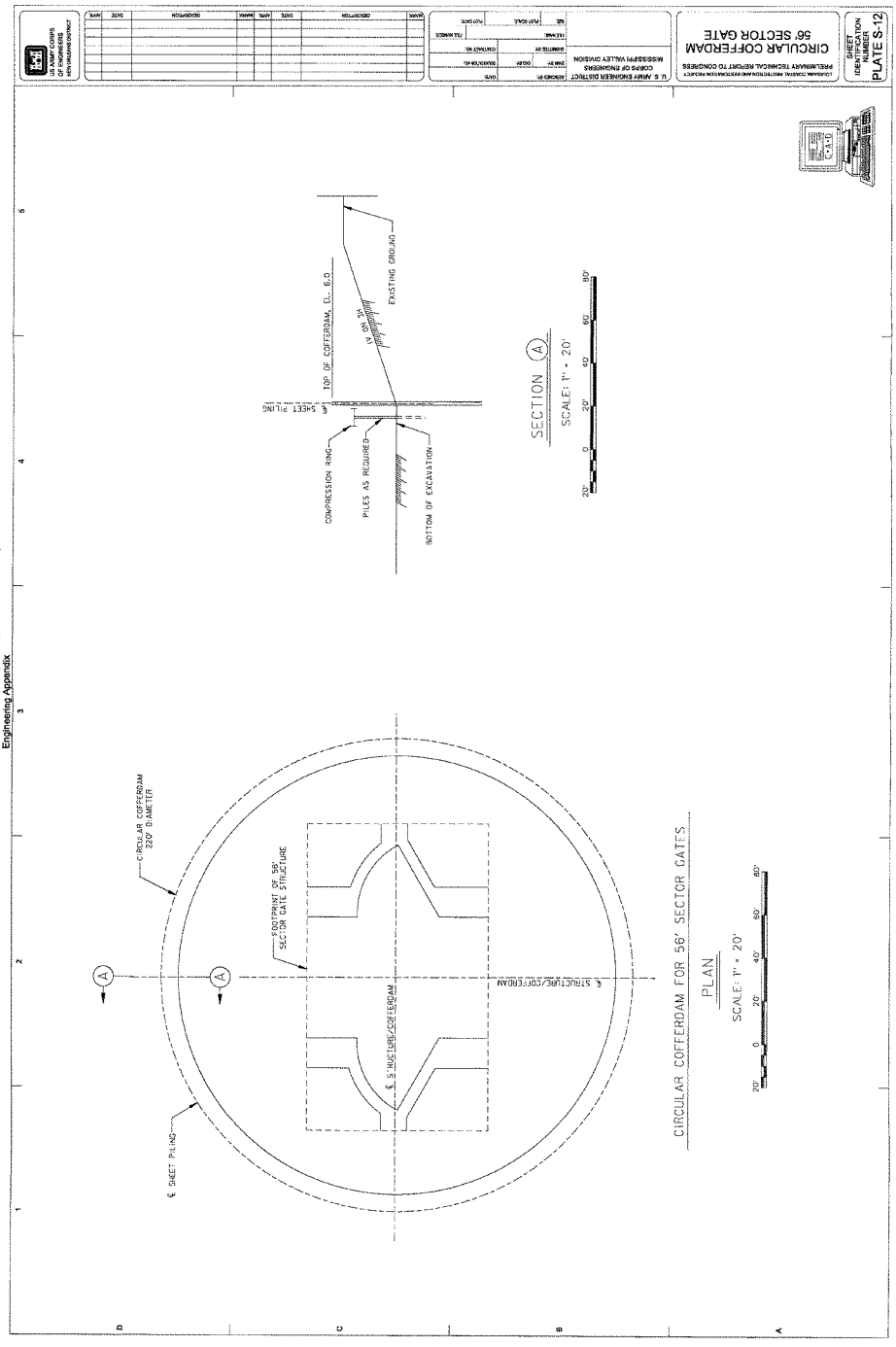

SLUICE GATE STRUCTURE
PROTECTION TO EL. 45.0
PLATE S-7
TYPICAL SLUICE GATE SECTION
SCALE: 1" = 10'
PROTECTED SIDE
FLOOD SIDE
LIFTING DEVICE
EL. 47.0
SURFACING
EL. 12.0
COMPACTED FILL
EL. 4.0
EL. 0.4
EL. -8.0
SLUICE GATE
TRASH RACK
BULKHEAD RECESS
EL. 15.0
EL. 10.0
RIPRAP OVER BEDDING
GEOTEXTILE SEPARATOR
E SHEET PILE

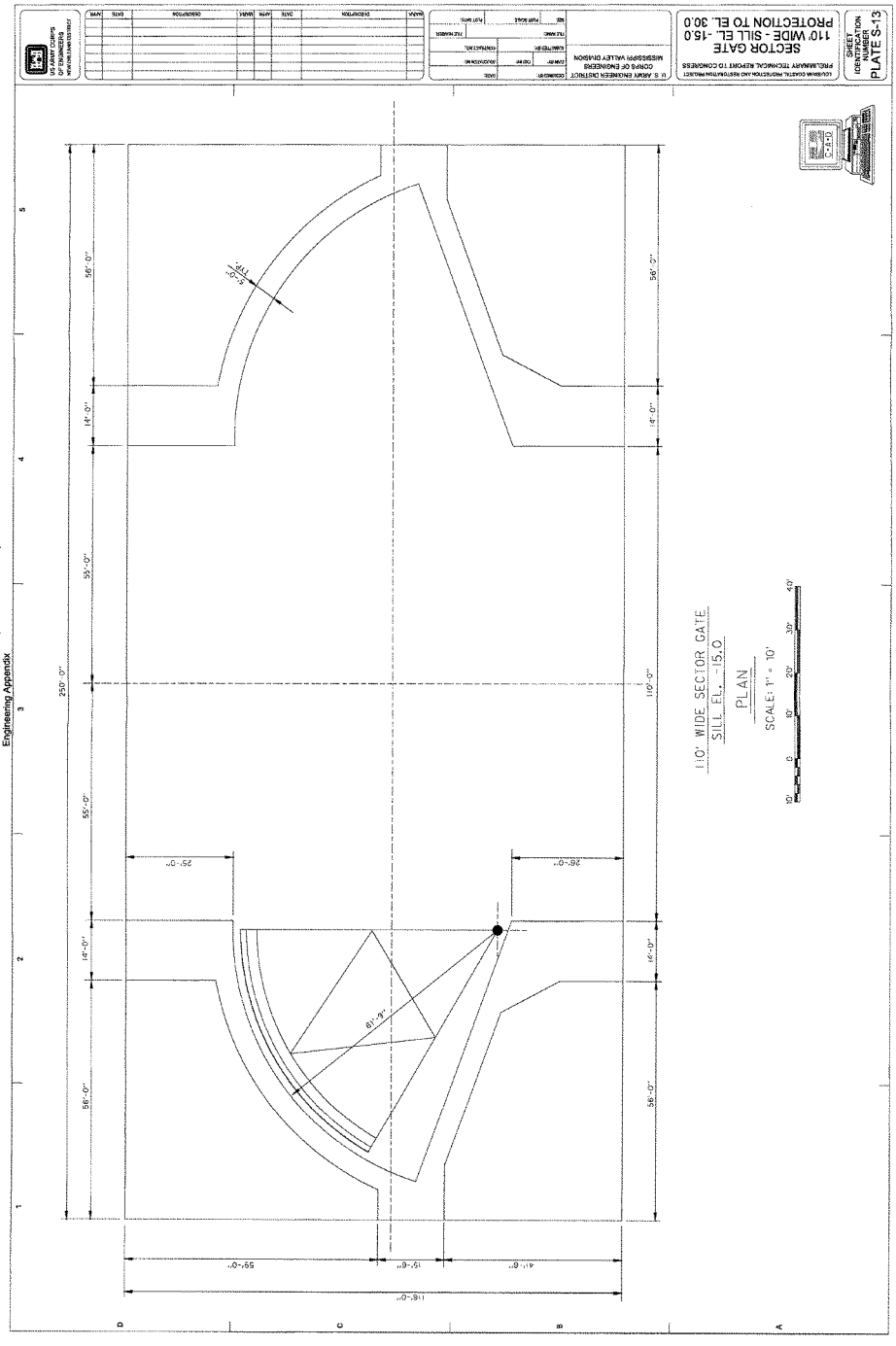

SECTOR GATE
56' WIDE - SILL EL. -9
PROTECTION TO EL. 30.0
PLATE C-8
PLAN
SCALE: 1" = 10'
SECTION A
SCALE: 1" = 10'
SCALE: 1" = 10'
EL. 30.0
EL. -9.0
EL. -23.0

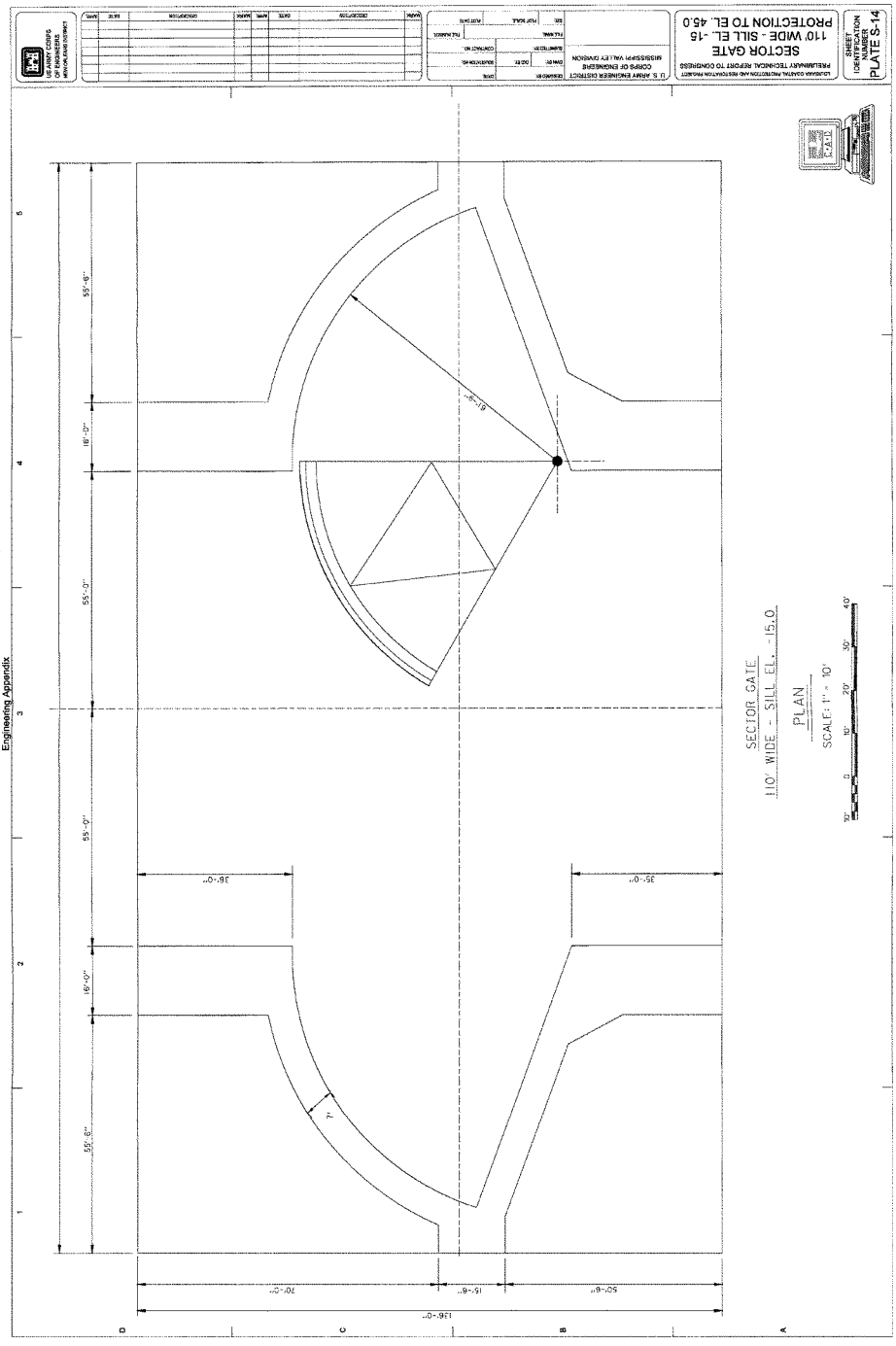

SECTOR GATE
56' WIDE - SILL EL -9
PROTECTION TO EL 45.0
SHEET IDENTIFICATION NUMBER
PLATE S-9
PLAN
SCALE: 1" = 10'
SECTION A
SCALE: 1" = 10'
SCALE: 1" = 10'
EL. 45.0
EL. -9.0
EL. 24.0

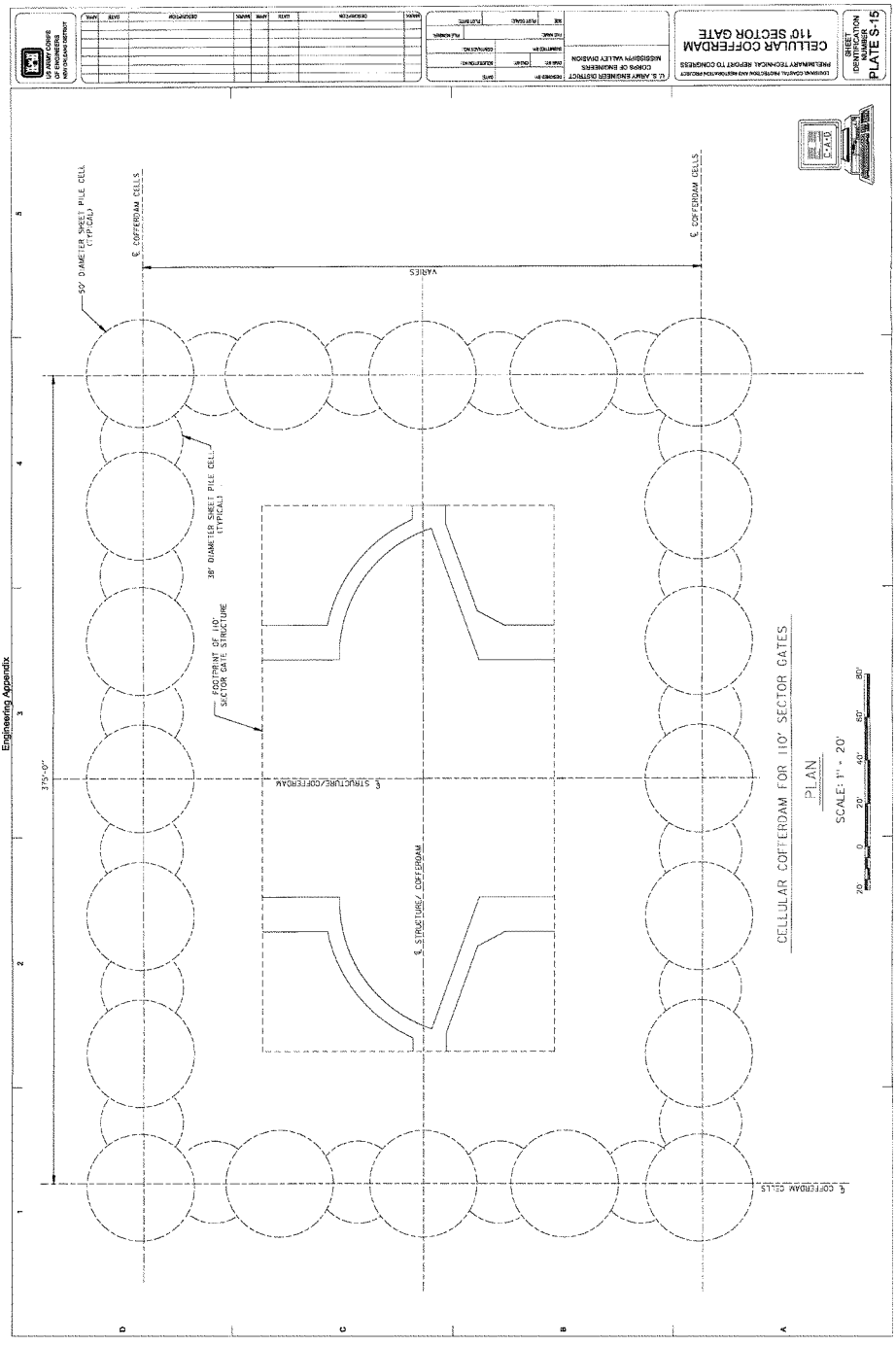

SECTOR GATE
55' WIDE - SILL EL. -15
PROTECTION TO EL. 30.0
PLATE S-10
PLAN
SCALE: 1" = 10'
SECTION (A)
SCALE: 1" = 10'
SCALE: 1" = 10'
Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix

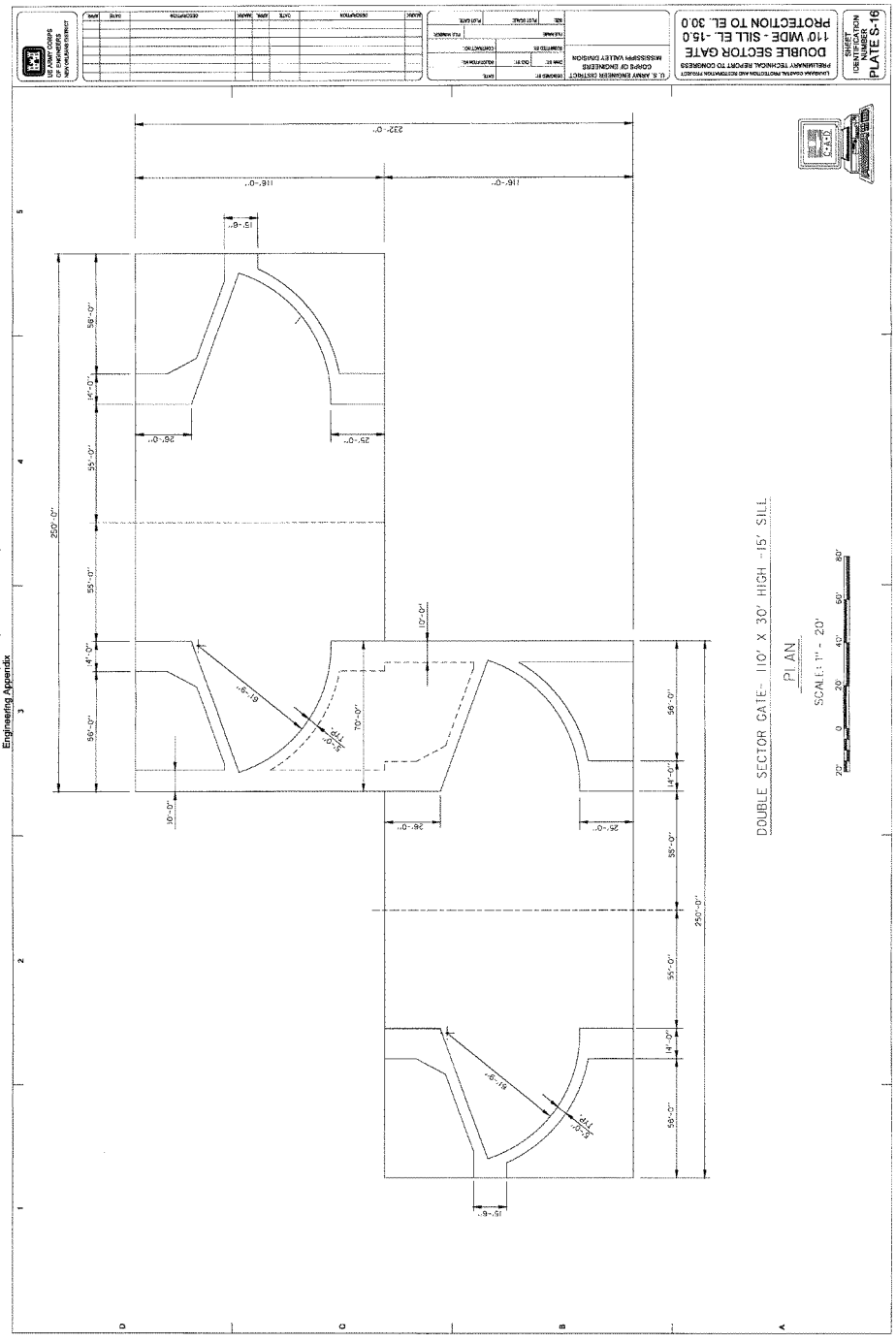

SECTOR GATE
56' WIDE - SILL EL. -15
PROTECTION TO EL. 45
PRELIMINARY TECHNICAL REPORT TO CONGRESS
LOUISIANA COASTAL PROTECTION AND RESTORATION STUDY
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
SHEET IDENTIFICATION NUMBER
PLATE G-11
PLAN
SCALE: 1" = 10'
SECTION (A)
SCALE: 1" = 10'
SCALE: 1" = 10'
64'-6"
38'-0"
12'-0"
26'-6"
42'-6"
56'-0"
14'-0"
29'-3"
Elv. -45.0
Elv. -15.0
Elv. -30.0

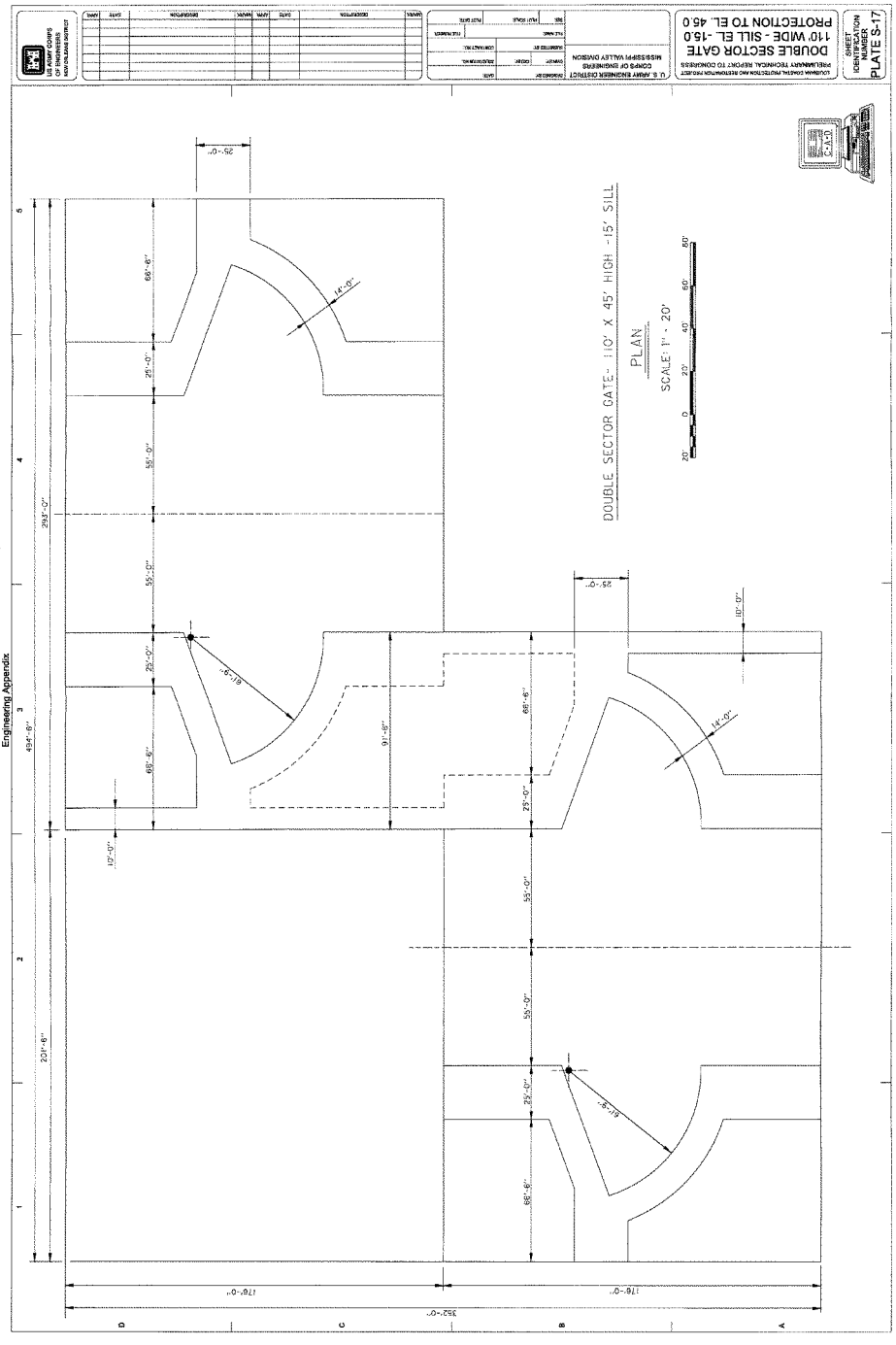

CIRCULAR COFFERDAM
56' SECTOR GATE
SHEET IDENTIFICATION NUMBER
PLATE S-12
CIRCULAR COFFERDAM FOR 56' SECTOR GATES
PLAN
SCALE: 1" = 20'
CIRCULAR COFFERDAM 220' DIAMETER
FOOTPRINT OF 56' SECTOR GATE STRUCTURE
℄ SHEET PILING
℄ STRUCTURE/COFFERDAM
℄ 3 BOX CULVERTS/COFFERDAM
SECTION A
SCALE: 1" = 20'
TOP OF COFFERDAM, EL. 6.0
EXISTING GROUND
SHEET PILING
COMPRESSION RING
PILES AS REQUIRED
BOTTOM OF EXCLUSION

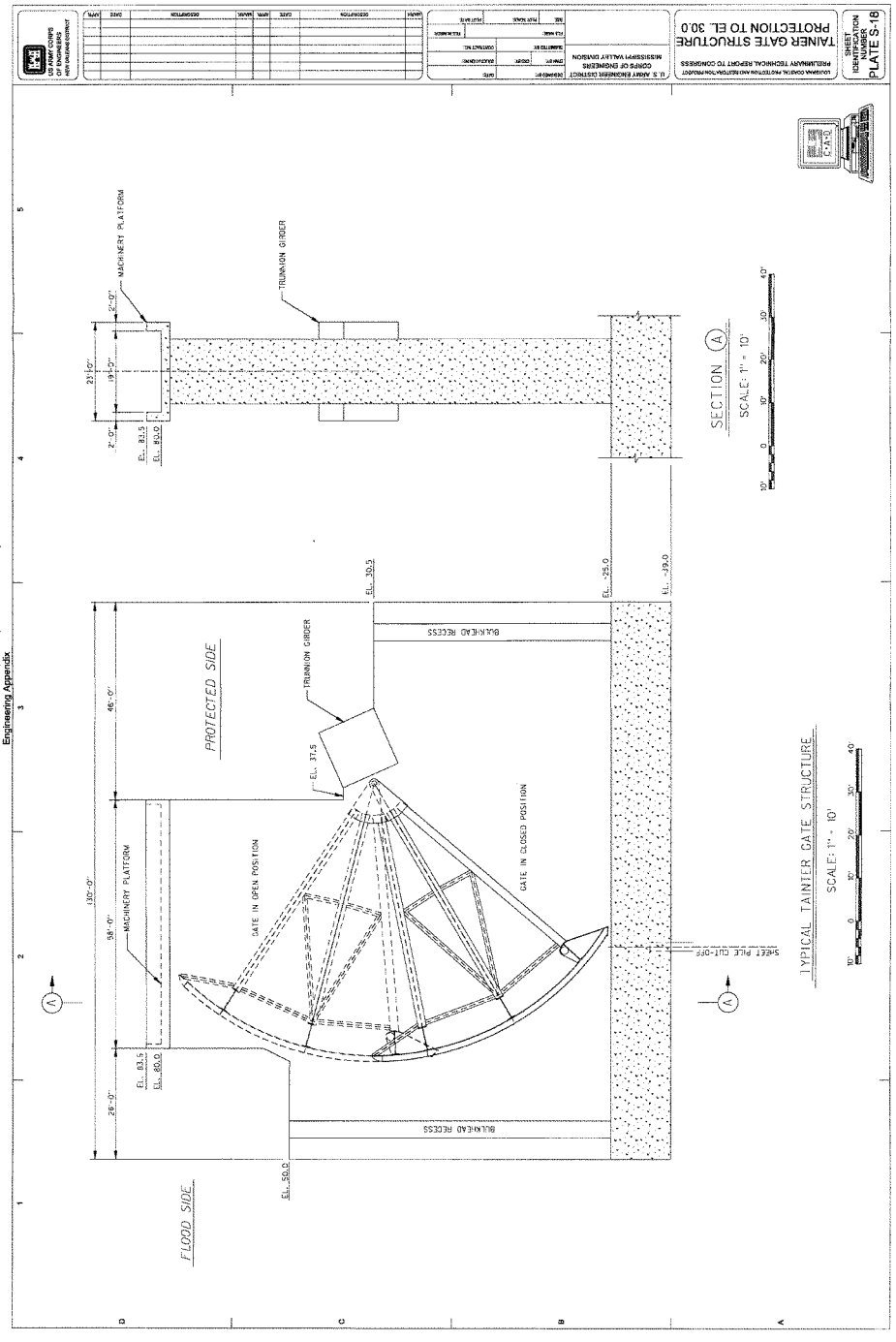

SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION TO EL. 30.0
SHEET IDENTIFICATION NUMBER
PLATE C-13
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
110' WIDE SECTOR GATE
SILL EL. -15.0
PLAN
SCALE: 1" = 10'
250'-0"
56'-0"
14'-0"
55'-0"
110'-0"
25'-0"
26'-0"
59'-0"
15'-6"
41'-6"
118'-0"


SECTOR GATE
110' WIDE - SILL EL. -15.0
PLAN
SCALE: 1" = 10'
PLATE S-14
SECTOR GATE
110' WIDE - SILL EL. -15
PROTECTION TO EL. 45.0
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
55'-6"
18'-0"
55'-0"
55'-0"
16'-0"
55'-6"
36'-0"
35'-0"
70'-0"
15'-6"
50'-6"
136'-0"


CELLULAR COFFERDAM 110' SECTOR GATE
PLATE S-15
CELLULAR COFFERDAM FOR 110' SECTOR GATES
PLAN
SCALE: 1" = 20'
50' DIAMETER SHEET PILE CELL (TYPICAL)
38' DIAMETER SHEET PILE CELL (TYPICAL)
FOOTPRINT OF 110' SECTOR GATE STRUCTURE
℄ COFFERDAM CELLS
VARIES
℄ STRUCTURE/ COFFERDAM
375'-0"


DOUBLE SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION TO EL. 30.0
PLATE S-16
DOUBLE SECTOR GATE- 110' X 30' HIGH -15' SILL
PLAN
SCALE: 1" = 20'
232'-0"
116'-0"
250'-0"
70'-0"
26'-0"
25'-0"
15'-0"
10'-0"
58'-0"
14'-0"
55'-0"
41'-0"
R=9'-8"
R=10'-0"
R=8'-10"
R=10'-0"


DOUBLE SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION TO EL. 45.0
SHEET IDENTIFICATION NUMBER
PLATE S-17
U.S. ARMY ENGINEER DISTRICT
CWPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
DOUBLE SECTOR GATE- 110' X 45' HIGH -15' SILL
PLAN
SCALE: 1" - 20'
25'-0"
68'-6"
25'-0"
55'-0"
55'-0"
25'-0"
68'-6"
293'-0"
49'-8"
10'-0"
91'-8"
14'-0"
R=45'
68'-6"
25'-0"
55'-0"
55'-0"
25'-0"
68'-6"
203'-8"
176'-0"
352'-0"
176'-0"
A
B
C
D
1
2
3
4
5



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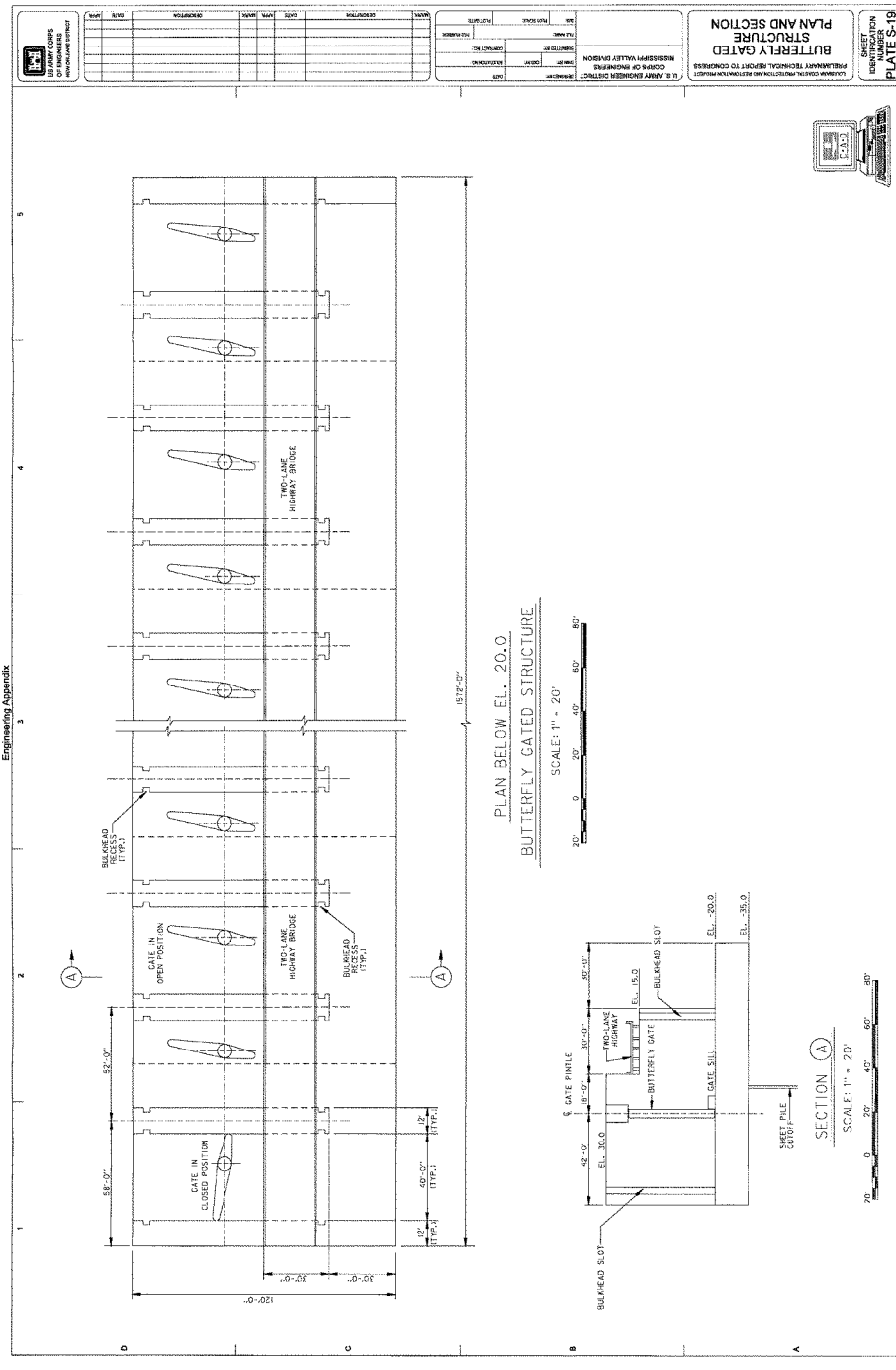
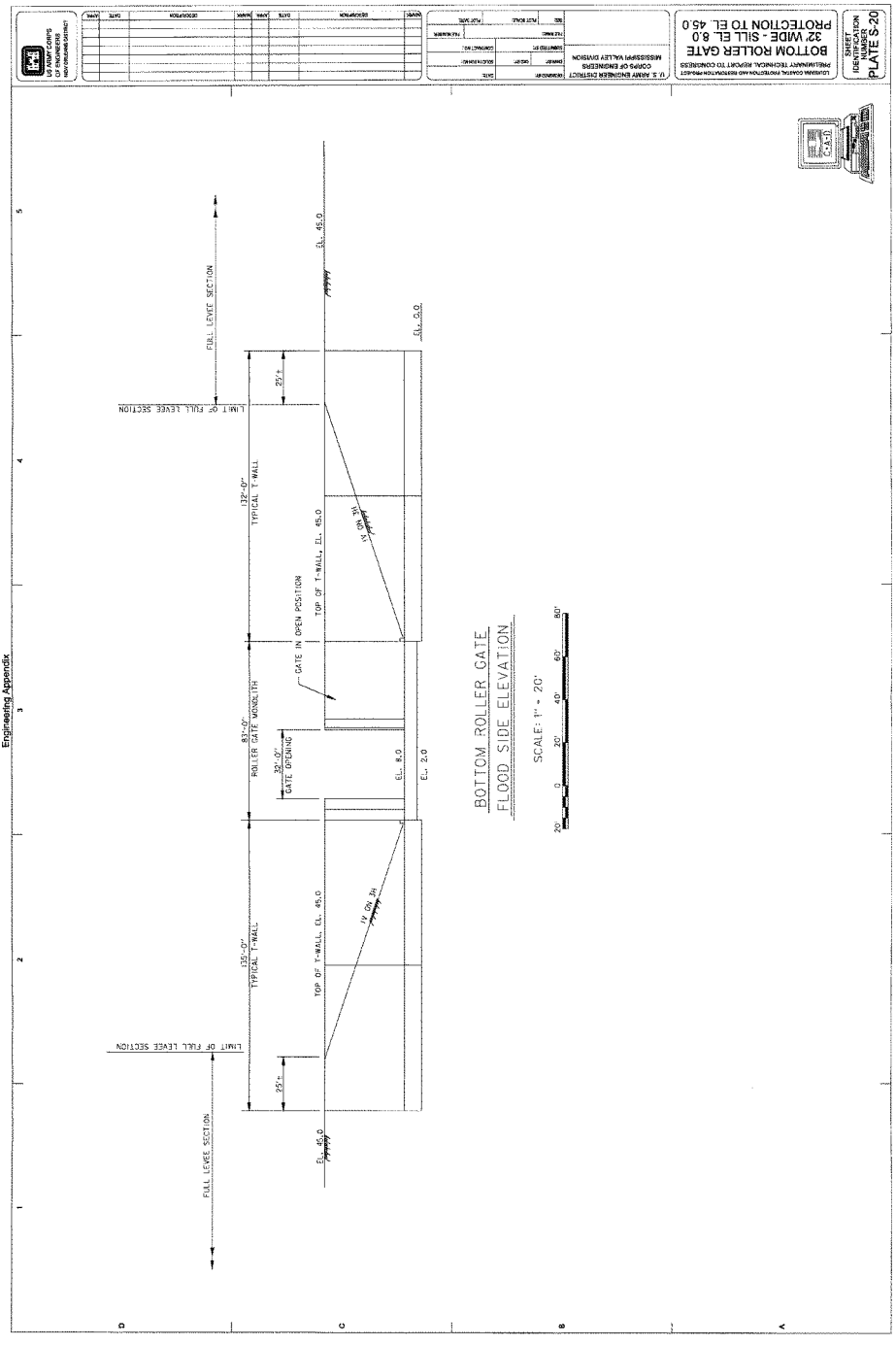
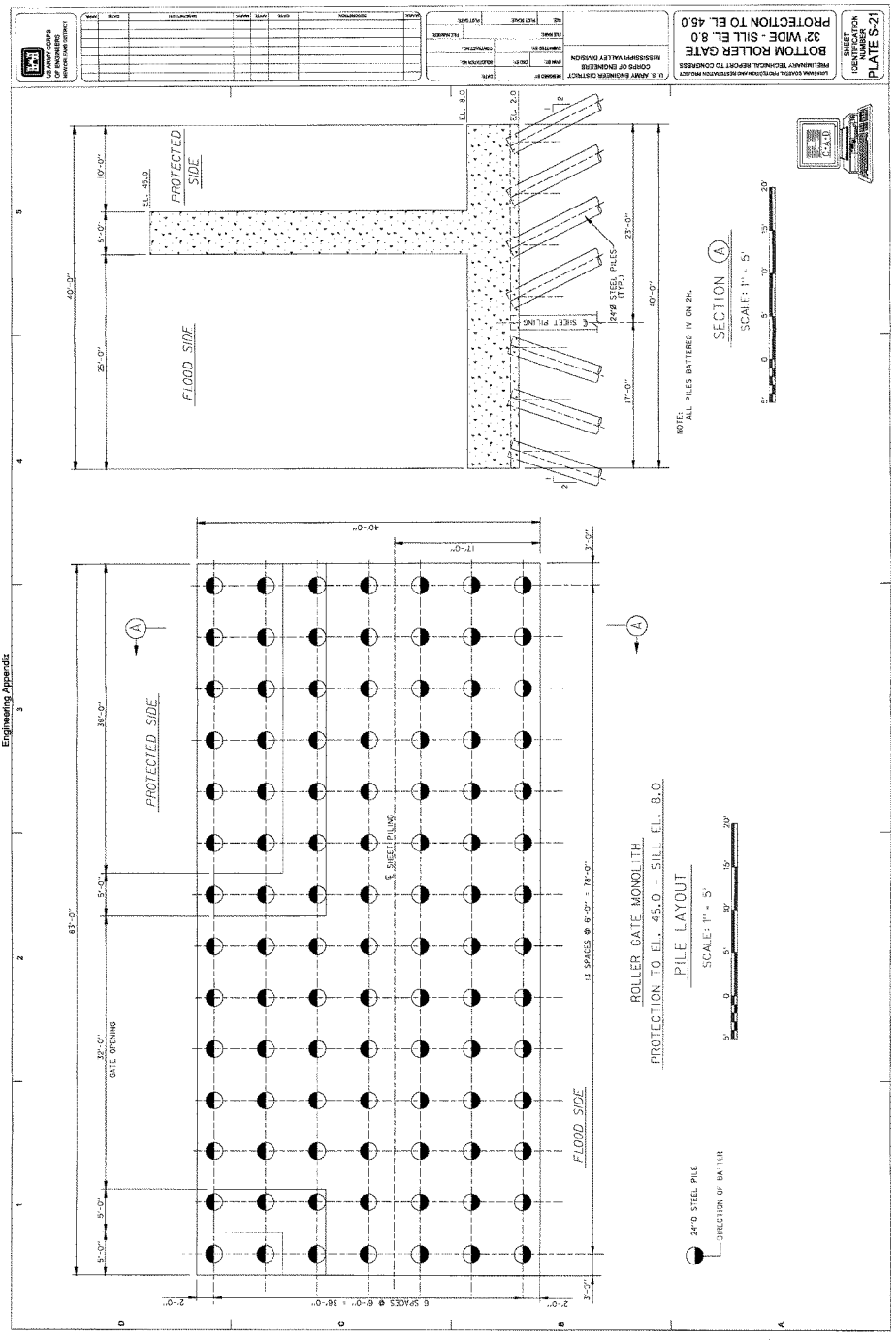

BUTTERFLY GATED STRUCTURE
PLAN AND SECTION
SHEET IDENTIFICATION NUMBER
PLATE C-19
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
PLAN BELOW EL. 20.0
BUTTERFLY GATED STRUCTURE
SCALE: 1" = 20'
SECTION (A)
SCALE: 1" = 20'
GATE IN OPEN POSITION
GATE IN CLOSED POSITION
TWO-LANE HIGHWAY BRIDGE
BULKHEAD RECESS (TYP.)
BULKHEAD SLOT
BUTTERFLY GATE
GATE SILL
SHEET PILE CUTOFF
EL. 15.0
EL. 20.0
EL. 30.0
120'-0"
82'-0"
58'-0"
42'-0"
30'-0"
8'-0"
12' TYP.
42'-0" TYP.
152'-0"


BOTTOM ROLLER GATE
32' WIDE - SILL EL. 8.0
PROTECTION TO EL. 45.0
SHEET IDENTIFICATION NUMBER
PLATE E-20
BOTTOM ROLLER GATE
FLOOD SIDE ELEVATION
SCALE: 1" = 20'
FULL LEVEE SECTION
LIMIT OF FULL LEVEE SECTION
135'-0" TYPICAL T-WALL
25'±
83'-0" ROLLER GATE MONOLITH
32'-0" GATE OPENING
132'-0" TYPICAL T-WALL
GATE IN OPEN POSITION
TOP OF T-WALL, EL. 45.0
EL. 45.0
EL. 8.0
EL. 2.0
1V ON 3H



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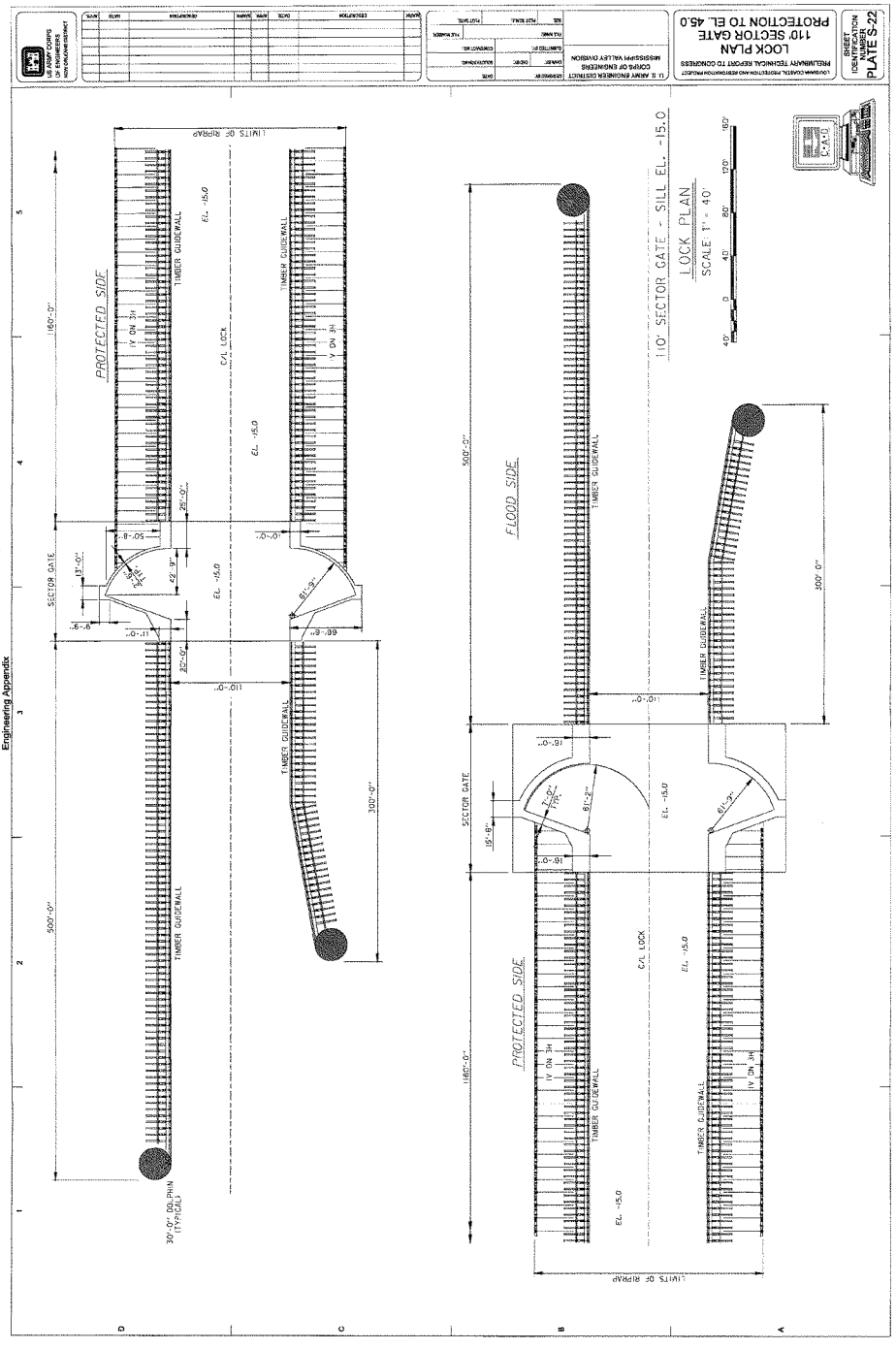
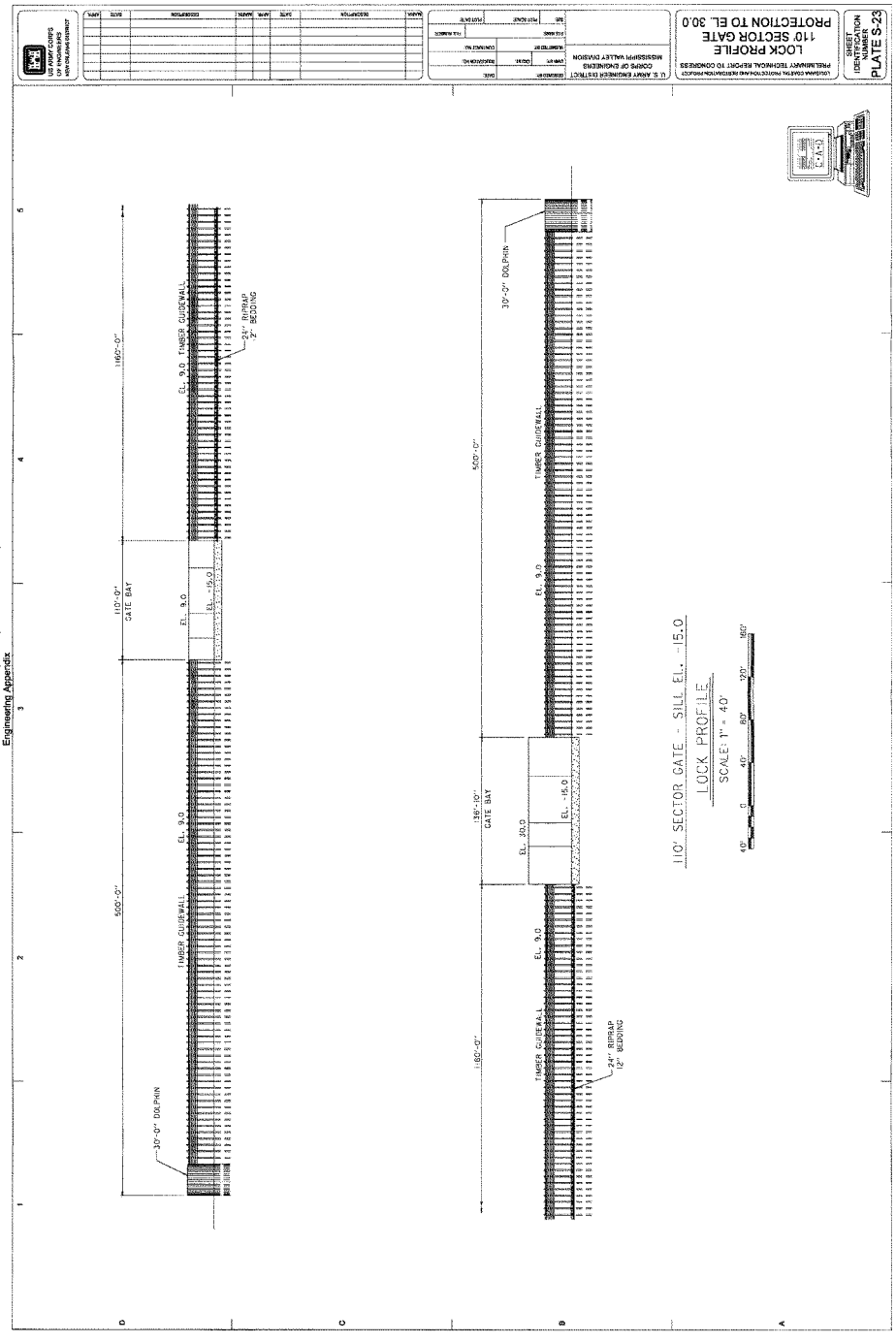
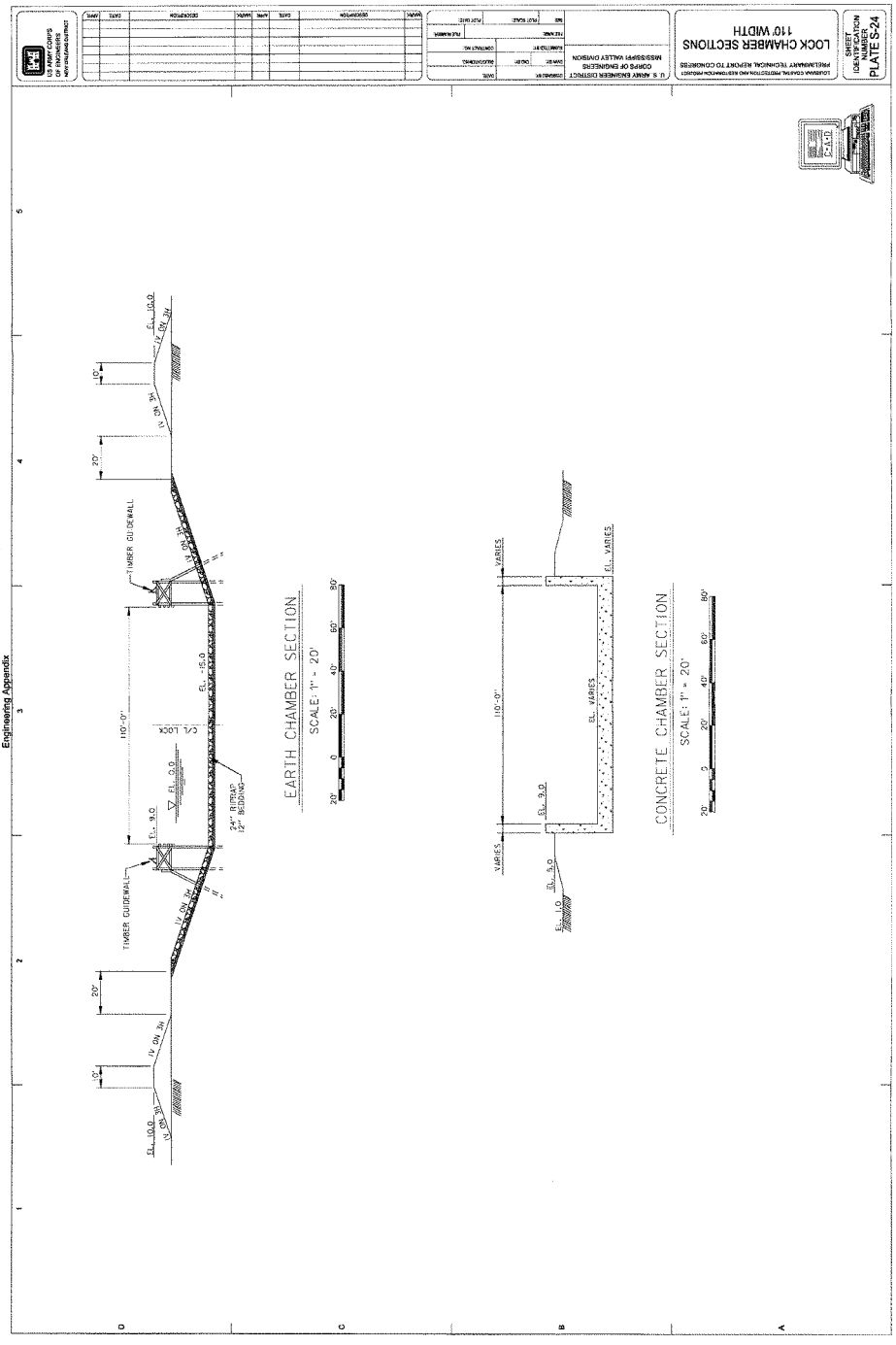
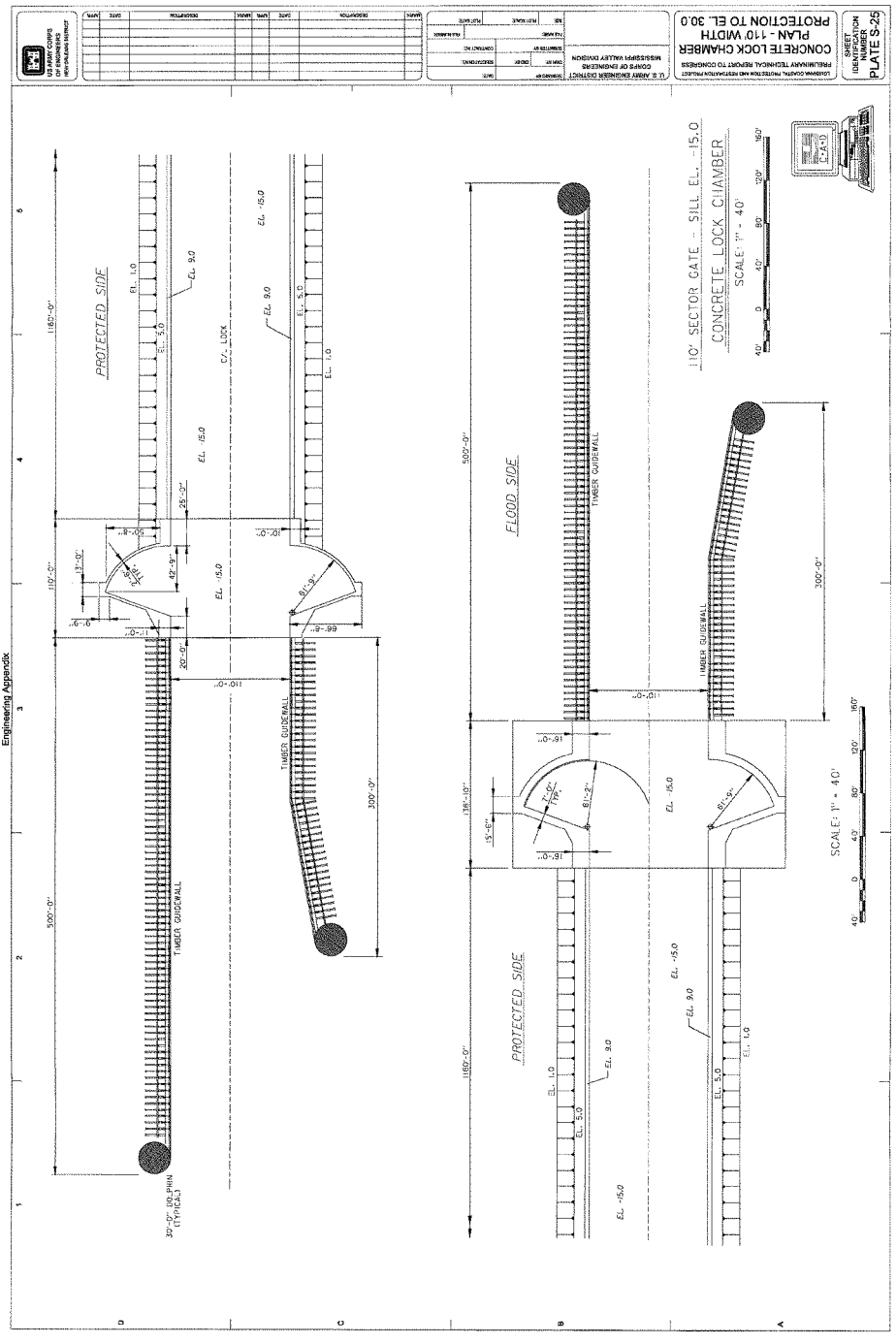
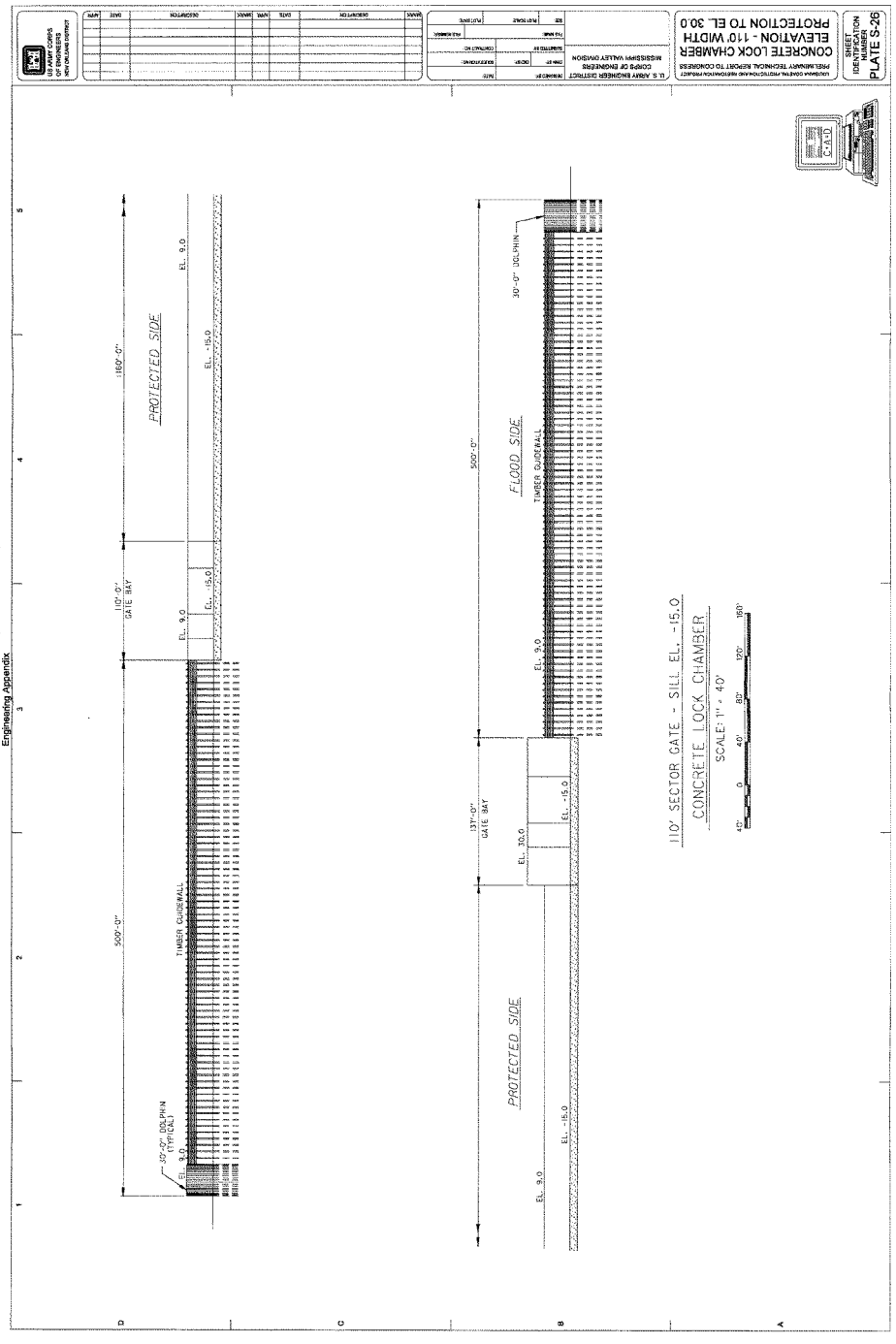
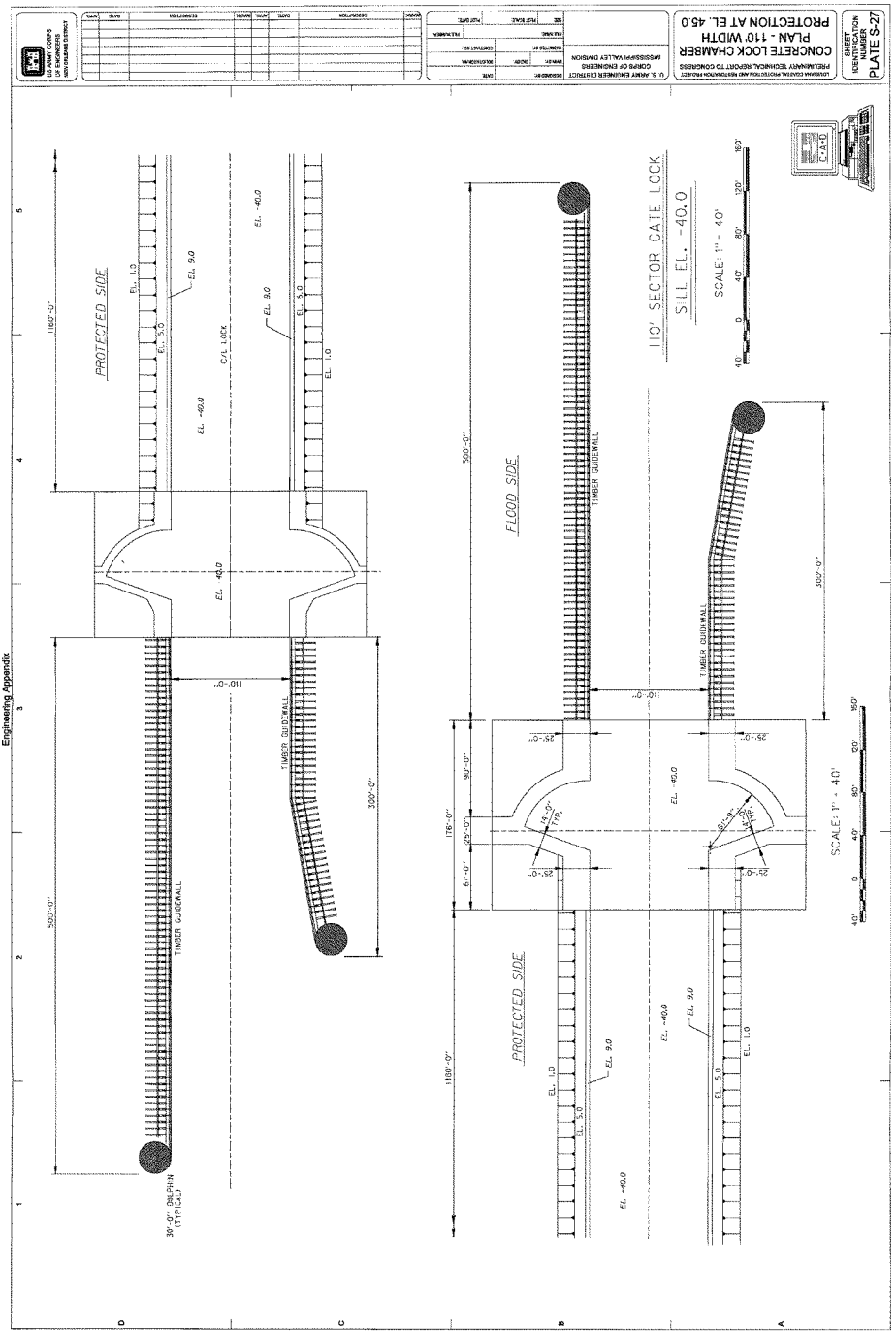

110' SECTOR GATE
LOCK PLAN
PROTECTED SIDE
FLOOD SIDE
TIMBER GUIDEWALL
C/L LOCK
EL. -15.0
SECTOR GATE
LIMITS OF RIPRAP
30'-0" DOLPHIN (TYPICAL)
LOCK PLAN
SCALE 1" = 40'
SILL EL. -15.0
PLATE S-22


LOCK PROFILE
110' SECTOR GATE
PROTECTION TO EL. 30.0
SHEET IDENTIFICATION NUMBER
PLATE S-23
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
110' SECTOR GATE - SILL EL. -15.0
LOCK PROFILE
SCALE: 1" = 40'
500'-0"
110'-0"
GATE BAY
EL. 9.0
EL. -15.0
TIMBER GUIDEWALL
30'-0" DOLPHIN
24" RIPRAP
2' BEDDING
500'-0"
136'-10"
GATE BAY
EL. 10.0
EL. -15.0
EL. 9.0
TIMBER GUIDEWALL
30'-0" DOLPHIN
24" RIPRAP
12" BEDDING


LOCK CHAMBER SECTIONS
110' WIDTH
PLATE S-24
EARTH CHAMBER SECTION
SCALE: 1" = 20'
CONCRETE CHAMBER SECTION
SCALE: 1" = 20'
TIMBER GUIDEWALL
24" RIPRAP
12" BEDDING
110'-0"
VARIES
EL. VARIES
EL. 10.0
EL. 9.0
EL. -15.0
EL. -5.0
1V ON 3H
1V ON 4H
1V ON 5H
20'
10'


PROTECTED SIDE
FLOOD SIDE
TIMBER GUIDEWALL
110' SECTOR GATE - SILL EL. -15.0
CONCRETE LOCK CHAMBER
SCALE: 1" = 40'
CONCRETE LOCK CHAMBER
PLAN - 110' WIDTH
PROTECTION TO EL. 30.0
SHEET IDENTIFICATION NUMBER
PLATE S-25


CONCRETE LOCK CHAMBER
ELEVATION - 110' WIDTH
PROTECTION TO EL. 30.0
PRELIMINARY TECHNICAL REPORT TO CONGRESS
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
U.S. ARMY ENGINEER DISTRICT
COPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION
SHEET IDENTIFICATION NUMBER
PLATE S-26
C-1A-D
PROTECTED SIDE
FLOOD SIDE
TIMBER GUIDEWALL
30'-0" DOLPHIN
500'-0"
180'-0"
110' SECTOR GATE - SILL EL. -15.0
CONCRETE LOCK CHAMBER
SCALE: 1" = 40'
EL. 9.0
EL. -15.0
EL. 30.0
110'-0" GATE BAY
137'-0" GATE BAY
Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix


CONCRETE LOCK CHAMBER
PLAN - 110' WIDTH
PROTECTION AT EL. 45.0
PLATE E-27
110' SECTOR GATE LOCK
SILL EL. -40.0
SCALE: 1" = 40'
PROTECTED SIDE
FLOOD SIDE
TIMBER GUIDEWALL
C/L LOCK
500'-0"
1160'-0"
300'-0"
110'-0"
90'-0"
176'-0"
61'-0"
25'-0"
EL. -40.0
EL. 9.0
EL. 5.0
EL. 1.0
30'-0" DOLPHIN (TYPICAL)

PROTECTED SIDE
GATE BAY
TIMBER CUTOFFWALL
30'-0" DOLPHIN
FLOOD SIDE
110' SECTOR GATE LOCK
SILL EL. -40.0
SCALE: 1" = 40'
CONCRETE LOCK CHAMBER
ELEVATION - 110' WIDTH
PROTECTION TO EL. 30.0
SHEET IDENTIFICATION NUMBER
PLATE S-28

ANNEX 1

COASTAL RESTORATION FEATURES

ANNEX 1
COASTAL RESTORATION FEATURES

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ANNEX 1
COASTAL RESTORATION FEATURES

GENERAL

A group of coastal restoration measures were developed by the Habitiat Evaluation Team (HET). These measures were put together to develop alternative coastal restoration plans. The goal of these coastal restoration plans is to sustain the current coastline to maintain the current level of hurricane and storm damage risk reduction. Details of how these measures and alternatives were developed can be found in the Coastal Restoration Plan Component Appendix.

One of the alternative plans was selected as a representative plan for use in the Multi Criteria Decision Making Analysis (MCDA). Designs and costs were developed for those measures contained in the representative plan. A list of those measures is shown in table 1 below and shown on figures 1-5. These measures are described in detail in the paragraphs below. Measures are divided by Planning Units as shown in figures 1-5. These Planning Units are described in detail in the Background section of the Engineering Appendix.

Table 1 - Coastal Restoration Plan Component – Representative Plan Features

Figure #	Map ID #	Measure
Planning Unit 1 – Alternative R2 (Pulsed Diversion)		
1	2-1	Blind River Diversion
1	2-2	Hope Canal Diversion
1	2-3	Labranche Diversion
1	2-4	Bayou Bienvenue Diversion
1	2-5	East New Orleans Land Bridge Marsh Creation
1	2-6	Bayou LaLoutre Diversion
1	2-7	Biloxi Marshes Shore Protection
1	2-8	Biloxi Marshes Marsh Creation
1	2-9	Bayou Terre aux Bouefs Diversion
1	2-10	Bayou Terre aux Bouefs Marsh Creation
1	2-11	Breton Sound Strategic Land Bridge
1	2-12	Caernarvon Diversion
1	2-13	Caernarvon Area Marsh Creation
1	2-14	Bayou Lamoque Diversion
1	2-15	Grand Bay Diversion
Planning Unit 2 – Alternative R2 (Pulsed Diversion)		
2	2-1	Lagan Diversion

Table 1 - Coastal Restoration Plan Component – Representative Plan Features		
Figure #	Map ID #	Measure
2	2-2	Edgard Diversion
2	2-3	Davis Pond Freshwater Diversion Reauthorization
2	2-4	Naomi Diversion
2	2-5	Myrtle Grove Diversion
2	2-6	Strategic Marsh Creation in Lower Basin
2	2-7	North Bay Rim Marsh Creation /Protection
2	2-8	West Point a la Hache Diversion
2	2-9	Port Sulphur Diversion
2	2-10	Buras Diversion
2	2-11	Fort Jackson Diversion
2	2-12	Barrier Island Restoration
Planning Unit 3a – Alternative R1 (Mississippi River Diversions Alternative)		
3	1-1	HNC Lock Multi-purpose Operation
3	1-2	Convey Atchafalaya River Water via the GIWW
3	1-3	Lapeyrouse Canal Diversion
3	1-4	Blue Hammock Diversion
3	1-5	Upper Lake Boudreaux Basin Mississippi River Diversion
3	1-6	East Terrebonne Mississippi River Diversion
3	1-7	Grand Bayou & Jean LaCroix Basin Mississippi River Diversion
3	1-8	Pipeline Conveyance Marsh Creation
3	1-9	North Terrebonne Bay Rim Marsh Creation
3	1-10	DuLarge to Grand Caillou Landbridge Marsh Creation
3	1-11	South Caillou Landbridge Marsh Creation
3	1-12	Isles Dernieres Restoration
3	1-13	Timbalier Islands Restoration
Planning Unit 3b – Alternative R1 (Marsh Creation with Shoreline Protection)		
4	1-1	Penchant Basin Plan
4	1-2	Convey Atchafalaya River water via the GIWW
4	1-3	Relocate the Navigation Channel through Lower Atchafalaya River Delta
4	1-4	Increase Sediment Transport Down Wax Lake Outlet
4	1-5	Barrier Reef from Eugene Island to Pointe au Fer Island
4	1-6	Blue Hammock Bayou Freshwater Introduction
4	1-7	Gulfshore Protection at Point au Fer Island
4	1-8	Freshwater Bayou Bank Protection, Belle Isle to Lock
4	1-9	Southwest Pass Bank Protection
4	1-10	Marsh Island Shoreline Protection

Table 1 - Coastal Restoration Plan Component – Representative Plan Features		
Figure #	Map ID #	Measure
4	1-11	Gulfshore Protection from Freshwater Bayou to Southwest Pass
4	1-12	Shoreline Protection at Vermillion Bay & West Cote Blanche Bay
4	1-13	East Cote Blanche Bay Shore Protection
4	1-14	Bayou Decade Area Marsh Creation
4	1-15	Brady Canal Area Marsh Creation
4	1-16	Pointe au Fer Island Marsh Creation
4	1-17	Marsh Island Marsh Creation
4	1-18	Wax Lake Outlet Delta Marsh Creation
4	1-19	Bayou Penchant Area Marsh Creation
4	1-20	Terrebonne GIWW Area Marsh Creation
PU4 – Alternative R1 – (Marsh Creation with Shoreline Protection)		
5	1-1	Marsh Creation at Mud Lake
5	1-2	Marsh Creation at South Grand Chenier
5	1-3	Marsh Creation at South Pecan Island
5	1-4	Marsh Creation at East Pecan Island
5	1-5	Marsh Creation at No-Name Bayou
5	1-6	Marsh Creation at NW Calcasieu Lake
5	1-7	Marsh Creation at East Calcasieu Lake
5	1-8	Marsh Creation at Black Bayou
5	1-9	Marsh Creation at Gum Cove
5	1-10	Marsh Creation at Cameron Meadows
5	1-11	Marsh Creation at Central Canal
5	1-12	GIWW bank Stabilization
5	1-13	Grand Lake Bank Stabilization
5	1-14	White Lake Bank Stabilization
5	1-15	Gulf Shoreline Stabilization (Sabine River to Calcasieu River)
5	1-16	Gulf Shoreline Stabilization (Calcasieu River to Freshwater Bayou)

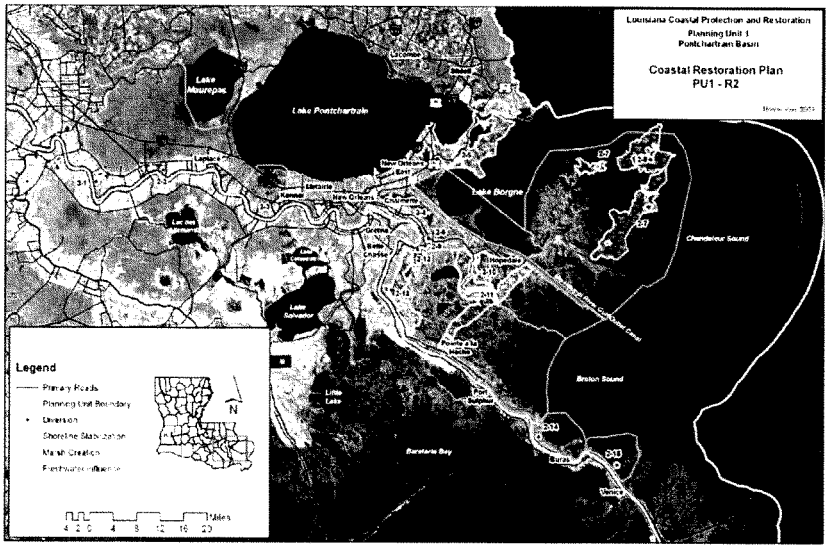

Figure 1: PU1 Representative Plan Features

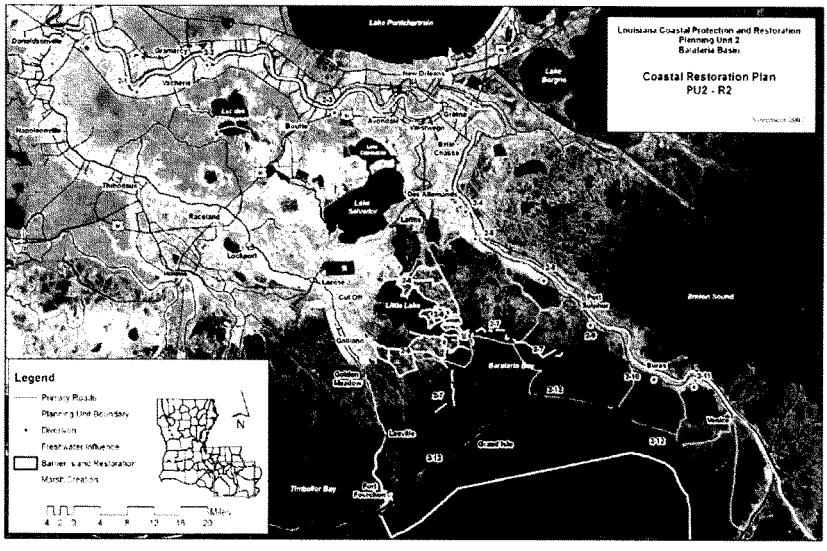

Figure 2: PU2 Representative Plan Features

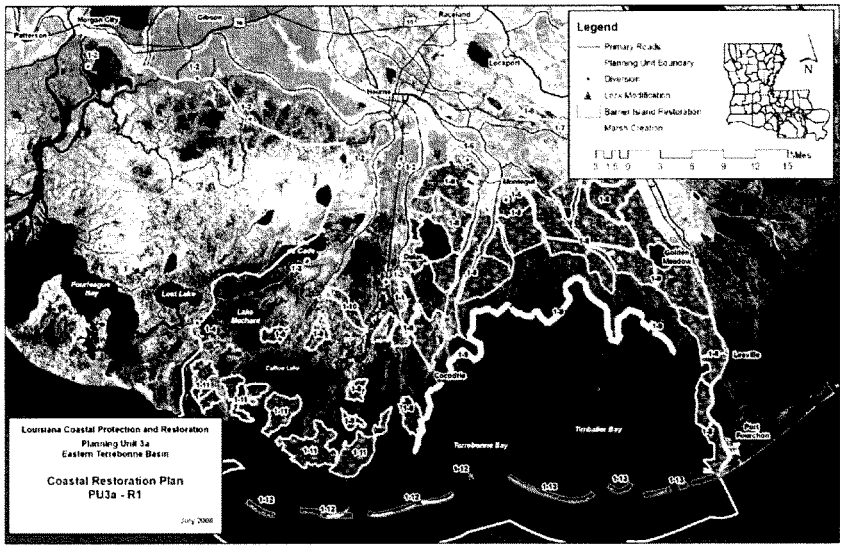

Figure 3: PU3a Representative Plan Features

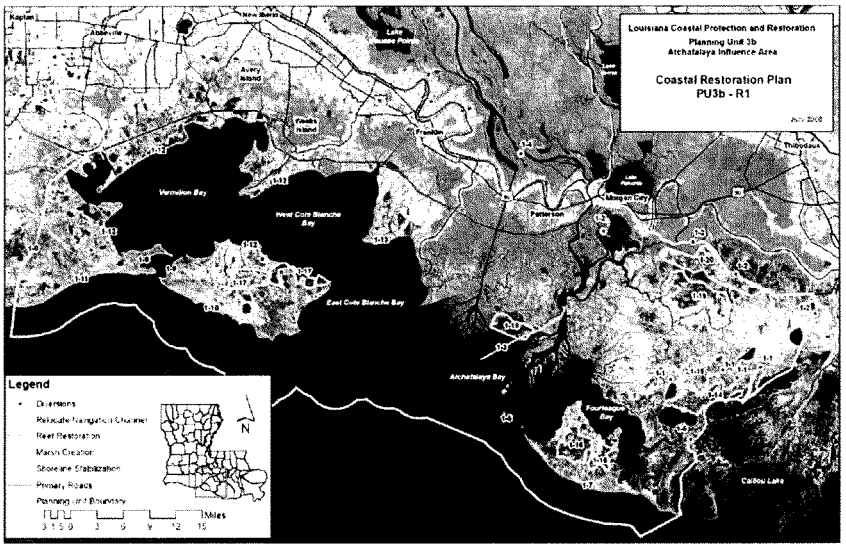

Figure 4: PU3b Representative Plan Features

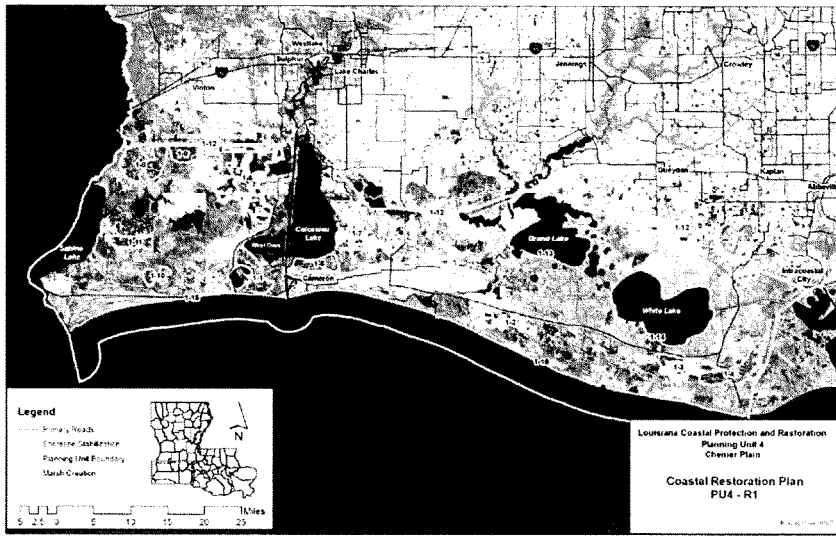

Figure 5: PU4 Representative Plan Features

DIVERSIONS

GENERAL

Freshwater diversions are diversions designed to take river water and its accompanying sediment and divert it into the surrounding marshlands to create or enhance marsh. For this effort diversions were sized in order to create or sustain a certain amount of marsh acreage. The NRCS-Boustany model, with ERDC modifications, was used to provide rough estimates of receiving area benefits for each year of the 100-year project life. Model benefits are based in part on Mississippi River discharge and the corresponding suspended sediment concentration. The results of the model were used to determine the required capacity for each diversion. These capacities are shown in Table 2. Details of the NRCS-Boustanay model and its application can be found in the Coastal Restoration Plan Component Appendix.

Table 2 – Required Diversion Capacities

Diversion Site	Low Flow Year		High Flow Year	
	Avg. Flow (cfs)	High Flow (cfs)	Avg. Flow (cfs)	High Flow (cfs)
PU1				
Blind River	2,121	3,881	19,612	36,256
Hope Canal	2,121	3,881	19,612	36,256
LaBranche	138	253	1,209	2,212

Bayou Bienvenue	5,000	9,150	96,000	175,680
Bayou LaLoutre	5,224	9,560	63,000	115,290
Bayou Terre aux Boeufs	2,714	4,967	30,690	56,163
Caernarvon	4,397	8,047	48,525	88,801
Bayou Lamoque	7,912	14,479	22,044	40,341
Grand Bay	971	1,777	10,074	18,435
PU2				
Lagan	2,198	4,022	9,048	16,558
Edgard	967	1,773	10,599	19,396
Naomi	328	600	3,273	5,990
Myrtle Grove	5,240	9,589	64,830	118,639
Port Sulphur	2,757	5,045	34,062	62,333
Buras	1,315	2,406	11,409	20,878
Ft. Jackson	1,122	2,053	15,630	29,152
West Point a la Hache	475	869	5,382	9,849

HYDRAULIC DESIGN OF DIVERSIONS

General

A goal in diverting flow from the Mississippi River is to successfully transport sediment to regions suitable for marsh creation. Location and angle of the diversion is critical for maximizing the sediment load transported from the river to the potential site. A total of seventeen river diversions were designed and placed at various locations along the river. All diversion locations lie between Mississippi R.M 161.9 and R.M. 16. Much of the emphasis was placed upon locating diversions along the river where it would be easiest to reach the intended site. In some cases, it was possible to reach a sediment starved region by diverting water from the river into existing canals or bayous. Four basic diversion designs were implemented including culvert, siphon, Old River Control (ORC), and uncontrolled type diversions. Of the seventeen diversions there were 6 culvert designs, 1 siphon design, 5 ORC designs, and 5 uncontrolled designs.

Methodology

Diversion designs include all pertinent information needed to design the diversion as well length of inflow and outflow channel needed to get the freshwater from the river to the site.
Each diversion was designed to accommodate maximum flow at high river stage. Culvert and ORC type designs implemented the use of the full pipe outlet control flow equation.

$$H = \left(1 + K_e + \left(29n^2 L / R^{1.333}\right)\right) \times \left(V^2 / 2g\right) \qquad \textit{(Refer to TM 5-820-4 pg B-12)}$$

, where H is the head (ft), K_e is the entrance loss coefficient, n is manning's roughness coefficient, L is the length of the structure (ft), R is the hydraulic radius, V is the velocity (ft/s), and g is gravity (ft/s^2).

Culvert design and quantity were determined based upon the above mentioned equation with a manning's n of 0.015 for float finished concrete (TM 5-820-4 pg B-12) and an entrance loss coefficient of 0.5 for a concrete pipe with squared edge.

Siphons were designed based upon the same equation as above with additional head loss coefficients added based upon the number of siphon bends in the design. The modified equation is given below.

$$H = \left(1 + K_e + K_{b1} + K_{b2} + K_{b3} + K_{b4} + \left(29n^2 L / R^{1.333}\right)\right) \times \left(V^2 / 2g\right)$$

where K_{b1}, K_{b2}, K_{b3}, and K_{b4} are the siphon bend loss coefficients. Bends were approximated based upon the existing levee cross section at the given location. Flow through the siphon was assumed to be full. With all critical information, the equation was solved and siphon diameter and quantity were determined.

The last type of design was based upon an uncontrolled diversion (UC). These diversions were designed as earthen channels trapezoidal in shape. The head loss from inflow to outflow was assumed as 0.5 feet and channel slope was calculated based upon this assumption. A 1:3 side slope was assumed in each of the UC type designs. Finally, Manning's equation was used in designing the bottom width of the channel given critical equation variables along with the maximum flow at high river stage.

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

, where Q is the flow (ft^3/s), n is manning's roughness coefficient, A is area of flow (ft^2), R is hydraulic radius, and S is slope of channel.

Inflow/ Outflow Channels

Inflow distance was maintained as the distance from the diversion side of the Mississippi River bank to the diversion outlet measured at the top of levee. Outflow channels are coarser in their approximation since the region cannot be specified over a few feet but over a much greater distance. Containment levees were designed to run along the outflow channels of the uncontrolled diversions. Containment length was determined based upon the slope of the water surface at the start of the outflow channel. Based upon water surface slope, the water was contained until reasonable water elevation was

attained. In some cases the length of containment was kept as the entire length of the outflow channel.

Results

Below is a list of all seventeen diversions including respective sizes and quantities.

Type	Location	Size (H x W)	Number
Culvert	Blind River	15ft x 18ft	5
Culvert	Hope Canal	15ft x 16ft	6
Culvert	Lagan	14ft x 15ft	3
Culvert	Edgard	15ft x 18ft	3
Culvert	Naomi	15ft x 20ft	1
Culvert	West Pt. a la Hache	12ft x 16ft	3

Type	Location	Size (Diam.) – Mat.	Number
Siphon	LaBranche	6ft – RCP	4

Type	Location	Size (H x W)	Number
ORC	Bayou Bienvenue	30ft x 52ft	5
ORC	Bayou LaLoutre	30ft x 45ft	4
ORC	Bayou Terre aux Boeufs	30ft x 44ft	2
ORC	Caernarvon	30ft x 46ft	3
ORC	Myrtle Grove	25ft x 46ft	5

Type	Location	Size (BW – S)	Number
UC	Bayou Lamoque	90ft – 0.00034ft/ft	1
UC	Grand Bay	25ft – 0.00024ft/ft	1
UC	Port Sulphur	320ft – 0.00012ft/ft	1
UC	Buras	25ft – 0.00027ft/ft	1
UC	Fort Jackson	100ft – 0.00018ft/ft	1

Notation:
RCP = Reinforced Concrete Pipe
BW = Bottom Width

STRUCTURES

General

Hydraulic design information such as project location, structure type, structure size (number of openings, size of openings, length of culvert) was provided for all controlled diversions. These diversions fell into three categories of projects: large diversion (Old River Auxiliary Structure type); box culverts (Davis Pond type); and siphons (Hero Canal

type). Figures 6 to 8 show typical diversion structures of the three types. The main cost estimate items were taken from the feasibility reports for each of the three projects, shown above in parentheses. The quantities for each of these items were determined based on the following assumptions:

- natural ground elevation taken from available LIDAR
- Mississippi River flood protection elevation based on nearest river mileage
- all excavations have 1V on 6H side slopes

Rudimentary designs were performed to establish the nominal dimensions of major structural components and high cost items. These rudimentary designs were based on proven concepts and historical data. Items deemed as not critical to the overall estimate were sized by engineering judgment, relying heavily on the experience of the designer.

Box Culverts

The structure used for these project features was modeled after the Davis Pond Diversion Structure. The diversion structure is compromised of inflow monoliths, a gate monolith, box culvert monoliths, a downstream bulkhead monolith, and outflow monoliths. A cofferdam will be required to maintain the Mississippi River flood protection during construction of the structure.

Large Diversions

The structure used for these project features was modeled after the Old River Auxiliary Structure. The structure will be a non-navigable pile supported, tainter gated structure. The tainter gates are similar to the gates used in the Old River Auxiliary Structure. A cofferdam will be required to maintain the Mississippi River flood protection during construction of the structure.

Siphon

The structure used for this project feature was modeled after the Hero Canal Diversion Structure. The project feature consists of a siphon which crosses over the Mississippi River levee, crosses under a highway, and discharges into the marsh.

Figure 6: Typical Box Culvert Structure

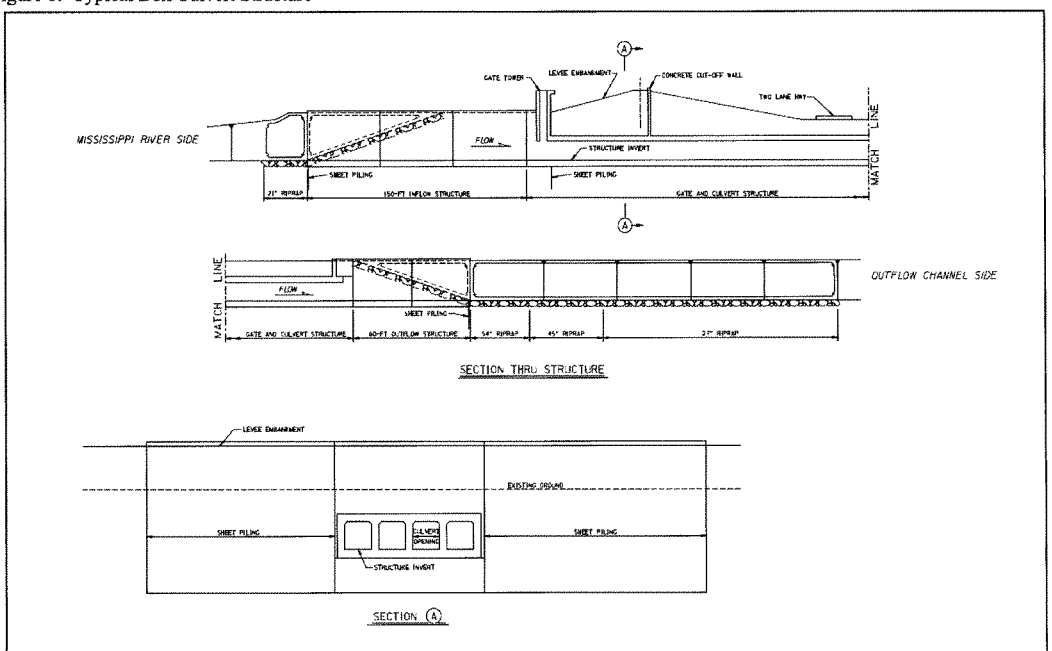

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Figure 7: Typical Large Diversion Structure

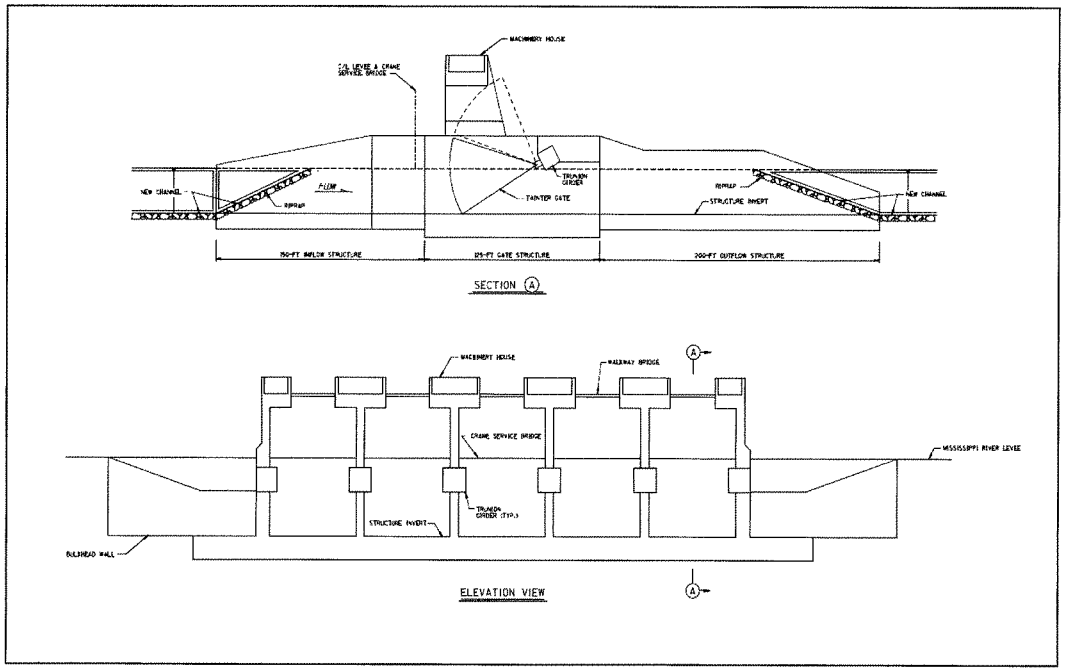

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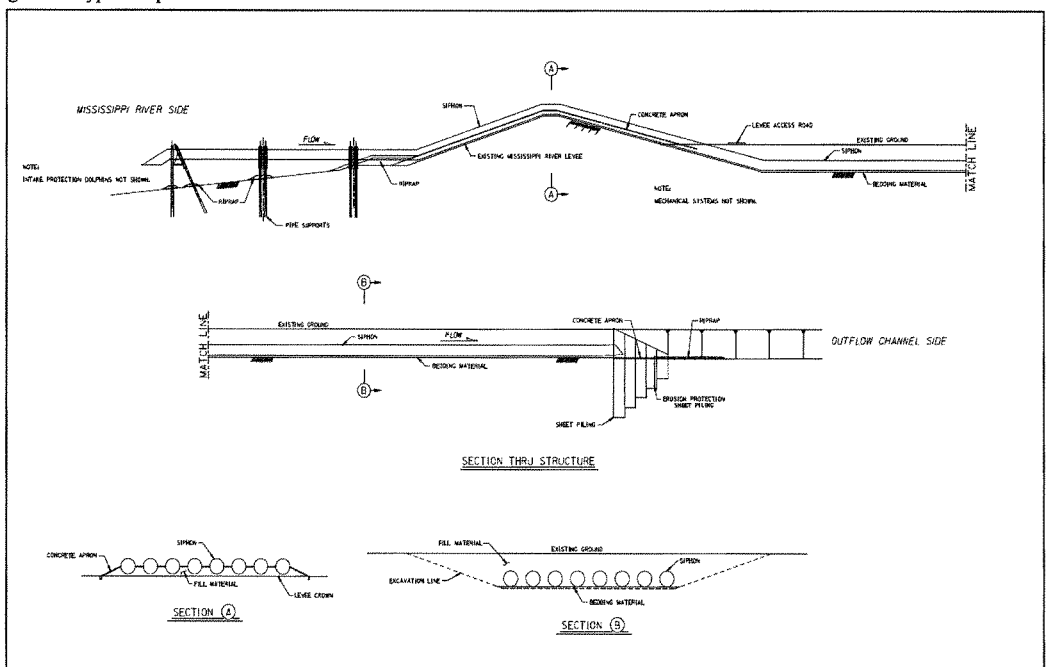
Figure 8: Typical Siphon Structure



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CHANNELS

Design Methods for Outflow Channels

Hydraulic design information including channel inverts, bottom width and length was provided for each of the proposed diversions. Plates 1 to 17 show the structure and channel location and channel cross section for each diversion. The diversion channels will be mechanically or hydraulically dredged, depending on the location of the diversion channel. The channels will be cut to the depth of the outflow inverts of the culverts. The alignment of the outflow channel will run from the diversion structure to the marsh. Cut material from the channel shall then used to create adjacent levees on each side of the channel. The levees will be offset 50' from the edge of the channel to the toe of the leaves. The levees will have 4:1 slopes with the dimensions of the levees calculated to use all of the cut material from the channel. The levees will then be capped with 2.5' of clay brought in from borrow pits. The levees and the slopes of the channel are to be hydro seeded once construction is completed. Specific channels were designated by the hydraulic design to have rip rap armored slopes. The following assumptions were made prior to the design of the diversion channels:

1. Borrow for the 2.5' clay caps will be brought in from borrow pits located within 30 miles of the diversion channel location. The exact location of the pits will be determined during detailed design.
2. Unless specified by the hydraulic design information 1,000ft armor lengths for the diversion channels were used.

MARSH CREATION

GENERAL

The proposed features include an extensive list of marsh creation and restoration. Marsh creation elements were included in all planning units. Each element is a combination of open water and deteriorated (broken) marsh. As surveys of each element were not performed for this analysis, an assumed lift was used for open water areas, deemed marsh creation, and for deteriorated marsh, deemed marsh nourishment. Quantity needed to fill elements within each planning unit is included in table 3 below. Required acreage was determined from maps developed by the HET.

Table 3 – Marsh Creation Borrow Requirements

Planning Unit 1	Acreage	% Open Water	Borrow Needed
East New Orleans Landbridge	42,590	100%	178,633,000cy
Biloxi Marshes	124,440	100%	655,983,000cy
Bayou Terre aux Boeufs	12,190	100%	127,822,000cy

Breton Sound Landbridge	31,900	100%	334,661,000cy
Caernarvon Area	17,780	100%	186,453,000cy
subtotal PU1	**228,900 acres**		1,483,552,000cy
Planning Unit 2			
Strategic Lower Basin	52,385	50%	146,000,000cy
Planning Unit 3a			
Pipeline Conveyance	156,250	100%	1,638,532,000cy
duLarge to Grand Caillou Landbridge	2,920	100%	30,611,000cy
South Caillou Lake Landbridge	27,850	100%	291,956,500cy
Subtotal PU3a	**187,020**		**1,961,095,500cy**
Planning Unit3b			
Bayou Decade	12,419	100%	100,179,933
Brady Canal	9,351	100%	75,431,400
Pointe au Fer	9,619	100%	77,593,000
Marsh Island	18,525	100%	149,432,820
Wax Lake Outlet Delta	4,920	100%	39,688,000
Bayou Penchant Area	13,182	100%	106,334,800
Terrebonne GIWW	12,733	100%	102,712,867
subtotal PU3b	**33,203 acres**		268,000,000cy
Planning Unit 4			
Mud Lake	9,625	60%	68,516,331
South Grand Chenier	5,570	80%	46,728,587
South Pecan Island	7,993	100%	77,372,240
East Pecan Island	12,123	50%	78,233,760
No Name Bayou	6,697	40%	38,896,176
NW Calcasieu Lake	24,280	100%	235,030,400
East Calcasieu Lake	12,086	100%	116,992,480
Black Bayou	13,315	70%	103,111,360
Gum Cove	5,356	70%	41,476,864
Cameron Meadows	3,736	50%	24,109,653
Central Canal	5,355	80%	44,924,880
subtotal PU4	**106,163 acres**		1,404,958,948cy
Total all planning units			

The major factor in the cost for marsh creation and nourishment is location of borrow. Borrow is limited for the enormous quantity needed to complete all planning units.

Sources included in this design include maintenance (O&M) quantities from federal channels, lakes, bays, and the Gulf of Mexico.

a. O&M quantities were calculated from current dredge cycles. Each channel has an average quantity and time between dredge cycles. Most channels have an ongoing beneficial use program. This project would pull all material from those programs. No material was left in the cycle to complete ongoing beneficial use of materials dredged from the O&M program. In addition, reductions in maintenance quantities may occur by stabilizing banks and redirecting channels as required by other elements within this project. For example, widening and deepening the Wax Lake Outlet or armoring the GIWW or Freshwater Bayou may result in less maintenance dredging. The magnitude of these reductions was not estimated at this level of analysis. Lastly, the available quantity in the dredge cycle did not consider addition material available for deepening or widening the existing authorized dimensions.

b. Many lakes were considered for borrow. Borrow from interior lakes was limited to 20% of the existing lake bottom.

c. Many bays were considered for borrow. Borrow from bay areas was limited to 10% of the existing bay bottom.

d. Due to the fact that borrow required exceeds borrow available from O&M, lakes, and bays, the Gulf of Mexico was added as a borrow source. Borrow from the Gulf of Mexico will impact certain marsh creation sites due to high salinities in both the borrow and slurry water.

Many marsh creation sites require multiple boosters and miles of discharge pipe. As such, an access corridor is included for some sites. Access corridor for booster, pipeline, and work boats includes an 80ft bottom width at elevation (-)8.0ft. Access corridors excavated from the Gulf shall be backfilled and armored when it intercepts existing gulf side shoreline. Access corridors from the Gulf to be backfilled and armored include Pecan Island and Grand Chenier.

Dredge quantities are well beyond normal magnitudes for existing large dredge contracts. Due to the time and cost to prepare access for these sites, a larger contract must be approved. For the purpose of this report, a maximum contract of 3 years and 40,000,000cys was used.

Costs do not include planting the newly constructed/nourished marsh elements. Future submittal will include environmental input into planting scheme. It is also assumed that bio-accumulation of newly constructed marsh will maintain marsh elevation. No additional lifts are included on any marsh creation element.

SHORELINE PROTECTION

GENERAL

Three separate and distinct shoreline protection features were evaluated under this effort:

(1) Shoreline protection along the gulf coastline within planning units 4 and 3b

(2) foreshore protection along both banklines of the GIWW within planning unit 4

(3) nearshore protection along the entire perimeter of Grand and White Lakes not currently stabilized by previous stone construction efforts.

The section of coastline between Peveto Beach and Holly Beach was excluded since shoreline protection already exists in that section. Stabilization measures were included in order to decrease gulf shore erosion rates and to prevent the inward degradation of wetlands in order to reduce loss rates and enhance the sustainability success of other restoration measures.

PLANNING UNITs 3a AND 3b

Atchafalaya and Chenier Plain Gulf Shoreline Stabilization

The design for the breakwaters were based on conventional and proven existing designs. This shoreline protection was estimated based on conventional stone structures. The Grand Isle offshore segmented breakwater template was used as the basis for offshore breakwaters. The cross section has a five foot crown at elevation +4.0, 1 vertical on 2 horizontal side slopes, and was aligned along the elevation -6.0 contour (figure 9). For initiation of this planning cost estimate, segments are anticipated to be 200 feet long with 300 foot gaps between segments. Approximately 98 miles of proposed breakwater overlaid on geotextile fabric is proposed under this study evaluation.

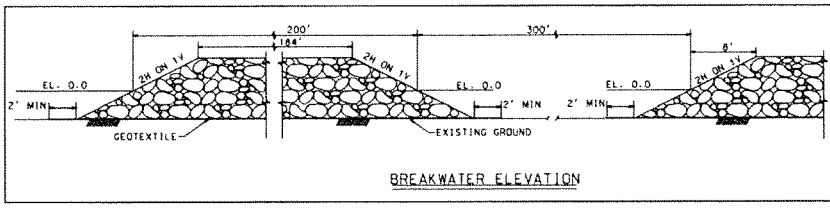


Figure 9: Gulf Shoreline Stabilization Section

The area was divided into reaches and geotechnical estimates of construction losses for each reach were provided. Structure side slopes and construction losses due to foundation conditions were estimated based on available foundation data, general geology

information, and experience in the coastal area. Site specific surveys, hydraulic analysis, and geotechnical analysis will be required to refine this proposed design. Table 4 shows the reach, estimated length, and construction losses used for estimating purposes.

Table 4 - Assumed Length and Construction Loss for Measure

Reach	Length (LF)	Construction Loss
Stone Breakwaters – Freshwater Bayou to Southwest Pass	58,646	50%
Stone Breakwaters – Freshwater to Rollover	72,153	50%
Stone Breakwaters – Rollover to Mermentau	160,560	45%
Stone Breakwaters – Mermentau to Calcasieu	107,169	40%
Stone Breakwaters – Calcasieu to Holly Beach	45,673	35%
Stone Breakwaters – Perveto Beach to Sabine	74,770	30%

PLANNING UNIT 4

Near Shoreline Stabilization, Grand and White Lakes

Stone shoreline protection along the entire rim of Grand and White Lakes is proposed to halt degradation of the shoreline and breaching into existing marsh and upland areas as a result of wind induced wave energy. A successful CWPPRA Project ME-22 located along the southern shoreline of White Lake from Will's Point to the western shore of Bear Lake in Vermilion Parish, Louisiana, included constructing segmented breakwaters to protect approximately 61,500 linear feet of shoreline to protect 687 acres of shoreline and interior marsh (figure 10). The breakwaters were constructed with gaps to allow aquatic organisms and water to move freely. Material dredged to create a flotation channel was placed beneficially behind the rock dike to create marsh substrate.

Figure 10: White Lake Shoreline Protection Completed Aug 2006

This shoreline protection design was used as the basis for estimating required lake shoreline protection. The breakwater would be located along the –1.5 foot NAVD 88 contour in approximately 2.0- to 3.0 feet of water, stage dependant. The dike crown would be 4.0 feet wide and would be set at an elevation of +3.5 feet NAVD 88 (figure 11). The breakwater would have side-slopes of 1.0 vertical on 1.5 horizontal. Gaps for fish access would be built at approximately 1,000-foot intervals, with a top width of 50 feet. Approximately 106 miles of proposed shoreline is estimated under this study effort.

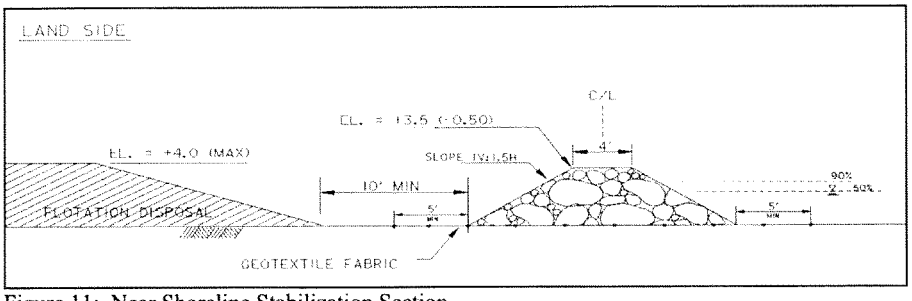


Figure 11: Near Shoreline Stabilization Section

Channel Stabilization, GIWW

Foreshore dike construction is proposed along both banks of the GIWW, from Freshwater Bayou to the Calcasieu River to halt shoreline erosion and widening of the channel. The rock dike would be aligned in the vicinity of the waters edge, assuring safe navigation and continued usage of the authorized channel. The template for this rock design would consist of a continuous nearshore dike section with a 4' crown, a 1.0 vertical on 2.0 horizontal channel side slopes, and a 1.0 vertical on 1.5 landside slope. The flatter channel side slope will be incorporated to minimize wave rollup and reduce maintenance requirements. The section will likely be tucked into the shoreline to minimize rock

quantities while maximizing the navigation safety zone. Fetch driven waves are not likely a concern within this inland waterway. A crown height in the vicinity of +3.0 to +5.0 will be established as required to break vessel waves. The rock dike would be overlaid upon a base geotextile fabric. Approximately 205 miles of shoreline protection is estimated under this planning study.

CONCERNS

Significant concerns remain that have to be evaluated prior to final selection of a preferred plan of restoration. As previously stated, general assumptions were made regarding geotechnical and bathymetric conditions across these wide arrays of worksites. Site specific data collection will be required to verify settlement requirements and existing site conditions. Unanticipated stability berms, if required, as well as increases in anticipated settlement would increase these estimated rock quantities. Construction sequencing and duration will have to be further analyzed to assure adequate availability of materials and construction equipment. Access considerations as well as existing obstructions to construction will have to be considered during final plan selection. Construction production rates in the Gulf of Mexico will certainly be weather dependent. In addition this extensive proposal of rock structure construction will likely result in associated long-term maintenance needs not addressed in this report.

OTHER MEASURES

GENERAL

Some of the proposed measures have been investigated under other studies and programs such as the Louisiana Coastal Area (LCA), Ecosystem Restoration Study, the Coastal Wetlands Planning Protection and Reatoration Act (CWPPRA) and Louisiana's Comprehensive Master Plan for a Sustainable Coast. For these measures the existing cost estimates were used and were updated when necessary. These measures are described below.

PLANNING UNIT 3a

IHNC Lock Multi-purpose Operation

The measure would include development and implementation of a multi-purpose operational plan for the Houma Navigation Lock that would allow the lock to be utilized for coastal restoration purposes by making more efficient use of Atchafalya River water and sediment flow and maintaining salinity regimes favorable for wetlands in the Lake Boudreaux, Lake Mechant and Grand Bayou areas. The lock is proposed to be located on the Houma Navigation Canal (HNC), 1.75 miles south of the Bayou Sale intersection and is an integral part of the Morganza to the Gulf Hurricane Protection Project.

Costs associated with this measure would consist of additional O&M costs for the lock. There would be no modifications to the lock structure required. Estimates for O&M cost

were taken from costs developed for Louisiana's Comprehensive Master Plan for a Sustainable Coast (Apr 07).

Convey Atchafalaya River Water via the GIWW

This measure includes increased flow of Atchafalaya River water via the Gulf Intracoastal Waterway into Terrebonne marshes from an area west of the Avoca Island Levee for freshwater and sediment delivery to enhance and sustain wetlands.

This feature would produce enhanced areas for both the Larose to Houma reach and Houma to Morgan City reach through feature components such as a small diversion in the Avoca Island levee, repairing eroding banks of the GIWW, enlarging constrictions in the GIWW below Gibson and in Houma, and Grand Bayou conveyance channel construction/enlargement. Cost for this feature were developed for the Louisiana Coastal Area (LCA), Ecosystem Restoration Study and were updated to current price levels.

Lapeyrouse Canal Diversion

The project area is experiencing marsh deterioration due to subsidence, rapid tidal exchange, and human-induced hydrologic changes that result in increased salinities. Saltwater intrusion has caused a shift in marsh type and a conversion of over 30 percent of emergent vegetation to open water habitat. Shoreline erosion along the south embankment of Lake De Cade threatens to breach the hydrologic barrier between the lake and interior marshes.

Proposed project components include installing three control structures along the south rim of Lake De Cade and enlarging Lapeyrouse Canal to allow the controlled diversion of Atchafalaya River water, nutrients, and sediments south into project area marshes. Outfall management structures are planned in the marsh interior to provide better distribution of river water. In addition, approximately 1.6 miles of foreshore rock dike is planned to protect the critical areas of the south lake shoreline from breaching (see figure 12). This project was selected as a part of CWPPRA Priority Project List 9. The cost estimate for this measure was taken from the Aug 08 CWPPR Project Manager's Technical Fact Sheet.

Upper Lake Boudreaux Basin Mississippi River Diversion, East Terrebonne Mississippi River Diversion, Grand Bayou & Jean LaCroix Basin Mississippi River Diversion

This measure is similar to the proposed Third Delta Project (diversion of Mississippi River water into the Terrebonne Basin by constructing a conveyance channel following the eastern slope of the natural Bayou Lafourche levee system and splitting into two channels near Raceland) except that the channels would not go as far south. Figure 13 shows the conceptual plan. A design and cost estimate were developed for the Third Delta Plan as part of the LCA study. This cost has been updated and modified to take into account the differences in the two plans.

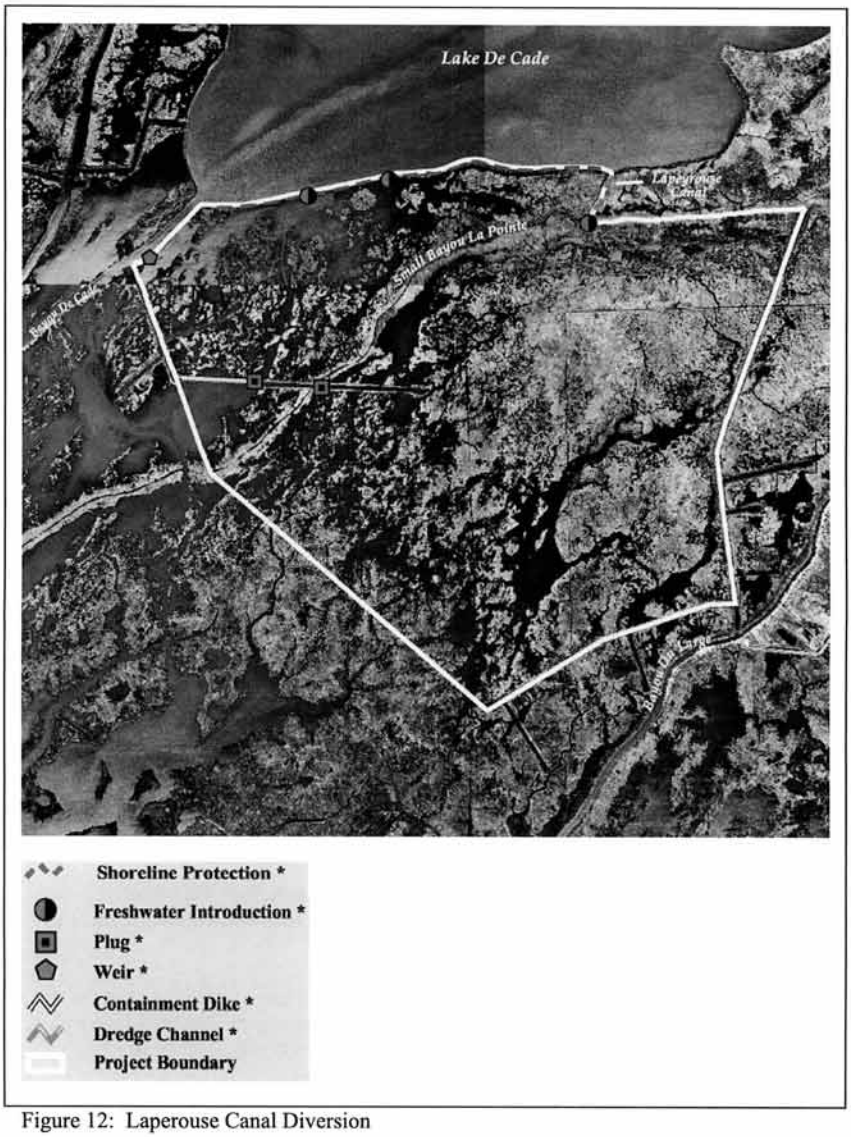


Figure 12: Laperouse Canal Diversion

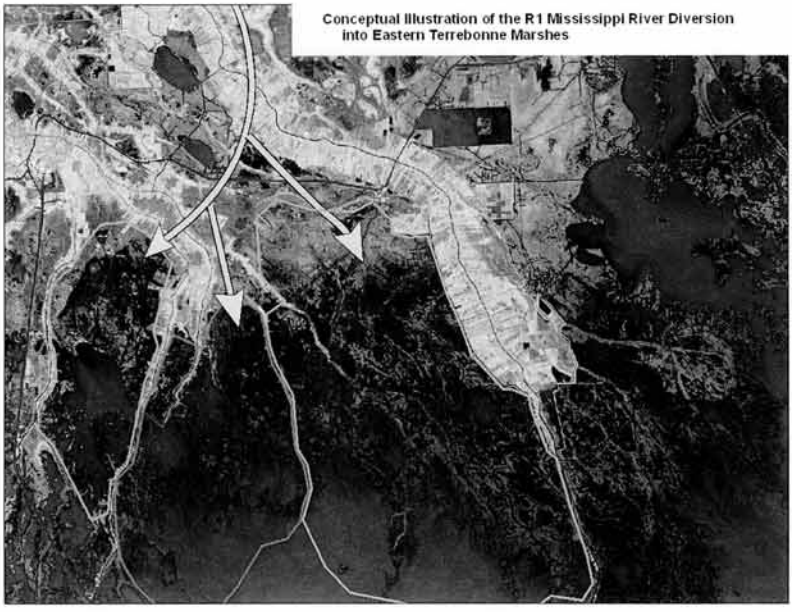


Figure 13: Mississippi River Diversion into Terrebonne

Blue Hammock Diversion

Grand Pass is a major tidal exchange point through the Bayou du Large ridge, which carries higher salinity water directly from Sister Lake into Lake Mechant and the surrounding marshes. Historically, this pass did not cut straight through the ridge and was a less efficient channel than it is now. Periodic increases in salinity in Lake Mechant are contributing to the loss of intermediate and brackish marshes in the basin. Freshwater input into the basin currently comes through the Gulf Intracoastal Waterway (GIWW) and the Penchant system on the north and Atchafalaya River input from Four League Bay via Bayous Carencro and Blue Hammock Bayou. However, the size of Blue Hammock Bayou decreases substantially west of Lake Mechant, which limits the easterly flow of sediment-laden river water into Lake Mechant and surrounding marshes.

The proposed project consists of the following components: construct a weir in Grand Pass; construct a weir in Buckskin Bayou; dredge Blue Hammock Bayou to increase the cross section; place 5 plugs in smaller channels; use material dredged from Blue hammock Bayou to restore surrounding marsh. These components can be seen in figure 14. Estimates were taken from costs developed for Louisiana's Comprehensive Master Plan for a Sustainable Coast (Apr 07).

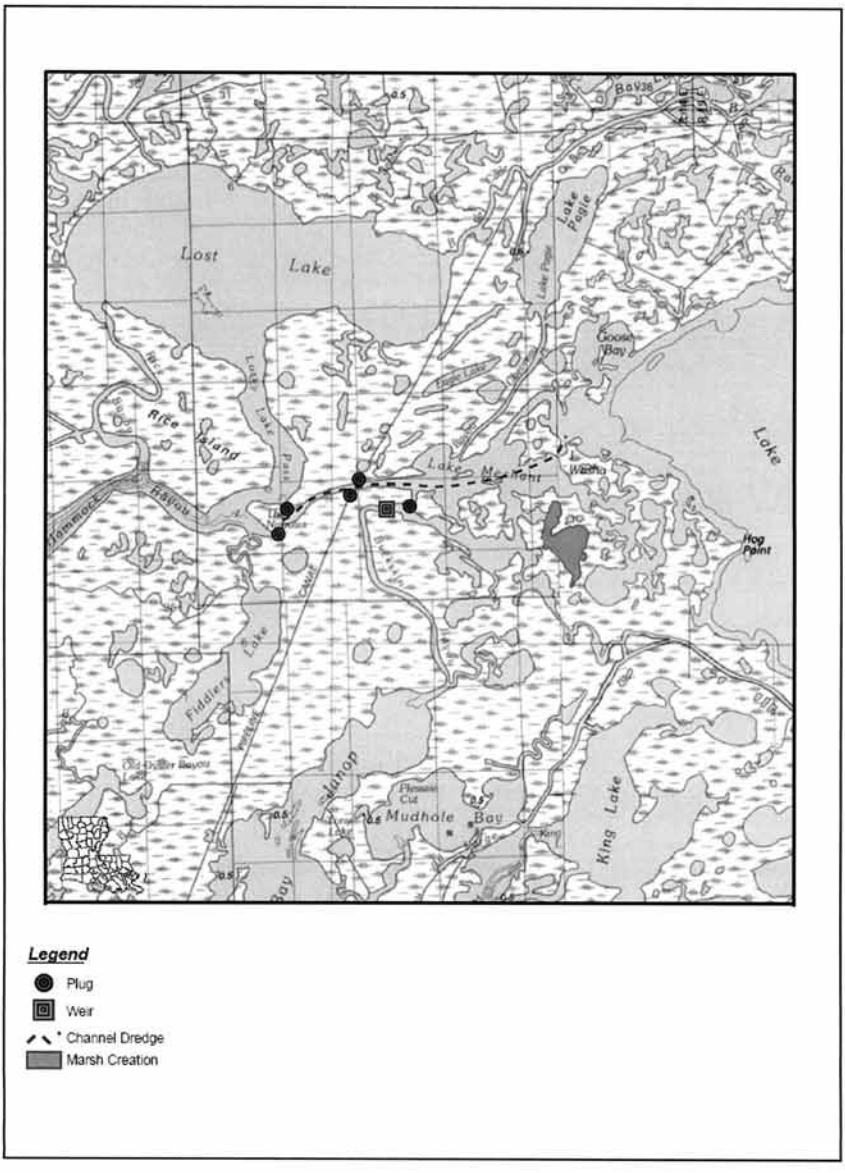


Figure 14: Blue Hammock Bayou

PLANNING UNIT 3b

Penchant Basin

Area problems include major hydrologic alterations, interior marsh erosion, subsidence, saltwater intrusion, herbivory, and hurricane damage. This project will combine the long-term realignment of Penchant Basin hydrology with restoration and protection measures aimed at maintaining the physical integrity of the area during the transition toward greater riverine influence.

Proposed project components include: a rock weir with a barge bay in the northern end of Big Carencro Bayou at its intersection with Bayou Penchant; a steel sheet-pile weir with variable crest sections and flapgates in the Bayou Mauvais Bois at its intersection with the Superior Canal; rock bank stabilization; dredging and marsh creation at the mouth of Bayou Penchant; a rock weir with a barge bay at the southern shoreline of Raccourci Bay; maintenance of existing weirs along Bayou De Cade; shell plugs with rock rip-rap cover along Bayou De Cade; three steel sheet-pile variable crest weirs along Bayou De Cade; two steel sheet-pile variable crest weirs with boat bays along Bayou De Cade; a rock liner in Little Deuce Bayou at its intersection with Bayou De Cade; a rock weir with barge bay in Bayou la Loutre at its intersection with the Superior Canal; a steel sheet-pile weir with boat bay and variable crest sections in Brady Canal at its intersection with Bayou Penchant; an earthen bank stabilization along Bayou De Cade; and bank maintenance. These features are shown in figure 15. Project cost was taken from the CWPPRA Project Manager's Technical Fact Sheet dated August 08.

Relocate the Navigation Channel through Lower Atchafalaya River Delta

This plan consists of relocating the end of the Atchafalaya River navigation channel through Shell Island Pass and to the West (see figure 16). The cost estimate was taken from the estimates done for the LCA study and updated to current price levels.

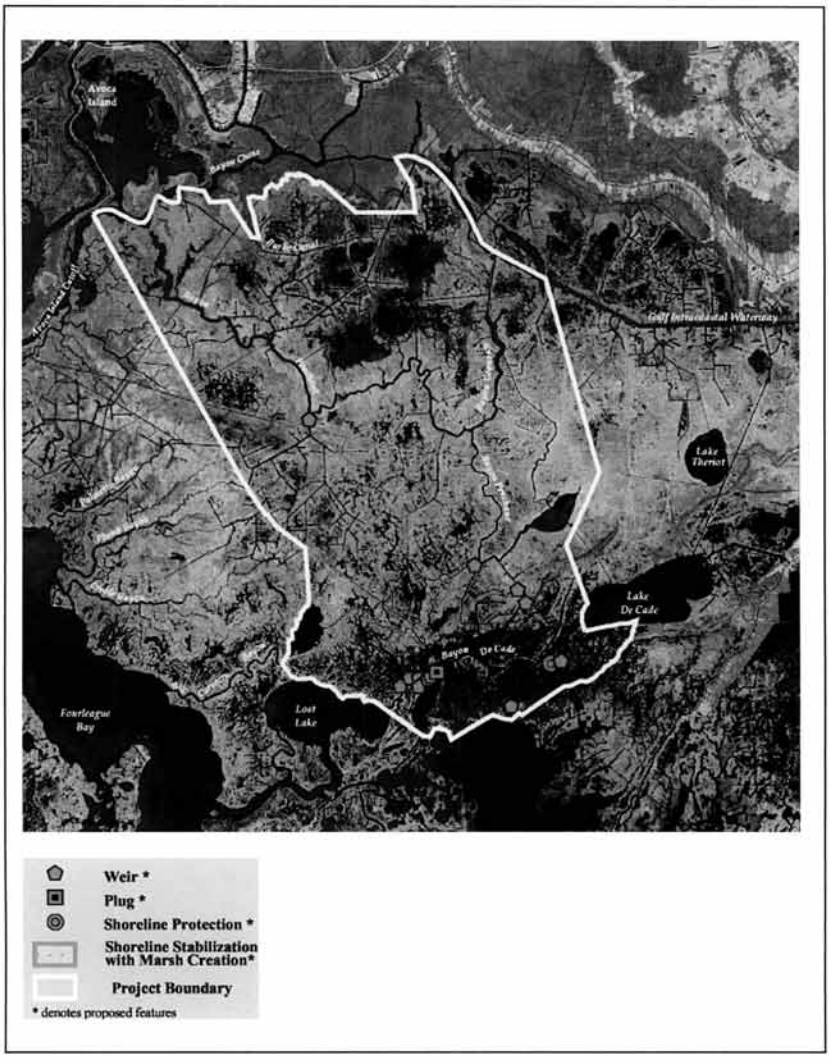


Figure 15: Penchant Basin Plan

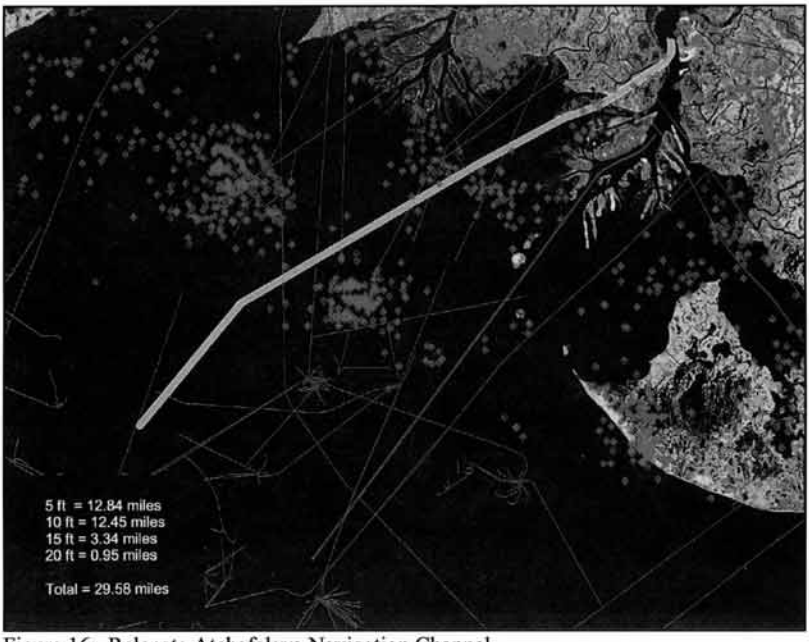


Figure 16: Relocate Atchafalaya Navigation Channel

Increase Sediment Transport Down Wax Lake Outlet

Wax Lake Outlet is located in St. Mary Parish at the eastern side of East Cote Blanche Bay. The outlet discharges water and sediment from the Atchafalaya River into a forming delta. The measure is to relocate the alignment of the Wax Lake Outlet inflow channel to a more direct route to the Atchafalaya River channel with a deeper inflow channel, capturing additional sediment with the deeper inflow channel to be deposited in the delta of Wax Lake (see figure 17). The proposed new channel would allow flow from the Lower Atchafalaya to by-pass Six Mile Lake. The proposed new channel would begin about mile 110 of LAR and extends to range 0.0 on WLO. The channel would be cut to an elevation of -38.0 feet with a bottom width of 800 feet and side slopes of 1 vertical on 5 horizontal. This measure was included as part of the Lower Atchafalaya Basin Reevaluation Study and the LCA study. The cost estimate was taken from estimates done for the LCA Study and updated to current price levels.

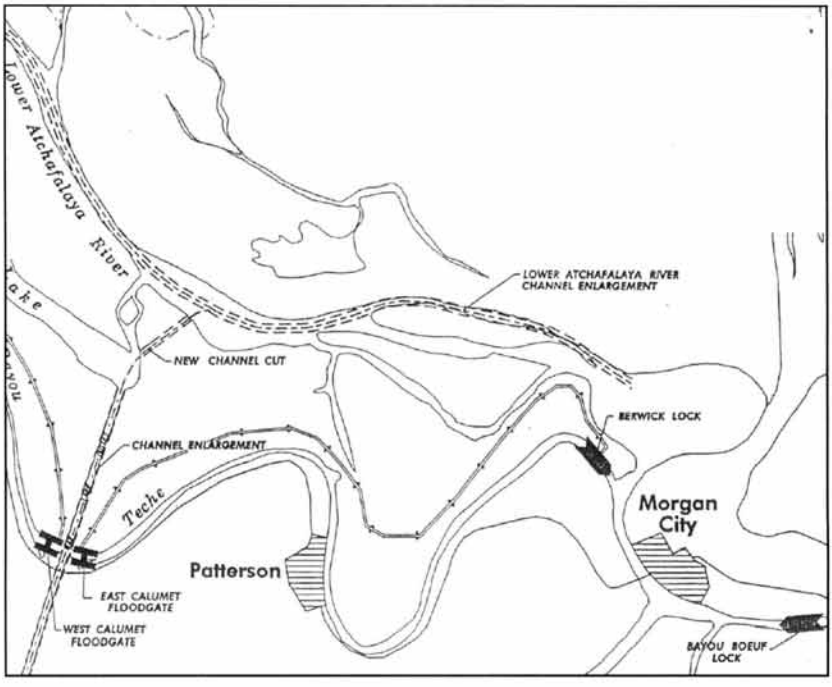


Figure 17: Wax Lake Outlet

Shoreline Protection

Freshwater Bayou Bank Stabibization,

The stabilization is located on the eastern bank of the Freshwater bayou Canal between Freshwater Bayou Canal and Belle Isle Bayou in Vermilion Parish. The canal bankline is experiencing significantly erosion from boat traffic in the canal. A 40,000-foot-long near-bank rock dike is to be constructed. The dike will be continuous except for openings left at the mouths of several oil-well canals where the dike will be tied into the bank on both sides of each canal.

Southwest Pass Bank Protection

Southwest Pass is an open channel located between the Gulf of Mexico and Vermilion Bay. The Pass has experienced significant erosion and is more than 10 meters deep. The features would be a total of 9.0 miles of stone dike along the top bank of either side of the present channel (4.5 miles plus 4.5 miles). Dikes would be 6-feet high, 3-feet in

water and 3-feet above water. A 500-foot section on the west bank would be 33-feet in the water and 3-feet above water.

Marsh Island Shoreline Protection

The site is the gulf shoreline between the east bank of Freshwater Bayou (west end of Marsh Island) and South Point (eastern tip of Marsh Island). Stabilization would minimize loss of this gulf shoreline. Dredged sediment from off shore would be source material for this project. Material would be placed just into the marsh at 6 to 12-inches deep in thin layers.

Shoreline Protection and Vermillion Bay and West and East Cote Blanche Bays

This measure would provide shoreline protection at strategic locations in Vermilion Bay, West Cote Blanche Bay, and East Cote Blanche Bay using hard structure detached breakwaters.

Gulfshore Protection at Point Au Fer Island

Gulf shoreline of Point Au Fer Island is located in the southeastern area of St. Mary Parish, south and slightly east of the mouth of the Atchafalaya River. The stabilization will minimize annual losses along the gulf shoreline of the island. Stone segments 250-feet in length, 5-feet in height and 5-feet in top width with 50-foot gaps between segments will be placed in water two-feet deep approximately 100 feet from shoreline.

Estimates for all theses shoreline protection features were taken from costs developed for Louisiana's Comprehensive Master Plan for a Sustainable Coast (Apr 07).

BARRIER ISLANDS

Several studies have been and are being conducted on restoration of the Barataria and Terrebonne basin barrier islands. These include the Louisiana Coastal Area Ecosystem Restoration Study (LCA), the Barrier Island Feasibility Study and the ongoing Barataria Basin Barrier Island Feasibility Study. These existing costs and designs were used for this effort. Costs were updated to current price levels where necessary. Figures 18 and 19 show the location of the Barataria and Terrebonne barrier islands

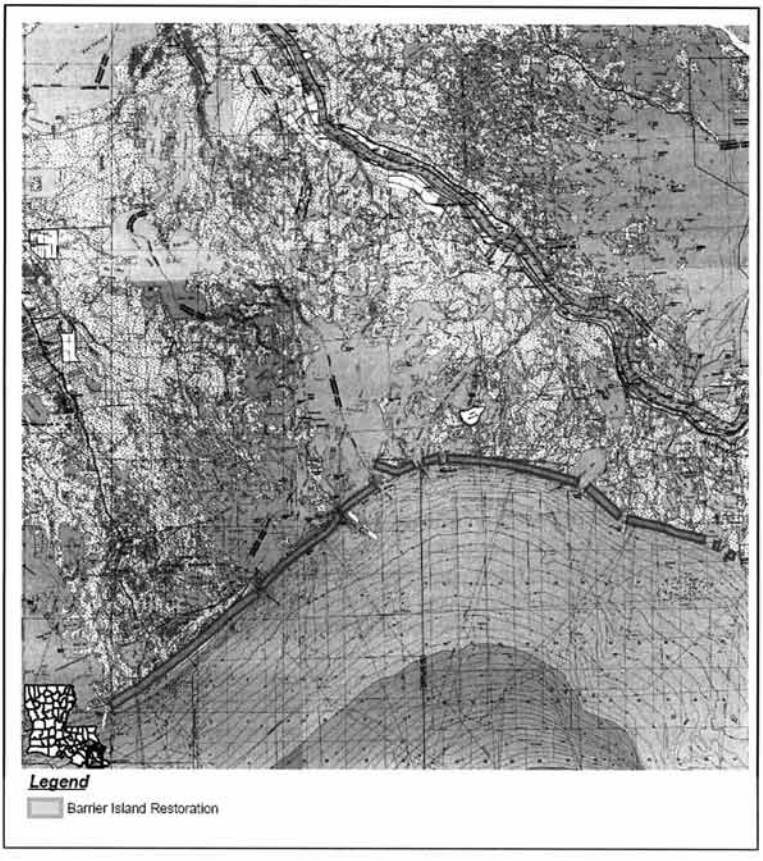


Figure 18: Barataria Basin Barrier Islands

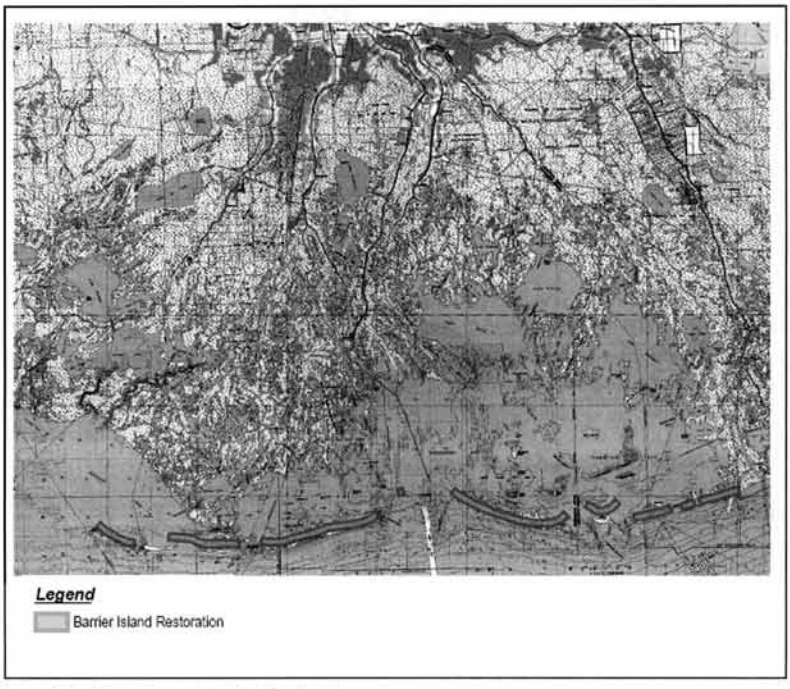


Figure 19: Terrebonne Basin Barrier Islands

COST ESTIMATES

GENERAL

Costs used for the MCDA analysis were developed by the HET based on costs for existing marsh creation, shore protection and diversion projects. These costs are included in the alternative costs shown in attachments 6 to 13 in Annex 2.

Subsequent to this analysis a detailed cost for each measure was developed for determining the cost for the final array of alternatives. The following discussion pertains to these more detailed estimates. The estimated costs were based upon an analysis of each line item evaluating quantity, production rate, and time, together with the appropriate equipment, labor and material costs. All of the construction work is generally common to the New Orleans District. Given the magnitude of this study, access to the construction sites is assumed to be available, either by land or water or both depending on the exact location of the feature.

Given the magnitude of the overall project, large quantities of equipment are assumed available based on adequate funding levels existing.

The costs prepared for and presented in this report are not to be used for making funding decisions. They are sufficient in detail for comparison of plans for initial screening, for use in performing MCDA and defining costs for the final array of alternatives. A more detailed cost estimate per regulation will be generated in the future upon completion of additional engineering analysis.

DIVERSIONS

The diversions were based on similar type structures previously built within the New Orleans District (Davis Pond Diversion and Old River Auxiliary Control Structure). Construction technique is basic to all large concrete structure construction. Costs for the structures were developed using recent historical data and by ratioing and indexing features from the original template structures. The project costs also include the channel excavation and channel rock armoring for the diversion structures. The channel material will either be placed adjacent for future levees or hydraulically dredged and pumped to create marsh. Major materials are assumed to be hauled in by truck or barged or combination.

MARSH CREATION

Marsh creation work was based on using standard industry techniques and equipment operating 24 hours per day 7 days per week. Due to the magnitude of the jobs and potential working environments, it is assumed the larger hydraulic dredges (cutterhead and/or hopper dredges) will be used for excavation and transporting material from borrow site to marsh creation areas. A general cost per distance method was used when specific placement-borrow site combinations were unknown along the Gulf area. Borrow sources include existing O&M, Mississippi River, Gulf of Mexico, and existing bays and lakes.

OTHER MEASURES

As stated above estimates for some of the proposed measures were originally developed for other efforts such as the Louisiana Coastal Area Ecosystem Restoration Study (LCA), the Coastal Wetlands Planning Protection and Restoration Act (CWPPRA) and Louisiana's Comprehensive Master Plan for a Sustainable Coast. For these measures an escalation factor was applied to update the existing cost estimates. This escalation was determined by using EM1110-2-1304, Civil Works Construction Cost Index System (CWCCIS).

CONTINGENCY

This study document represents many projects and alternatives that are very preliminary in nature, yet it was decided that it was necessary to present a contingency to better portray a likely cost value for the final array of alternatives. Since the projects are very

preliminary, a risk-based study of the many alternatives would likely provide no better value or confidence in a contingency calculation. After much consideration, it was accepted that the contingency should be a value similar to the risk-based contingencies calculated on the GNO HSDRRS program. For this reason, a value of 50% was chosen as the likely value that can be supported through comparable studies on a similar program.

COASTAL ALTERNATIVE COST

Tables 5-9 show the first costs of each measure by Planning Unit. These costs apply to the low sea level rise scenario. Similar costs were developed for the high sea level rise scenario. These costs are combined with costs for other plan components (structural and nonstructural) to develop estimates for the final array of alternatives.

Table 5 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU1

Measure #	Measure	Cost ($M)
2-1	Blind River Diversion	268.3
2-2	Hope Canal Diversion	311.3
2-3	Labranche Diversion	122.0
2-4	Bayou Bienvenue Diversion	331.6
2-5	East New Orleans Land Bridge Marsh Creation	1,851.2
2-6	Bayou LaLoutre Diversion	722.2
2-7	Biloxi Marshes Shore Protection	225.1
2-8	Biloxi Marshes Marsh Creation	10,726.4
2-9	Bayou Terre aux Bouefs Diversion	289.8
2-10	Bayou Terre aux Bouefs Marsh Creation	(included in 2-11)
2-11	Breton Sound Strategic Land Bridge	20,928.9
2-12	Caernarvon Diversion	324.3
2-13	Caernarvon Area Marsh Creation	(included in 2-11)
2-14	Bayou Lamoque Diversion	86.8
2-15	Grand Bay Diversion	29.5
TOTAL:		36,217.4

Table 6 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU2		
Measure #	Measure	Cost ($M)
2-1	Lagan Diversion	256.2
2-2	Edgard Diversion	223.1
2-3	Davis Pond Freshwater Diversion Reauthorization	2.2
2-4	Naomi Diversion	119.4
2-5	Myrtle Grove Diversion	316.3
2-6	Strategic Marsh Creation in Lower Basin	1,675.6
2-7	North Bay Rim Marsh Creation /Protection	6,404.1
2-8	West Point a la Hache Diversion	130.7
2-9	Port Sulphur Diversion	54.9
2-10	Buras Diversion	42.6
2-11	Fort Jackson Diversion	46.5
2-12	Barrier Island Restoration	786.8
TOTAL:		10,058.4

Table 7 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU3a		
Measure #	Measure	Cost ($M)
1-1	HNC Lock Multi-purpose Operation	0
1-2	Convey Atchafalaya River Water via the GIWW	329.6
1-3	Lapeyrouse Canal Diversion	6.8
1-4	Blue Hammock Diversion	43.6
1-5	Upper Lake Boudreaux Basin Mississippi River Diversion	4,296.5
1-6	East Terrebonne Mississippi River Diversion	(included in 1-5)
1-7	Grand Bayou & Jean LaCroix Basin Mississippi River Diversion	(included in 1-5)
1-8	Pipeline Conveyance Marsh Creation	53,965.2
1-9	North Terrebonne Bay Rim Marsh Creation	3,493.9
1-10	DuLarge to Grand Caillou Landbridge Marsh Creation	3,767.1
1-11	South Caillou Landbridge Marsh Creation	(included in 1-10)
1-12	Isles Dernieres Restoration	818.0
1-13	Timbalier Islands Restoration	(included in 1-12)
TOTAL:		66,720.7

Table 8 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU3b		
Measure #	Measure	Cost ($M)
1-1	Penchant Basin Plan	17.9
1-2	Convey Atchafalaya River water via the GIWW	(see PU3a 1-2)
1-3	Relocate the Navigation Channel through Lower Atchafalaya River Delta	203.5
1-4	Increase Sediment Transport Down Wax Lake Outlet	27.3
1-5	Barrier Reef from Eugene Island to Pointe au Fer Island	135.8
1-6	Blue Hammock Bayou Freshwater Introduction	(See PU3a 1-4)
1-7	Gulfshore Protection at Point au Fer Island	81.3
1-8	Freshwater Bayou Bank Protection, Belle Isle to Lock	37.3
1-9	Southwest Pass Bank Protection	51.6
1-10	Marsh Island Shoreline Protection	8.0
1-11	Gulfshore Protection from Freshwater Bayou to Southwest Pass	56.7
1-12	Shoreline Protection at Vermillion Bay & West Cote Blanche Bay	94.9
1-13	East Cote Blanche Bay Shore Protection	(included in 1-12)
1-14	Bayou Decade Area Marsh Creation	1,231.2
1-15	Brady Canal Area Marsh Creation	928.7
1-16	Pointe au Fer Island Marsh Creation	745.6
1-17	Marsh Island Marsh Creation	1,475.1
1-18	Wax Lake Outlet Delta Marsh Creation	390.1
1-19	Bayou Penchant Area Marsh Creation	1,569.8
1-20	Terrebonne GIWW Area Marsh Creation	1,244.5
TOTAL:		8,299.3

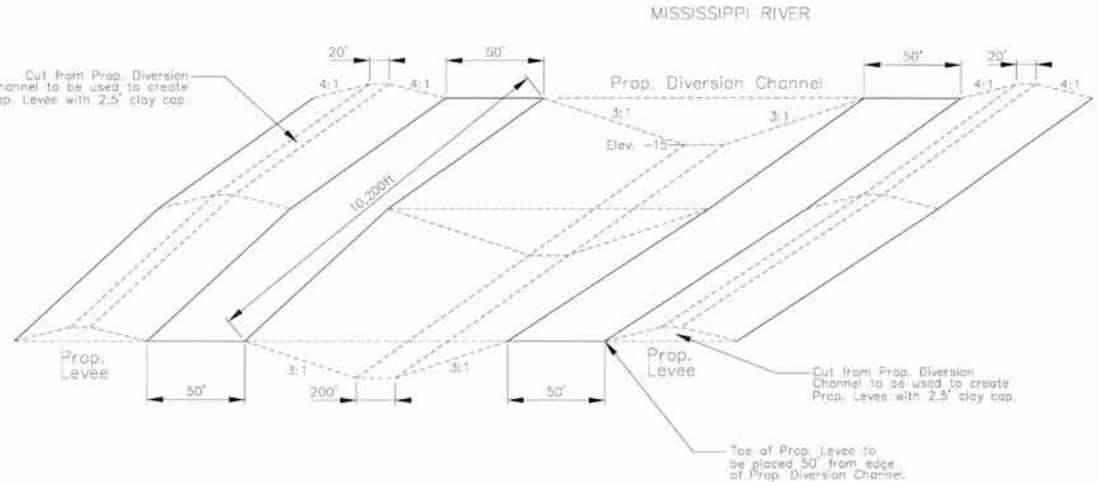
Table 9 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU4		
Measure #	Measure	Cost ($M)
1-1	Marsh Creation at Mud Lake	504.7
1-2	Marsh Creation at South Grand Chenier	352.6
1-3	Marsh Creation at South Pecan Island	785.5
1-4	Marsh Creation at East Pecan Island	796.6
1-5	Marsh Creation at No-Name Bayou	517.7
1-6	Marsh Creation at NW Calcasieu Lake	5,726.2
1-7	Marsh Creation at East Calcasieu Lake	1,082.1
1-8	Marsh Creation at Black Bayou	2,328.1

Table 9 - Coastal Restoration Plan Component – Representative Plan Features First Cost PU4		
Measure #	Measure	Cost ($M)
1-9	Marsh Creation at Gum Cove	940.1
1-10	Marsh Creation at Cameron Meadows	188.9
1-11	Marsh Creation at Central Canal	585.2
1-12	GIWW bank Stabilization	790.6
1-13	Grand Lake Bank Stabilization	314.0
1-14	White Lake Bank Stabilization	234.3
1-15	Gulf Shoreline Stabilization (Sabine River to Calcasieu River)	107.4
1-16	Gulf Shoreline Stabilization (Calcasieu River to Freshwater Bayou)	316.5
TOTAL:		15,570.5

BLIND RIVER DIVERSION
ILLINOIS CENTRAL
ROMEVILLE
MISSISSIPPI RIVER
VICINITY MAP
PLAN VIEW
PLATE 1

BLIND RIVER DIVERSION

CROSS SECTION



NOTES:

River Mile: 159 East Bank
Outflow Channel Invert: -15ft to -2ft (NAVD 88)
Channel Width: 200ft
Channel Length: 10,200ft
Levee Crown: 20ft
Leve Elev.: +15ft (NAVD 88)

US Army Corps of Engineers
NEW ORLEANS DISTRICT

BLIND RIVER DIVERSION
Cross Section

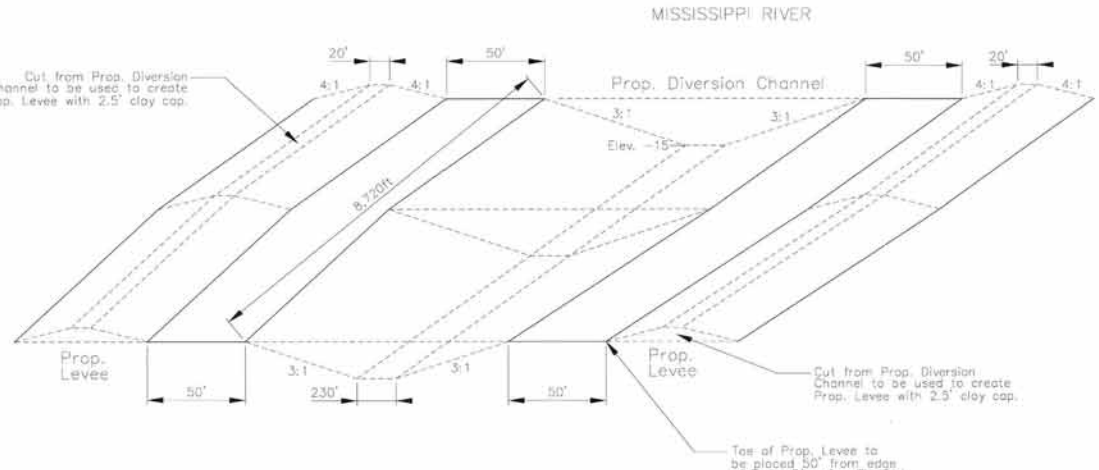
SHEET IDENTIFICATION
01

cost cross all.dgn 11/21/2008 8:45:01 AM

HOPE CANAL DIVERSION
AIRLINE HWY
SOUTHERN RAILROAD
ANGELINA
ILLINOIS CENTRAL
GARYVILLE
MOUNT ARIEL
TAILINGS PONDS
MISSISSIPPI RIVER
SCALE 1" = 500'
VICINITY MAP
PLATE 2

HOPE CANAL DIVERSION

CROSS SECTION



NOTES:
River Mile: 144 East Bank
Outflow Channel Invert: -15ft to -2ft (NAVD 88)
Channel Width: 230ft
Channel Length: 8,720ft
Levee Crown: 20ft
Leve Elev.: +15ft (NAVD 88)

HOPE CANAL DIVERSION
Cross Section

SHEET IDENTIFICATION
02

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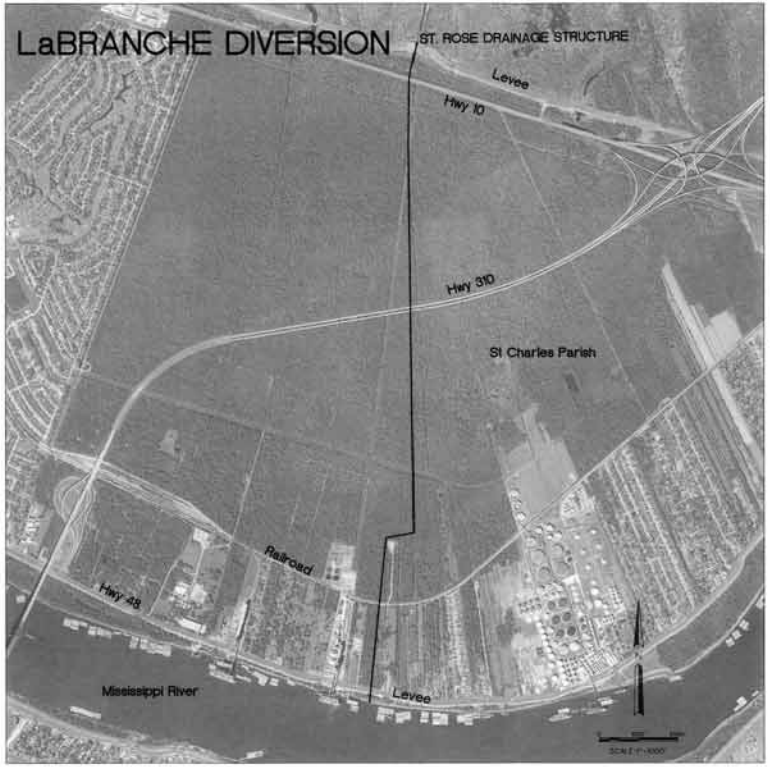
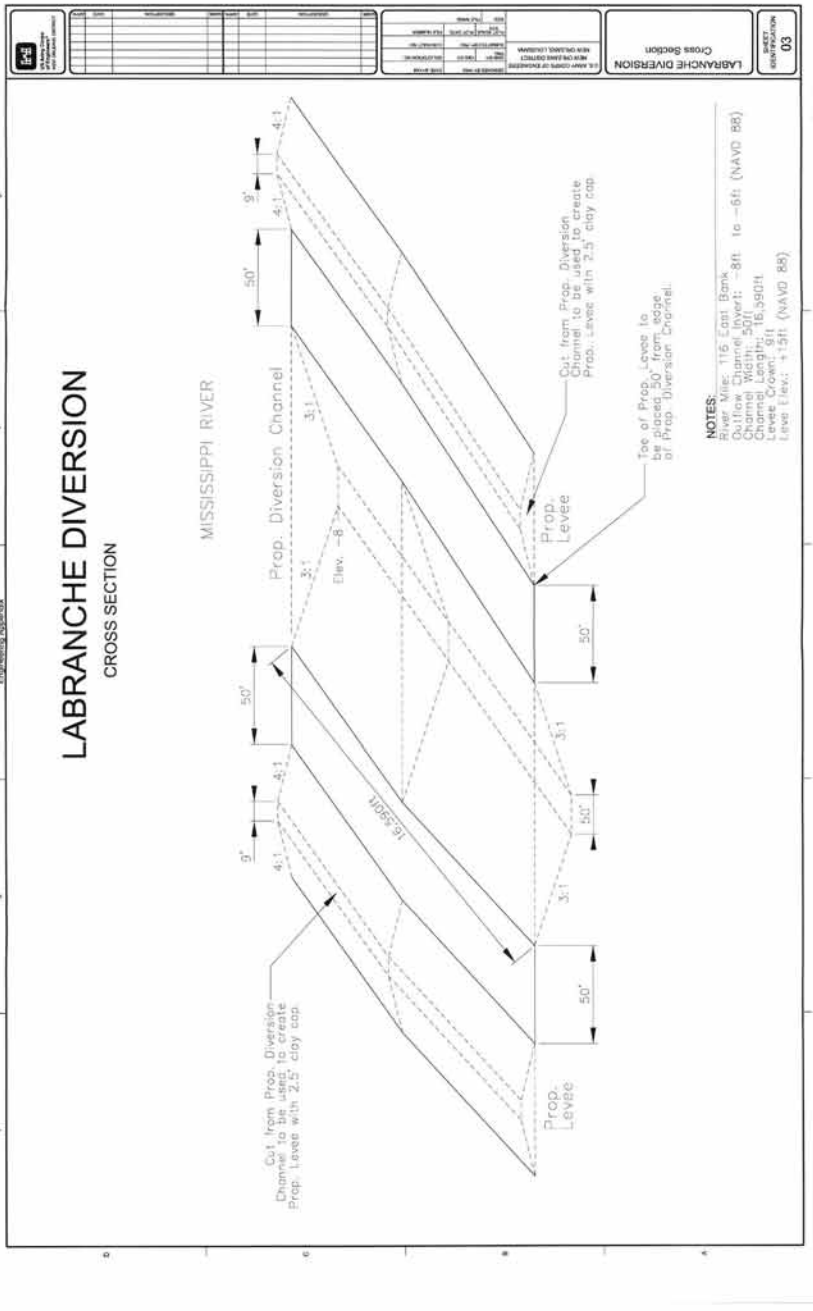


VICINITY MAP

NOTE:
1.) Approximately 170 ac needed.
2.) River Mile 116 (west bank).

SHEET IDENTIFICATION
PLATE 3

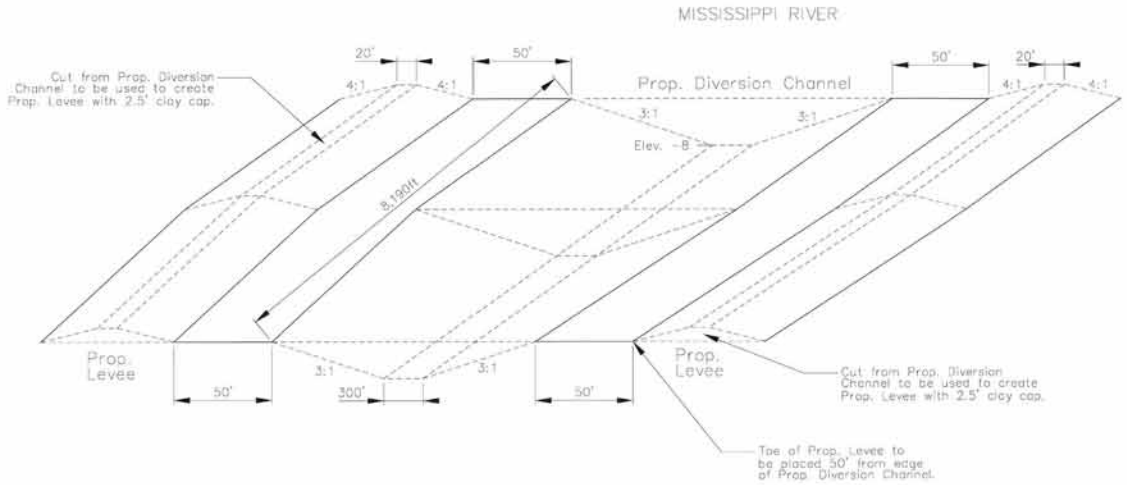
labranch.dgn 11/21/2008 9:19:48 AM


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
LABRANCHE DIVERSION
CROSS SECTION
MISSISSIPPI RIVER
Prop. Diversion Channel
Prop. Levee
Prop. Levee
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap
Toe of Prop. Levee to be placed 50' from edge of Prop. Diversion Channel.
Elev. -8'
16,590ft
50'
9'
4:1
3:1
NOTES:
River Mile: 116 East Bank
Outflow Channel Invert: -8ft to -6ft (NAVD 88)
Channel Width: 300'
Channel Length: 16,590ft
Levee Crown: 9ft
Levee Elev.: +15ft (NAVD 88)
LABRANCHE DIVERSION
Cross Section
SHEET
03
cost cross all.dgn 11/21/2008 9:32:49 AM

BAYOU BIENVENUE DIVERSION
BACK PROTECTION LEVEE
FORTY ARPENT CANAL
TWENTY ARPENT
E JUDGE PEREZ DR
E ST BERNARD HWY
CYPRESS GARDENS
ST BERNARD GROVE
MERAUX
FRANCES PLACE
MISSISSIPPI RIVER
SCALE: 1" = 500'
VICINITY MAP
PLAN VIEW
PLATE 4

BAYOU BIENVENUE DIVERSION

CROSS SECTION



NOTES:
River Mile: 92 East Bank
Outflow Channel Invert: -15ft to -2ft (NAVD 88)
Channel Width: 300ft
Channel Length: 8,190ft
Levee Crown: 20ft
Leve Elev.: +15tf (NAVD 88)

BAYOU BIENVENUE DIVERSION
Cross Section

SHEET IDENTIFICATION
04

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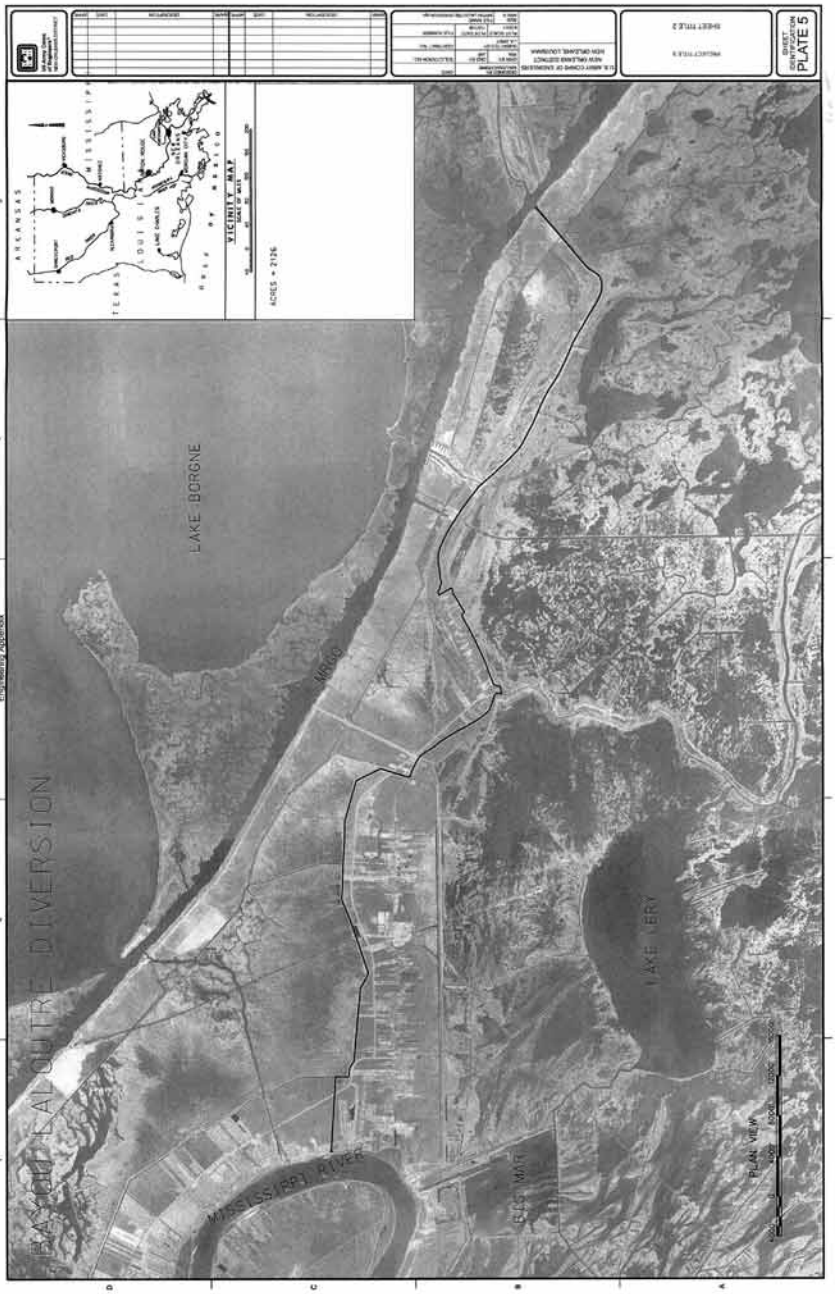
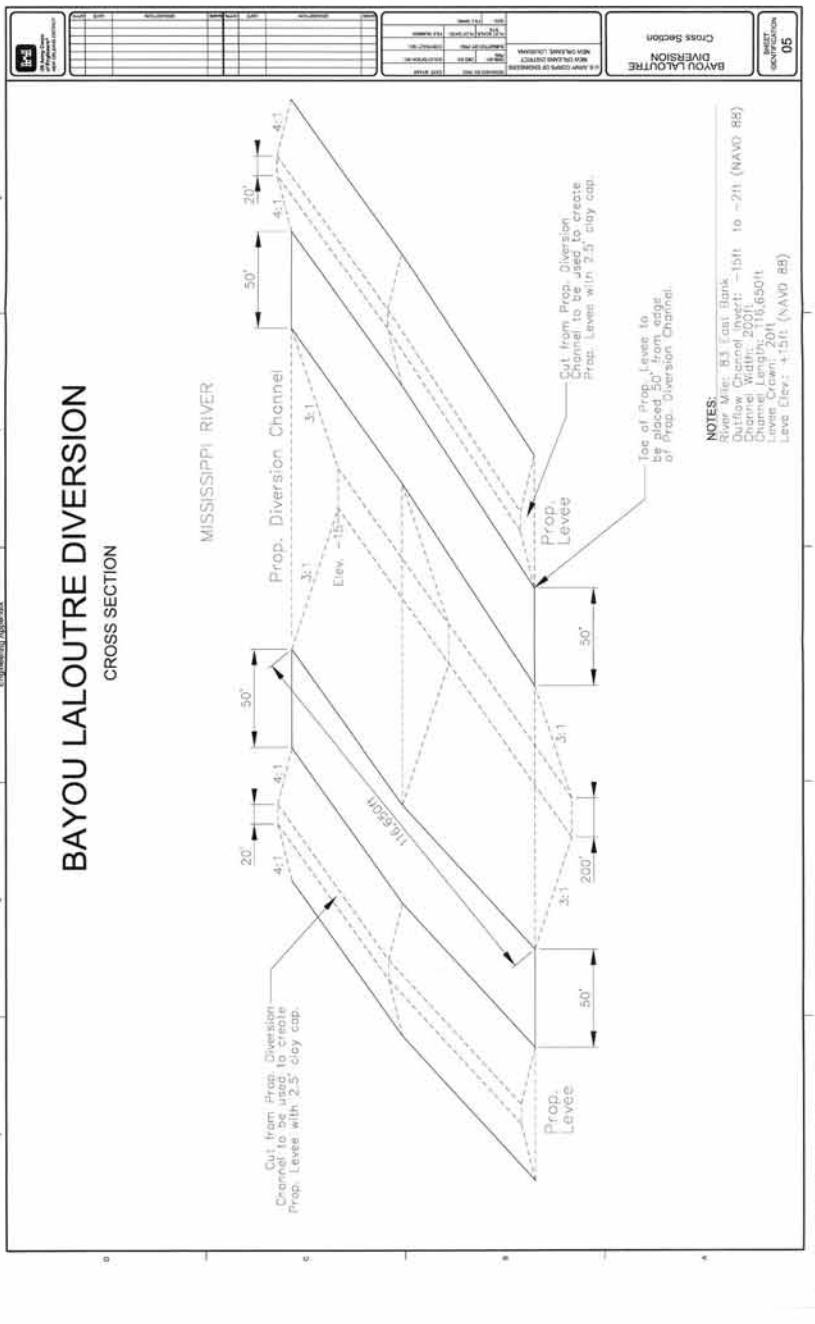

BAYOU LA LOUTRE DIVERSION
LAKE BORGNE
MRGO
MISSISSIPPI RIVER
LAKE ERY
VICINITY MAP
ACRES ± 2126
PLATE 5


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
BAYOU LALOUTRE DIVERSION
CROSS SECTION
MISSISSIPPI RIVER
Prop. Diversion Channel
Elev. -15ft
Tie Back
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Prop. Levee
Toe of Prop. Levee to be placed 50' from edge of Prop. Diversion Channel.
NOTES:
River Mile: 83 East Bank
Outflow Channel Invert: -15ft to -21ft (NAVD 88)
Channel Width: 200'
Channel Length: 16,650ft
Levee Crown: 20ft
Levee Elev: +15ft (NAVD 88)
BAYOU LALOUTRE DIVERSION
Cross Section
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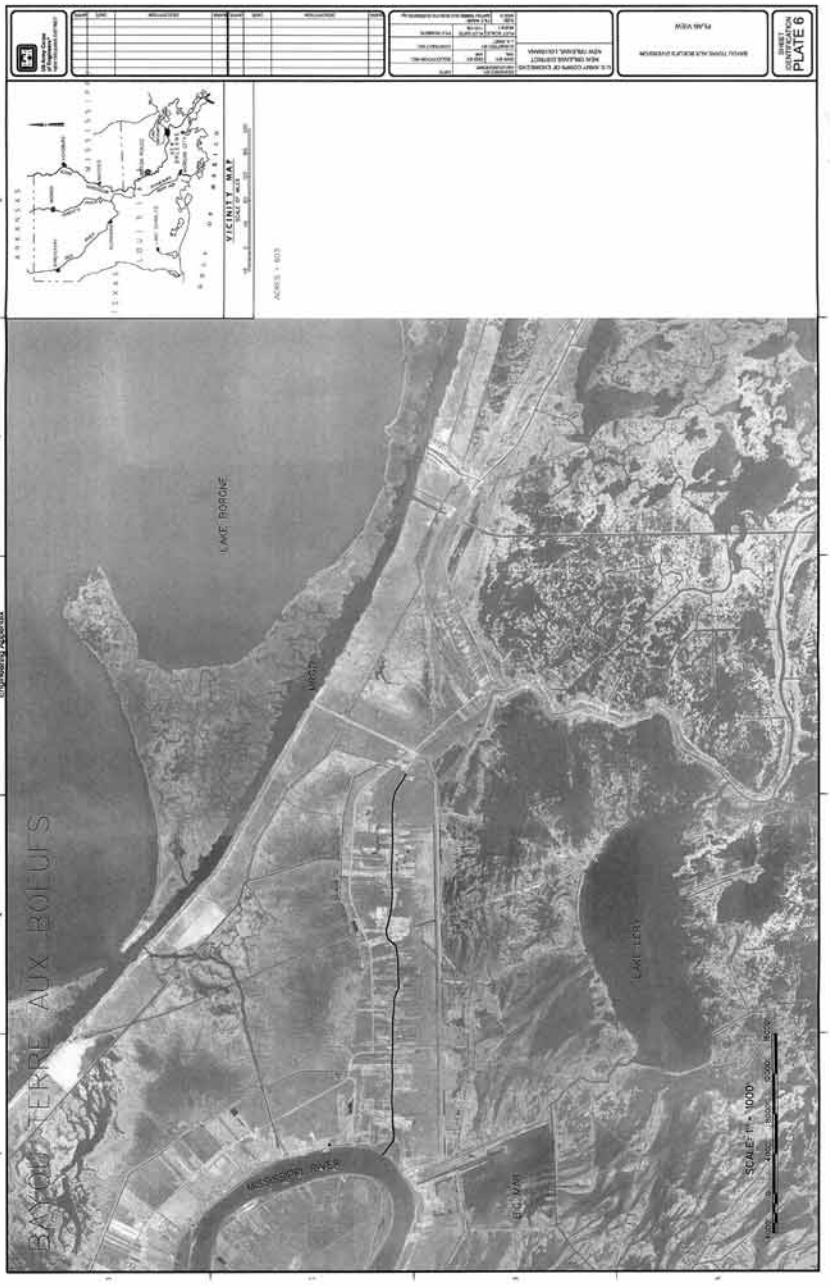
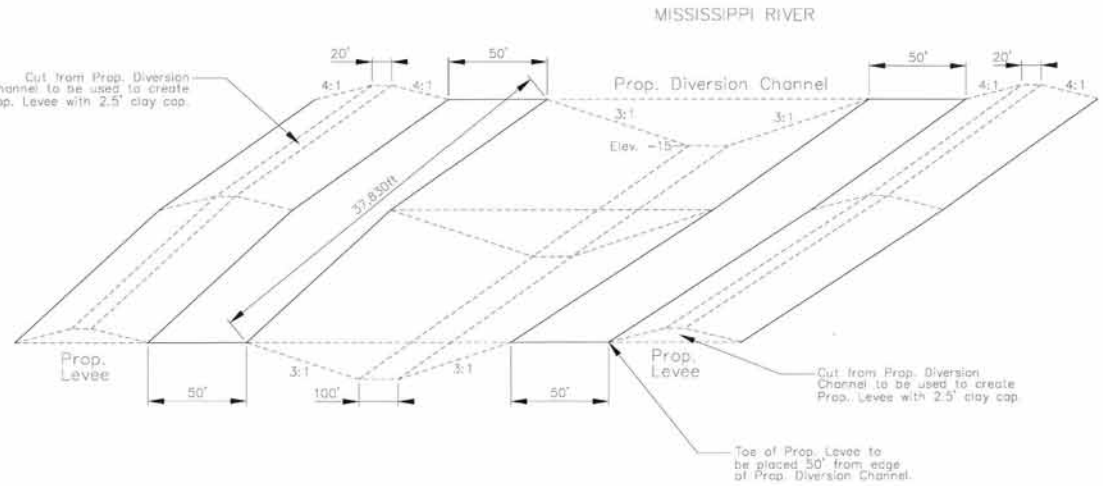

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
BAYOU TERRE AUX BOEUFS
LAKE BORGNE
MISSISSIPPI RIVER
LAKE LERY
SCALE: 1" = 1000'
VICINITY MAP
PLATE 6

BAYOU TERRE DIVERSION

CROSS SECTION



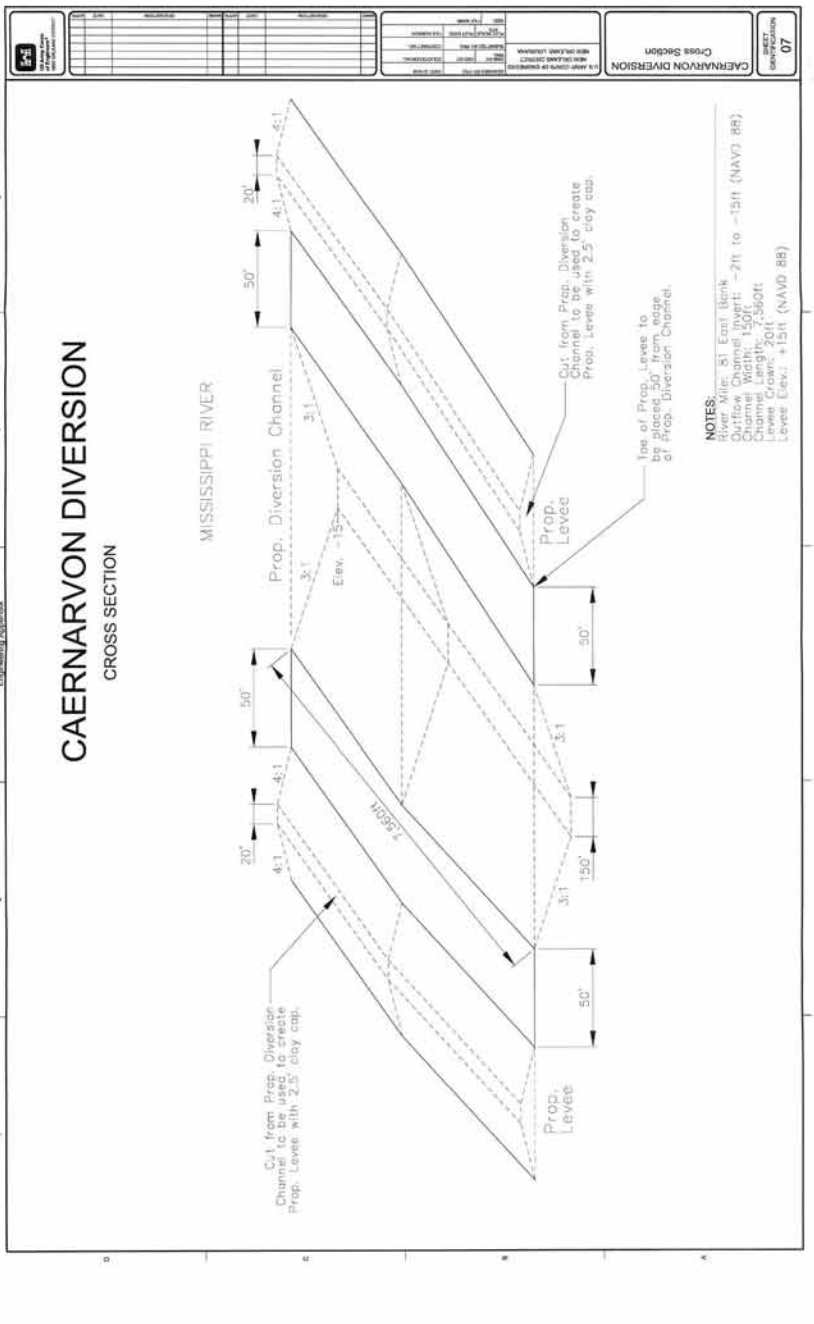
NOTES:
River Mile: 82 East Bank
Outflow Channel Invert: -15ft to -2ft (NAVD 88)
Channel Width: 100ft
Channel Length: 37,830ft
Levee Crown: 20ft
Leve Elev.: +15ft (NAVD 88)

BAYOU TERRE DIVERSION
Cross Section

SHEET IDENTIFICATION
06

cost cross all.dgn 11/21/2008 11:02:41 AM

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
CAERNARVON DIVERSION
MISSISSIPPI RIVER
CAERNARVON
BIG MR
SCALE: 1" = 1000'
VICINITY MAP
PLATE 7


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
CAERNARVON DIVERSION
CROSS SECTION
MISSISSIPPI RIVER
Prop. Diversion Channel
Elev. -15ft
Prop. Levee
Prop. Levee
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Toe of Prop. Levee to be placed 50' from edge of Prop. Diversion Channel.
20'
50'
4:1
3:1
150'
NOTES:
River Mile: 81 East Bank
Outflow Channel Invert: -2ft to -15ft (NAVD 88)
Channel Width: 150ft
Channel Length: 7,560ft
Levee Crown: 20ft
Levee Elev.: +15ft (NAVD 88)
CAERNARVON DIVERSION
Cross Section
SHEET IDENTIFICATION
07
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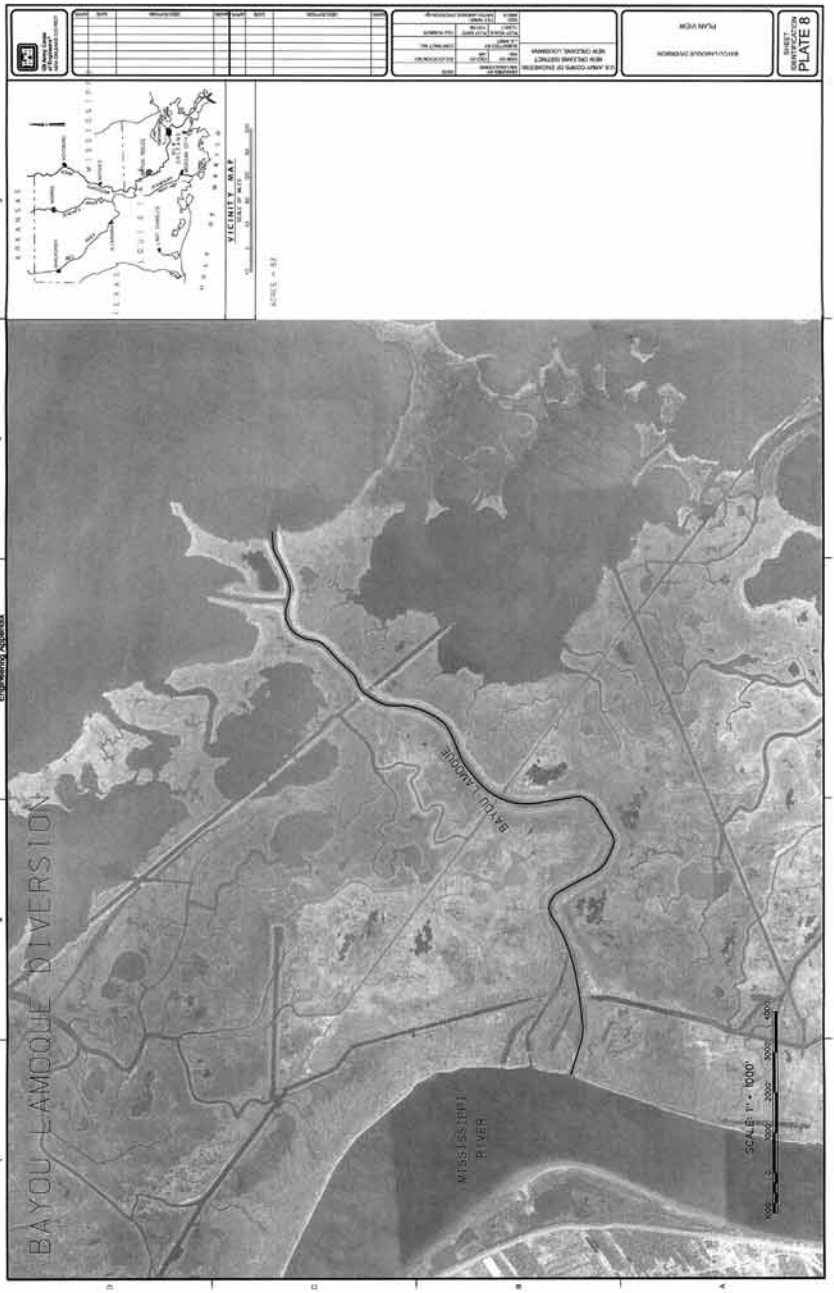
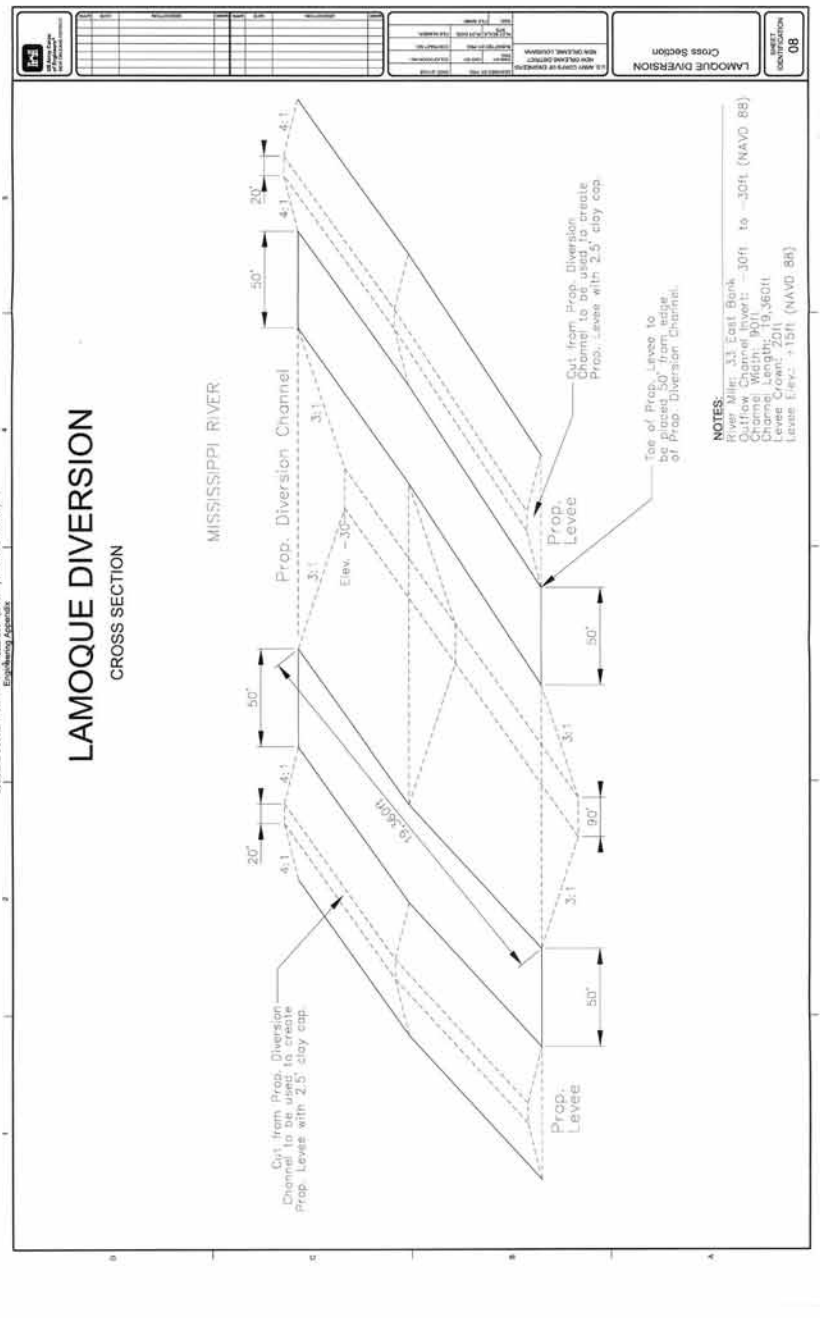
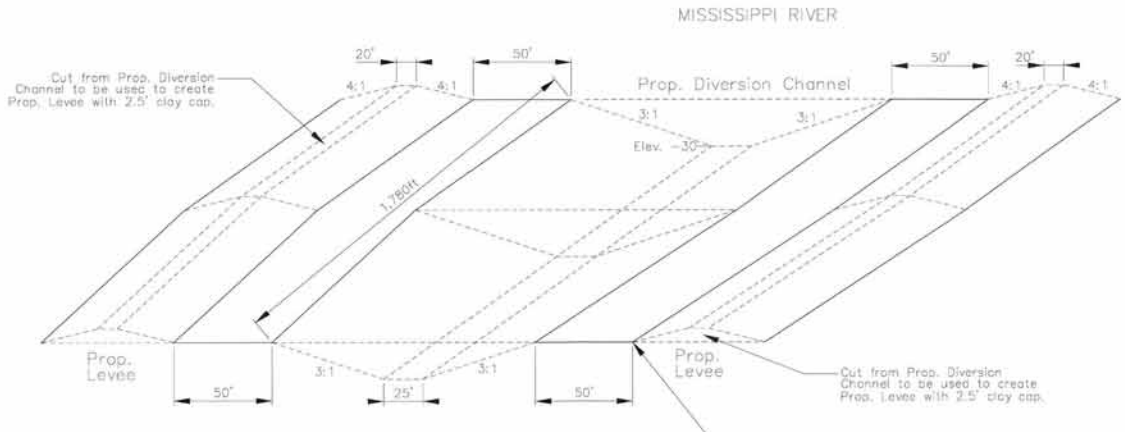

BAYOU LAMOQUE DIVERSION
BAYOU LAMOQUE
MISSISSIPPI RIVER
SCALE 1" = 1000'
VICINITY MAP
PLAN VIEW
PLATE 8
Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
LAMOQUE DIVERSION
CROSS SECTION
MISSISSIPPI RIVER
Prop. Diversion Channel
Prop. Levee
Prop. Levee
Elev. -30ft
150-600ft
20'
50'
50'
90'
50'
20'
4:1
4:1
3:1
3:1
3:1
3:1
4:1
4:1
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Toe of Prop. Levee to be placed 50' from edge of Prop. Diversion Channel.
NOTES:
River Mile: 33 East Bank
Outflow Channel Invert: -30ft to -30ft (NAVD 88)
Channel Width: 900
Channel Length: 9,360ft
Levee Crown: 20ft
Levee Elev.: +15ft (NAVD 88)
LAMOQUE DIVERSION
Cross Section
SHEET IDENTIFICATION
08
coast cross all.dgn 11/21/2008 8:52:37 AM

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
GRAND BAY DIVERSION
OLD STUMP LAKE
GRAND BAY
PIPELINE CANAL
MISSISSIPPI RIVER
SCALE 1" = 500'
VICINITY MAP
PLAN VIEW
PLATE 9

GRAND BAY DIVERSION

CROSS SECTION



NOTES:
River Mile: 16 East Bank
Outflow Channel Invert: -30ft to -30ft (NAVD 88)
Channel Width: 25ft
Channel Length: 1,780ft
Levee Crown: 20ft
Leve Elevs.: +15ft (NAVD 88)

US Army Corps of Engineers
NEW ORLEANS DISTRICT

GRAND BAY DIVERSION
Cross Section

SHEET IDENTIFICATION
09

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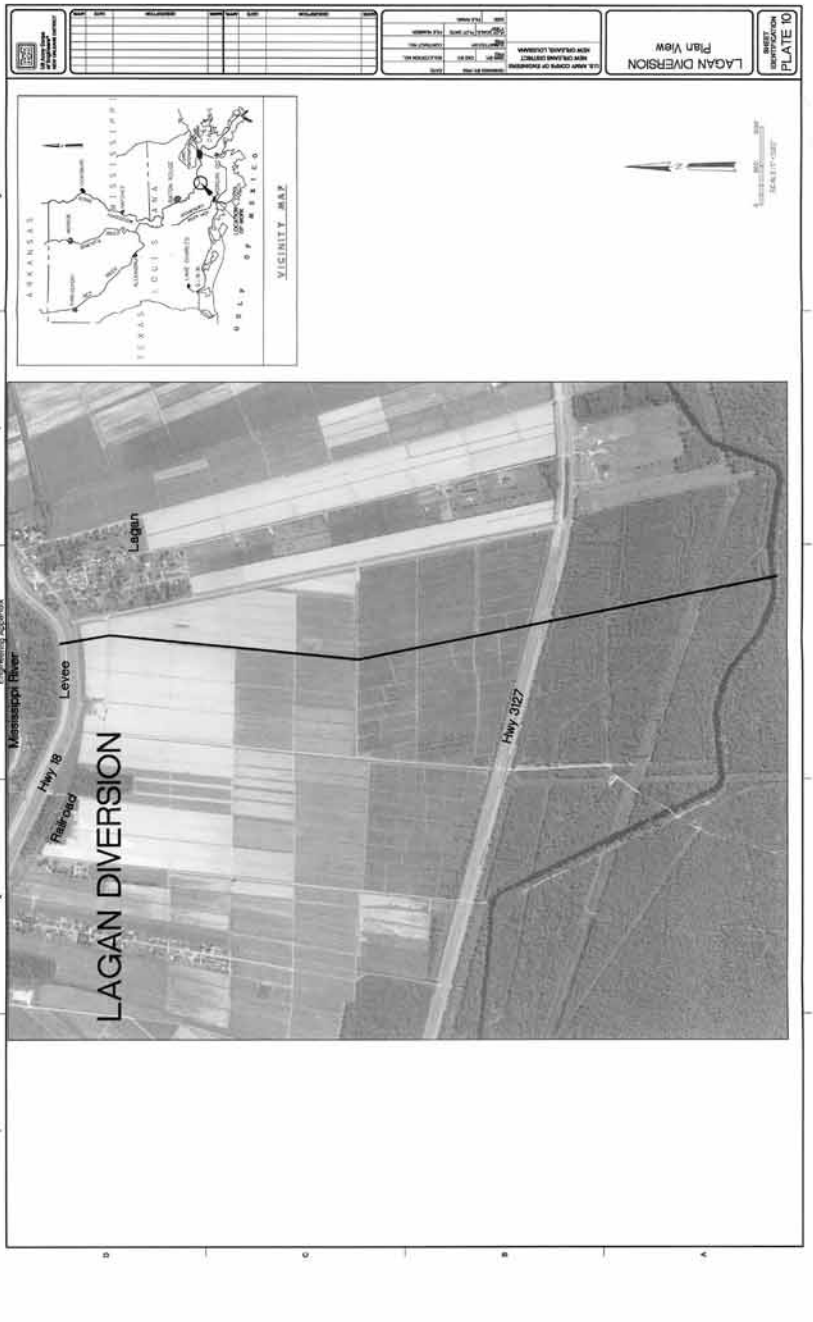
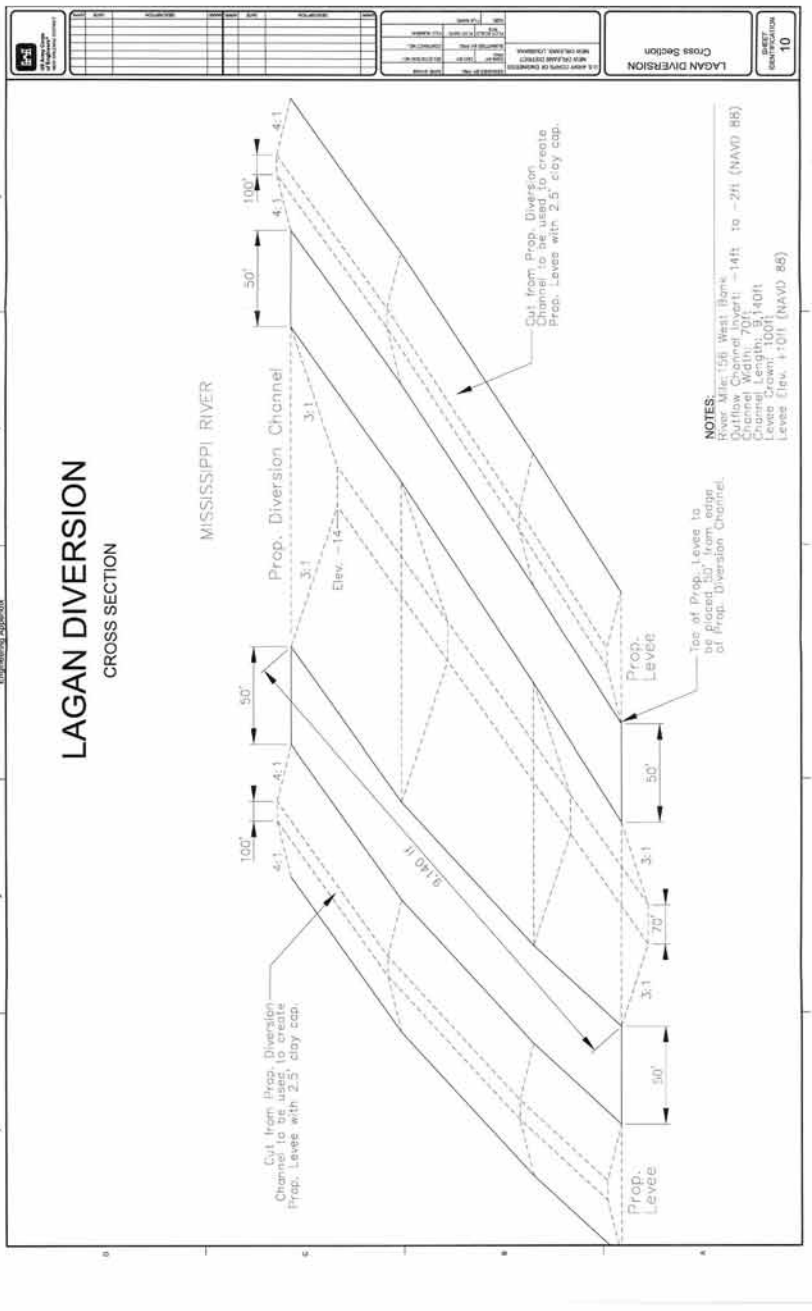

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
LAGAN DIVERSION
Mississippi River
Levee
Lagan
Hwy 18
Railroad
Hwy 3127
VICINITY MAP
LAGAN DIVERSION
Plan View
PLATE 10
Lagan Plan veiw.dgn 11/21/2008 8:30:47 AM


LAGAN DIVERSION
CROSS SECTION
MISSISSIPPI RIVER
Prop. Diversion Channel
Prop. Levee
Prop. Levee
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Cut from Prop. Diversion Channel to be used to create Prop. Levee with 2.5' clay cap.
Toe of Prop. Levee to be placed 50' from edge of Prop. Diversion Channel.
Elev. -14ft
9,140 ft
100'
50'
50'
100'
70'
50'
50'
4:1
3:1
NOTES:
River Mile 156 West Bank
Outflow Channel Invert: -14ft to -2ft (NAVD 88)
Channel Width: 140ft
Channel Length: 9,140ft
Levee Crown: 100ft
Levee Elev. +10ft (NAVD 88)
LAGAN DIVERSION
Cross Section
SHEET IDENTIFICATION
10
Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
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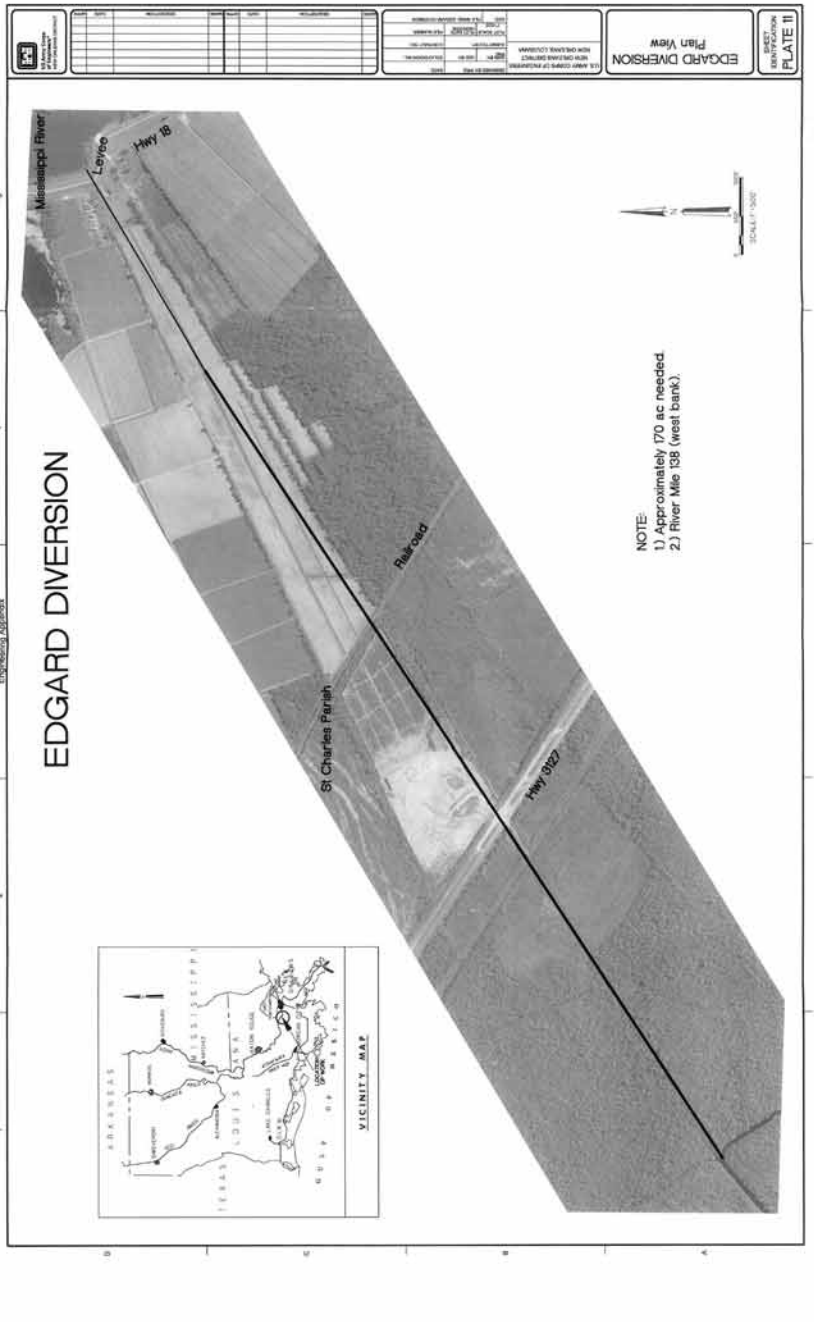
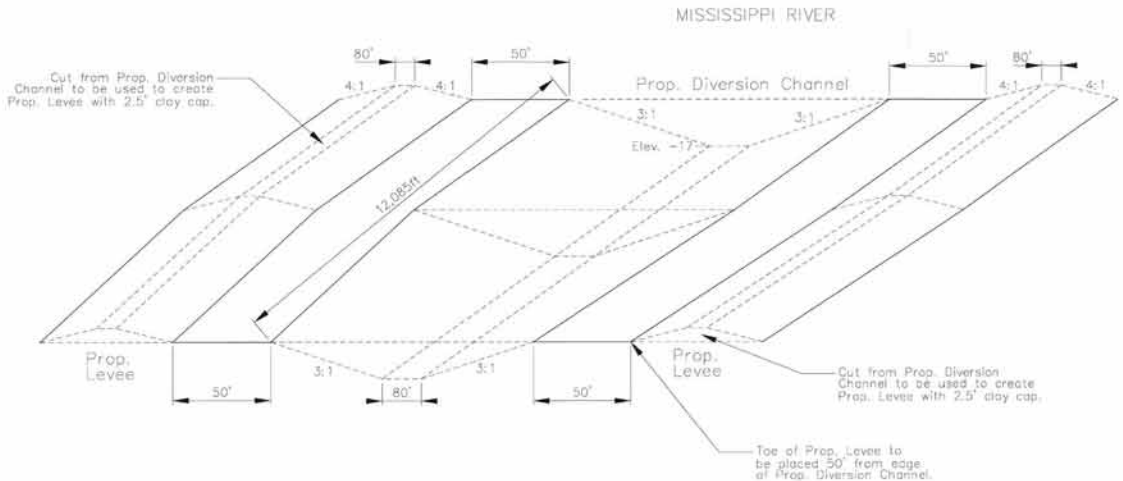

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
EDGARD DIVERSION
Mississippi River
Levee
Hwy 18
St Charles Parish
Railroad
Hwy 3127
NOTE:
1) Approximately 170 ac needed.
2) River Mile 138 (west bank).
VICINITY MAP
EDGARD DIVERSION
Plan View
PLATE 11
plan views alt.dgn 11/21/2008 9:06:42 AM

EDGARD DIVERSION

CROSS SECTION



NOTES:
River Mile: 138 West Bank
Outflow Channel Invert: -17ft to -2ft (NAVD 88)
Channel Width: 80ft
Channel Length: 12,085ft
Levee Crown: 80ft
Levee Elev: +10ft (NAVD 88)

EDGARD DIVERSION
Cross Section

SHEET IDENTIFICATION
11

cost cross all.dgn 11/21/2008 8:37:23 AM

NAOMI DIVERSION

VICINITY MAP

NOTE:
1.) Approximately 33 ac needed.
2.) River Mile 65 (west bank).

SCALE: 1" = 500'

US Army Corps of Engineers
NEW ORLEANS DISTRICT

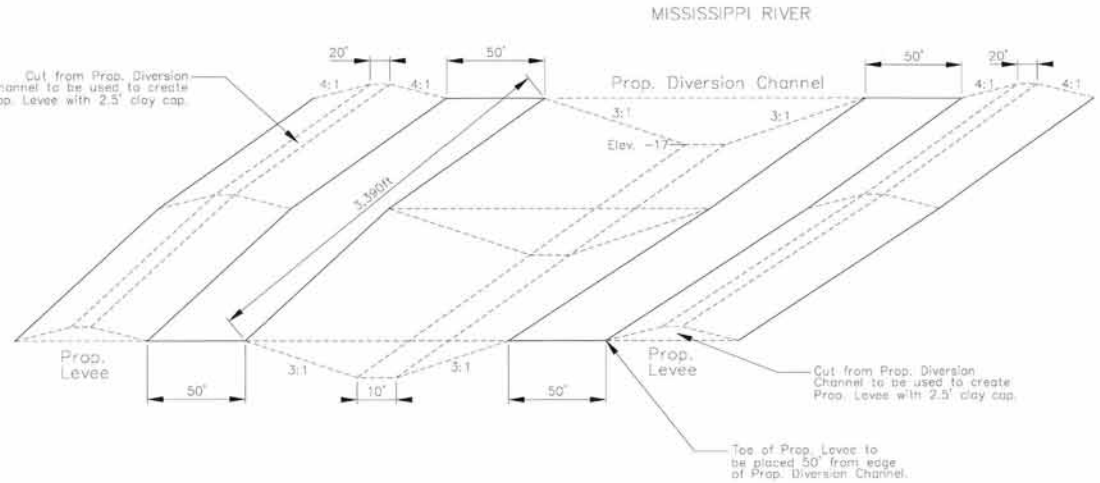
NAOMI DIVERSION
Plan View

SHEET IDENTIFICATION
PLATE 12

plan views all.dgn 11/21/2008 9:01:37 AM

NAOMI DIVERSION

CROSS SECTION

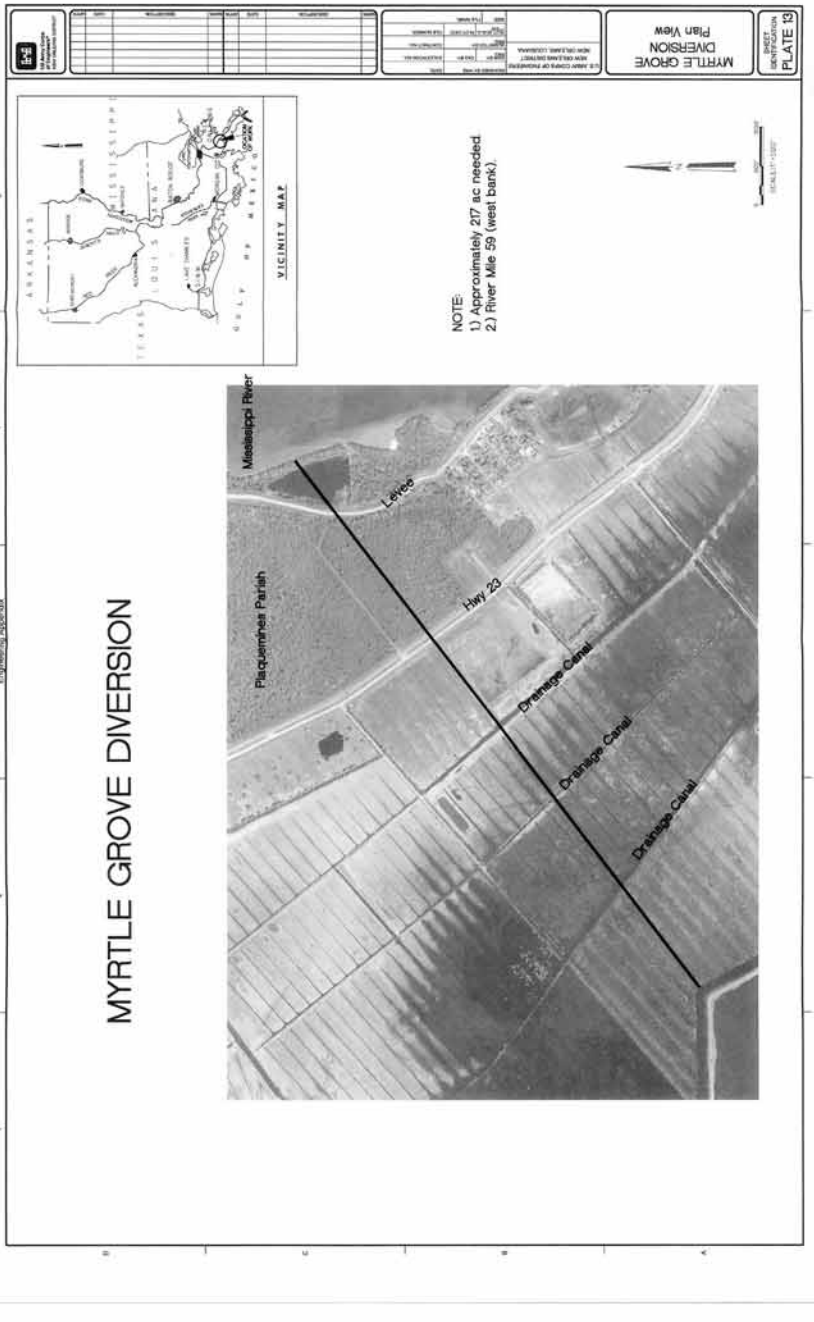


NOTES:
River Mile: 65 West Bank
Outflow Channel Invert: -17ft to -2ft (NAVD 88)
Channel Width: 10ft
Channel Length: 3,390ft
Levee Crown: 20ft
Leve Elev.: +10ft (NAVD 88)

NAOMI DIVERSION
Cross Section

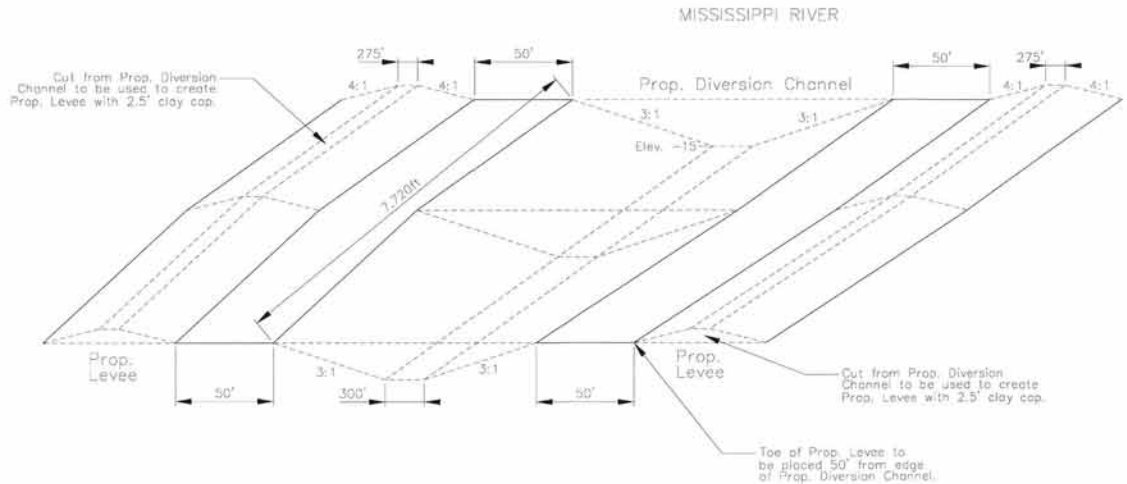
SHEET IDENTIFICATION
12

cost cross all.dgn 11/21/2008 8:38:43 AM


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
MYRTLE GROVE DIVERSION
VICINITY MAP
ARKANSAS
MISSISSIPPI
LOUISIANA
TEXAS
GULF OF MEXICO
NOTE:
1) Approximately 217 ac needed.
2) River Mile 59 (west bank).
Mississippi River
Levee
Hwy 23
Drainage Canal
Drainage Canal
Drainage Canal
Plaquemines Parish
MYRTLE GROVE
DIVERSION
Plan View
SHEET
PLATE 13
plan views all.dgn 11/21/2008 8:59:50 AM

MYRTLE GROVE DIVERSION

CROSS SECTION



NOTES:

River Mile: 59 West Bank
Outflow Channel Invert: -15ft to -2ft (NAVD 88)
Channel Width: 300ft
Channel Length: 7,720ft

MYRTLE GROVE DIVERSION
Cross Section

SHEET IDENTIFICATION
13

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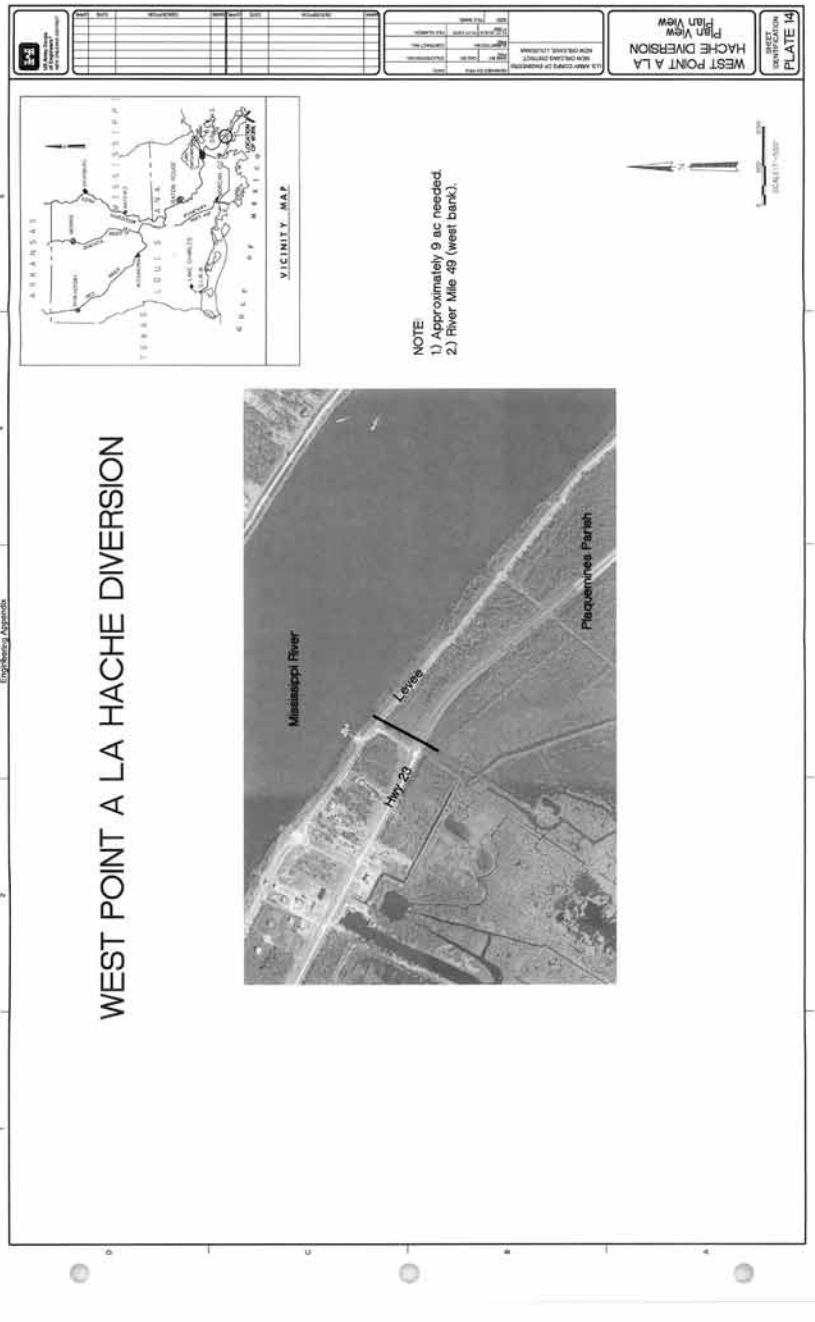
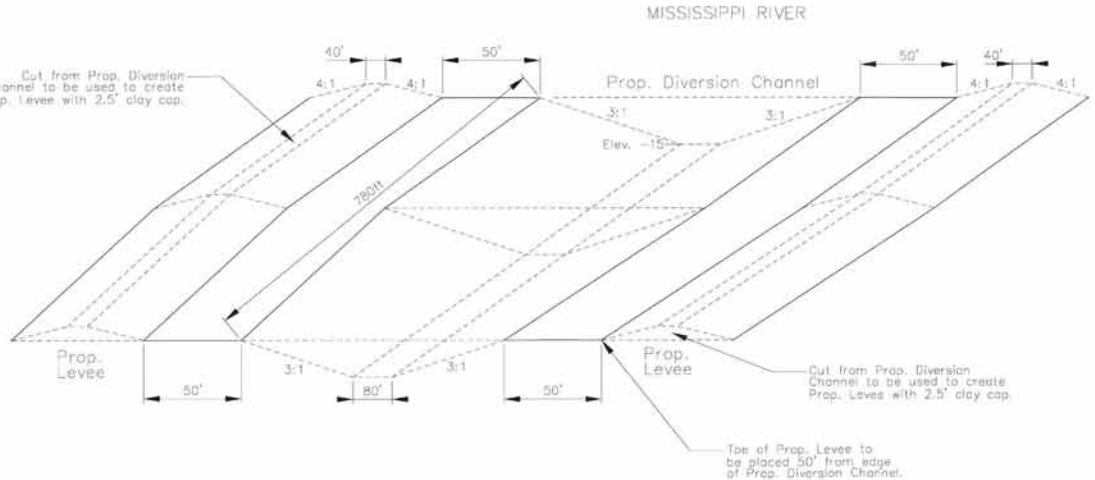

Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
WEST POINT A LA HACHE DIVERSION
VICINITY MAP
Mississippi River
Levee
Hwy 23
Plaquemines Parish
NOTE:
1) Approximately 9 ac needed.
2) River Mile 49 (west bank).
WEST POINT A LA
HACHE DIVERSION
Plan View
PLATE 14
plan views all.dgn 11/21/2008 8:58:40 AM

WEST POINT LA HACHE DIVERSION

CROSS SECTION

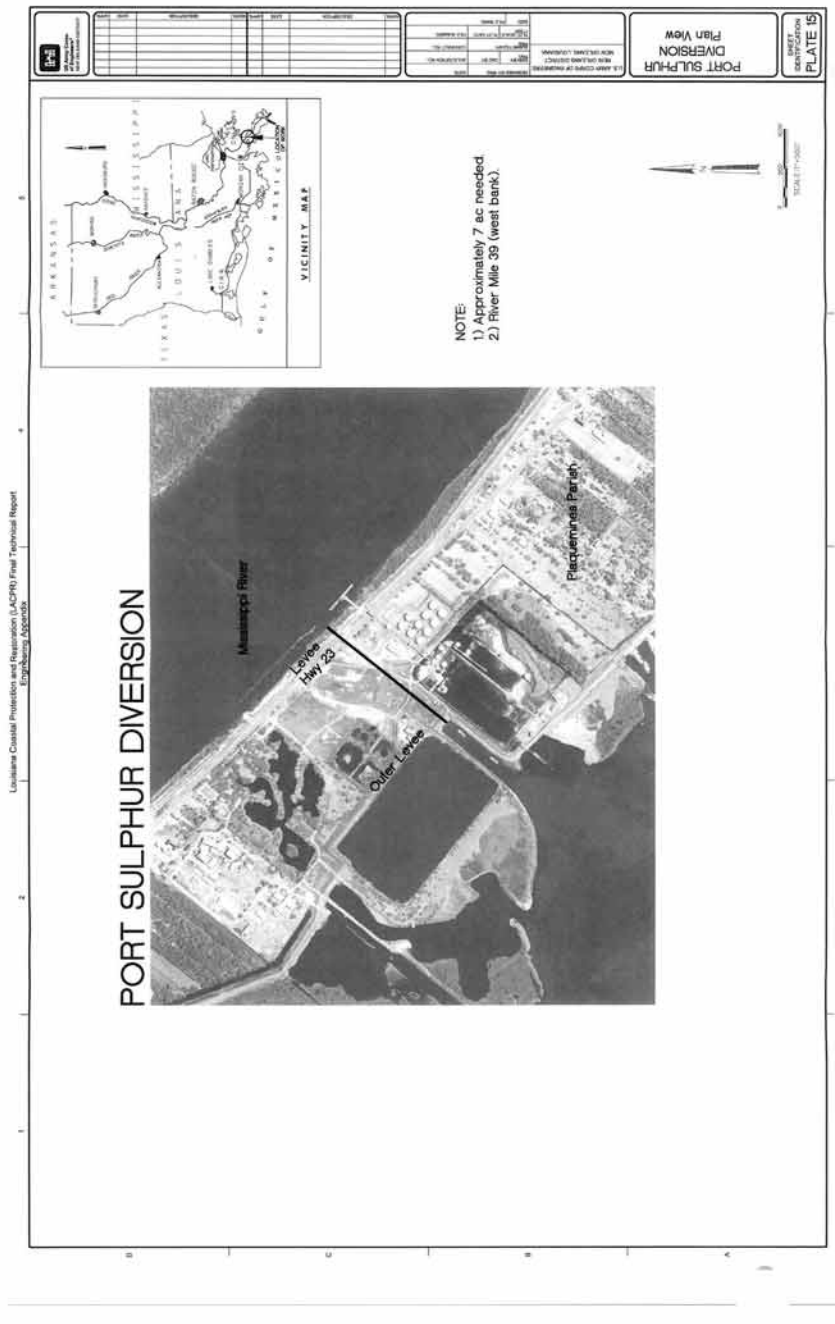


NOTES:
River Mile: 49 West Bank
Outflow Channel Invert: -12.5ft to-2ft (NAVD 88)
Channel Width: 80ft
Channel Length: 780ft
Levee Crown: 40ft
Levee elev.: 10ft

WEST POINT A LA HACHE DIVERSION
Cross Section

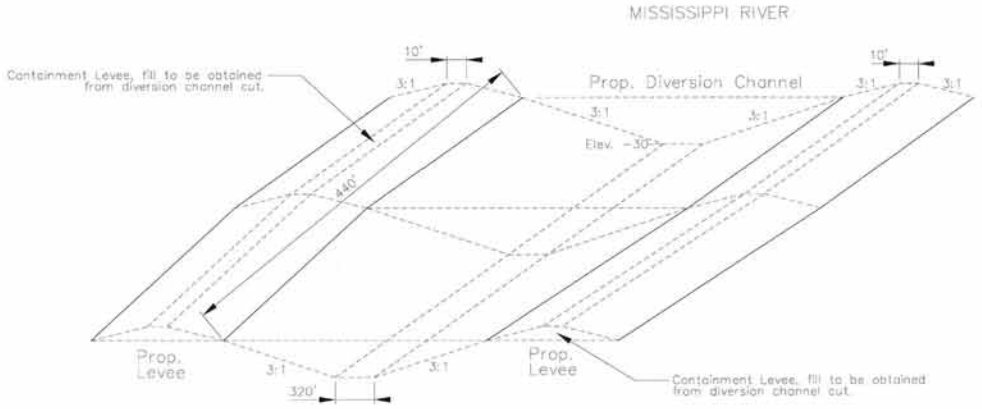
SHEET IDENTIFICATION
14

cost cross all.dgn 11/21/2008 8:40:42 AM


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
PORT SULPHUR DIVERSION
Mississippi River
Levee
Hwy 23
Outer Levee
Plaquemines Parish
VICINITY MAP
NOTE:
1) Approximately 7 ac needed
2) River Mile 39 (west bank).
PORT SULPHUR
DIVERSION
Plan View
PLATE 15
plan views all.dgn 11/21/2008 8:57:20 AM

PORT SULPHER DIVERSION

CROSS SECTION



NOTES:
River Mile: 59 West Bank
Outflow Channel Invert: -30ft to -2ft (NAVD 88)
Channel Width: 320ft
Channel Length: 440ft
Levee Crown: 10ft
Levee Elev.: +5ft (NAVD 88)

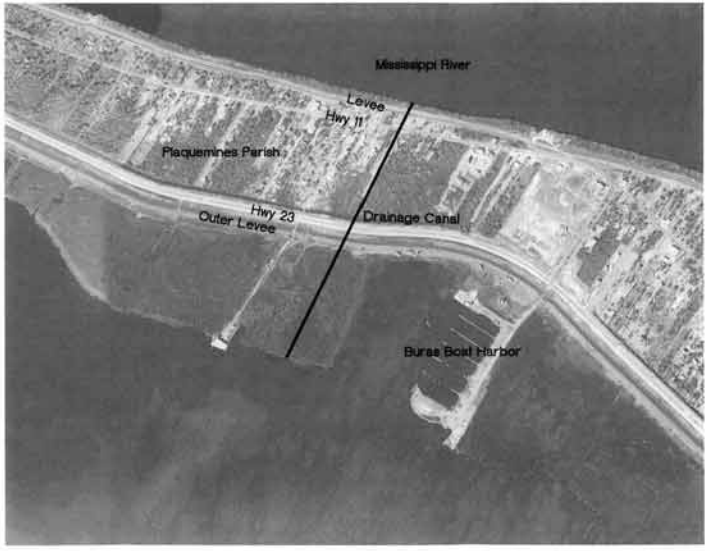
US Army Corps of Engineers
NEW ORLEANS DISTRICT

PORT SULPHER DIVERSION
Cross Section

SHEET IDENTIFICATION
15

cost cross all.dgn 11/21/2008 8:41:44 AM

BURAS DIVERSION



VICINITY MAP

NOTE:
1.) Approximately 85 ac needed.
2.) River Mile 25 (west bank).

SCALE 1"=500'

US Army Corps of Engineers
New Orleans District

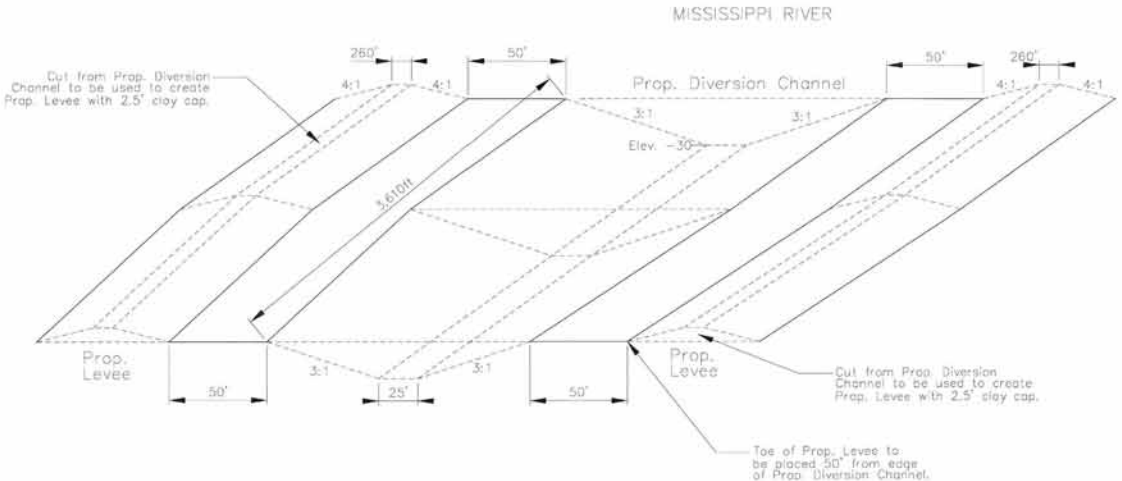
BURAS DIVERSION
Plan View

SHEET IDENTIFICATION
PLATE 16

plan views all.dgn 11/21/2008 8:54:08 AM

BURAS DIVERSION

CROSS SECTION

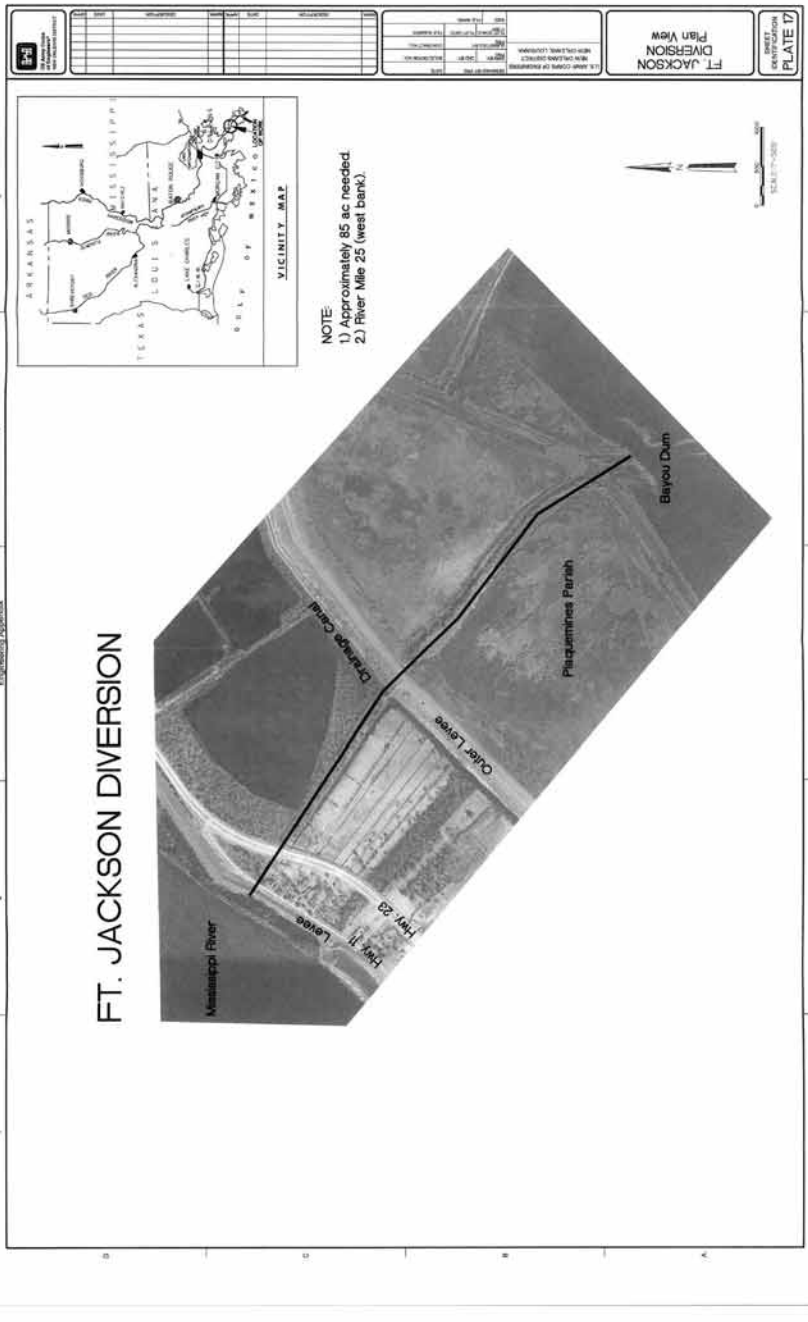


NOTES:
River Mile: 25 West Bank
Outflow Channel Invert: -30ft to-2ft (NAVD 88)
Channel Width: 3,610ft
Channel Length: 25ft
Levee Crown: 260ft
Levee Elev.: +10ft (NAVD 88)

BURAS DIVERSION
Cross Section

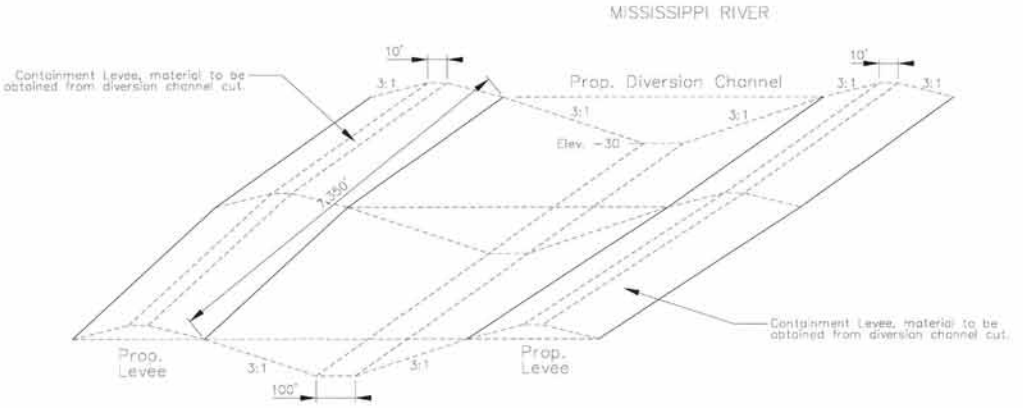
SHEET IDENTIFICATION
16

cost cross all.dgn 11/21/2008 8:42:40 AM


Louisiana Coastal Protection and Restoration (LACPR) Final Technical Report
Engineering Appendix
FT. JACKSON DIVERSION
Mississippi River
Levee
HWY 11
HWY 23
Outer Levee
Drainage Canal
Plaquemines Parish
Bayou Dum
NOTE:
1) Approximately 85 ac needed.
2) River Mile 25 (west bank).
VICINITY MAP
ARKANSAS
LOUISIANA
TEXAS
MISSISSIPPI
GULF OF MEXICO
FT JACKSON
DIVERSION
Plan View
PLATE 17
plan views all.dgn 11/21/2008 8:45:10 AM

FORT JACKSON DIVERSION

CROSS SECTION



NOTES:
River Mile: 20 West Bank
Outflow Channel Invert: -30ft to -2ft (NAVD 88)
Channel Width: 7,350ft
Channel Length: 100ft
Levee Crown: 10ft
Levee Elev.: +5ft (NAVD 88)

US Army Corps of Engineers
NEW ORLEANS DISTRICT

DATE | DESCRIPTION

DESIGNED BY: | DRAWN BY: | CHECKED BY: | SUBMITTED BY:

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
NEW ORLEANS, LOUISIANA

FT JACKSON DIVERSION
Cross Section

SHEET IDENTIFICATION
17

cost cross all.dgn 11/21/2008 8:43:30 AM

ANNEX 2

COST ESTIMATES

ANNEX 2
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ANNEX 2
COST ESTIMATES

GENERAL

This annex provides more information on the development of cost estimates for the alternatives for the LACPR investigation. Cost estimates were developed for the many potential features of the LACPR study which include levees, flood control structures, rock protection, diversion structures, clearing, soil mixing, marsh creation and nourishment, beach nourishment, and ridge restoration. The estimated costs were based upon an analysis of each line item evaluating quantity, production rate, and time, together with the appropriate equipment, labor and material costs. All of the construction work is generally common to the New Orleans District. In addition, all labor, equipment and materials are typical of this type of construction. Given the magnitude of this study, access to the construction sites is assumed to be available, either by land or water or both depending on the exact location of the feature.

Given the magnitude of the overall project, larger quantities of equipment are assumed available based on adequate funding levels existing.

Due to time constraints on this study, the delay in developing final design data, and the number of alternatives, a parametric matrix approach was used to combine and ratio the various individual features that were estimated into the costs for each structural alternative. A structural alternative is an alternative consisting solely of structural measures. Structural measures for LACPR primarily consist of physical structures that reduce surge and wave run-up, such as continuous or ring levees on land connected to floodgates acting as waterway barriers, where necessary. For example, detailed calculations and costs were done for levee templates at El. 25, 30, and 40. When the final design elevations for the different alternatives were determined, a ratio of the base costs was used to determine the cost for the alternatives.

The cost estimates for large structures which are a part of the structural alternatives (locks, closure gates, and pump stations) were developed as part of this study. Information on the development of these costs can be found in the report titled "Preliminary Technical Report for the Louisiana Coastal Protection and Restoration Study Structural Components" dated May 2006. These costs were also done using a parametric approach where detailed calculations and costs were done for structures at El. 30, 35 and 40 and interpolated to determine costs for required final design elevations.

The costs prepared for and presented in this report are not to be used for making funding decisions. They are sufficient in detail for comparison of plans for initial screening, for use in performing MCDA and presenting the final array of alternatives. A more detailed cost estimate per regulation will be generated in the future upon completion of additional engineering analysis.

PLANS OF OPERATION

Levees

The levee construction plan was developed based on the cross section and location of the particular alignment. Construction technique on each reach was split between combinations of truck hauling, barge hauling, and casting from adjacent borrow. Some moisture control work was assumed to be needed on 90 percent of the levee embankment. The majority of all other materials (rock, sand, geotextile, etc.) were assumed to be hauled in by truck or barge/truck combination.

Diversions

The diversions were based on similar type structures previously built within the New Orleans District (Davis Pond Diversion and Old River Auxiliary Control Structure). Construction technique is basic to all large concrete structure construction. Costs for the structures were developed using recent historical data and by ratioing and indexing features from the original template structures. The project costs also include the channel excavation and channel rock armoring for the diversion structures. The channel material will either be placed adjacent for future levees or hydraulically dredged and pumped to create marsh. Major materials are assumed to be hauled in by truck or barged or combination.

Marsh Creation

Marsh creation work was based on using standard industry techniques and equipment operating 24 hours per day 7 days per week. Due to the magnitude of the jobs and potential working environments, it is assumed the larger hydraulic dredges (cutterhead and/or hopper dredges) will be used for excavation and transporting material from borrow site to marsh creation areas. A general cost per distance method was used when specific placement-borrow site combinations were unknown along the Gulf area. Borrow sources include existing O&M, Mississippi River, Gulf of Mexico, and existing bays and lakes.

QUANTITIES

For the levee portions, typical design cross sections with varying heights and foundation types were developed for the various reaches across the State and used to calculate the cost of construction. Based on meetings with PDT members, major assumptions were developed for likely locations of borrow and probable methods of construction.

For the coastal restoration work (marsh creation/beach nourishment), most measurements were based on the excavation quantities and not in-place quantities.

WORK SCHEDULE

For the levee and associated features construction, 2 – 12 hour shifts, 7 days a week were assumed.

EQUIPMENT

Rates used were based on the USACE EP-1110-1-8, Region III and contractor furnished data for specialized equipment.

LABOR

Labor rates were based on historical rates taken from contractor payrolls for similar type work. For the levee and associated work, a labor premium and per diem were assumed.

MATERIAL QUOTES

Current price quotes were obtained from local suppliers.

CONTRACTING PLAN

The estimates assume that there will be one prime contractor that performs the work on each contract. At this time, the acquisition plans are unknown.

INDIRECT COSTS

Overhead, profit, and bond were assessed as one total percentage markup to the direct costs based on historical data and current market conditions for the various features of work.

CONTINGENCY

This study document represents many projects and alternatives that are very preliminary in nature, yet it was decided that it was necessary to present a contingency to better portray a likely cost value for the final array of alternatives. Since the projects are very preliminary, a risk-based study of the many alternatives would likely provide no better value or confidence in a contingency calculation. It was realized that the majority of alternatives are similar in nature to those included within the Greater New Orleans Hurricane and Storm Damage Risk Reduction System (GNO HSDRRS) program. After much consideration, it was accepted that the contingency should be a value similar to the risk-based contingencies calculated on the GNO HSDRRS program. For this reason, a value of 50% was chosen as the likely value that can be supported through comparable studies on a similar program. Initial screening and MCDA analysis were run using estimates with a 25% contingency based on USACE guidance in ER1110-2-1302 for Recon level projects. The revised 50% contingency was then applied to the final array of alternatives based on the results of the contingency development.

COST UNCERTAINTY

This section addresses the risk and uncertainty analysis of the design and cost estimate components of the study. We do recognize the many areas of risk and uncertainty inherent in this study. Some of the major areas of risk and uncertainty are listed below.

a. Survey Information. No additional surveys were taken for the LACPR technical evaluation. Elevations were taken from existing LIDAR surveys and other existing survey information. Construction quantities could be affected by elevation assumptions.

b. Geotechnical Information. No additional borings were taken for LACPR. Assumptions made about foundation conditions could affect channel and structure excavation slopes, foundation requirements for structures, and levee stability and settlement estimates. Estimates of future maintenance requirements could also be affected.

c. Borrow Material. Using available data, assumptions were made about the availability and suitability of borrow material for construction of levees, marsh, and barrier islands. Unit prices would be affected if material for construction is not available as anticipated.

d. Relocations. Estimates of required pipeline relocations were developed using existing pipeline maps. Other relocations such as water, sewerage, gas, electric, cable, etc. were estimated. Road and railroad relocations were also estimated using existing maps. Other pipelines, roads and railroads may exist that are not located on the maps. These are not expected to be significant and opportunities exist for avoidance of at least some of these relocations.

e. Project effects. An attempt was made to identify and quantify effects of the features on other areas such as drainage, navigation, flood protection, etc. These costs are included in the total cost. Because of the limited nature of this analysis, all project impacts may not have been accounted for.

f. HTRW. Because of the magnitude of the areas covered no HTRW investigations of specific measure sites were conducted. There should be enough latitude in the location of specific measures to avoid most HTRW problems.

STRUCTURAL ALTERNATIVE MEASURES

LEVEES

Cost estimates were developed for each of the levee sections shown on Plates 1,2 and 3. for 25 foot, 30 foot and 40 foot elevations for both soil mixing and geotextile designs. No geotextile sections were developed for soil reaches 1 and 2 for the 40 foot elevation

levee because the anticipated settlement was large enough to make constructing this levee impractical. Existing LIDAR information was used to determine the approximate ground elevation for each section.

These estimates were interpolated/extrapolated to provide estimates for levee elevations not specifically estimated. Estimates were also developed for lift construction necessary due to settlement (lifts 2 to 5 as required) in the same fashion as the initial costs. These estimates were also interpolated/extrapolated to provide additional estimates as necessary. For revised alignments such as those shown on Plates 4 and 5 already developed cost per mile estimates were used based on the developed section's assumed ground elevation and proximity to the revised section.

STRUCTURES

Estimates for structures were taken from a report produced in May 2006 by the URS Group in association with Brown Cunningham & Gannuch, Inc. for the New Orleans District titled "Preliminary Technical Report for the Louisiana Coastal Protection and Restoration Study Structural Components". Costs were developed for structure elevations of 30 feet, 35 feet and 45 feet because it was assumed that the structures would need to be 5 feet higher than the adjacent levees. These costs were also interpolated/extrapolated to provide estimates for additional elevations. A determination was made as to which structures fit into which levee sections and these costs were added to come up with a structure cost for each section.

ENVIRONMENTAL STRUCTURES

It was assumed that water control structures would need to be placed in the levee to mitigate for disruption of normal flow. These structures would be sluice gated box culvert structures and would be placed every 5 miles. These structures were assumed to be of a consistent size. The cost for these structures was taken from those developed for the May 2006 report described above.

REAL ESTATE

Real Estate costs were provided based on estimates of Right-of-Way acreage needed for the 25', 30' and 40' elevation levee sections .These estimates were interpolated/extrapolated as needed to develop real estate costs for design elevations.

SECTION COSTS

Costs for each section needed for the proposed levee alignments were developed for the high level, barrier and barrier weir costs for 100-yr, 400-yr and 1000-yr design levels based on the levee design elevations obtained from hydrodynamic modeling results for both the soil mix and geotextile levees. These section costs include real estate, levee, structures and environmental structures. A 25% contingency and 12% E&D was applied

to all levee estimates when the costs were added to individual section sections. Mitigation costs were not included as environmental impacts are being analyzed separately. Costs for required lift construction were also prepared in this manner. Lift costs assume no additional right of way will be required.

RELOCATIONS

Costs were developed for facility relocations. An estimate was made of the number of facilities in different levee reaches and unit costs for the different types of facilities were applied. Summary costs for alternatives were provided and are added in when alignment costs were developed.

ENGINEERING AND DESIGN

Engineering and design (12%) was applied to all levee estimates. Structures estimates already had 6% E&D added so an additional 6% was added to those estimates. E&D was applied when coming up with section costs.

CONSTRUCTION MANAGEMENT

Construction Management costs (8%) were applied to all levee and structure estimates. Construction Management was added when the section costs were combined to make alternatives for first costs and with section costs for lift construction.

ALTERNATIVE COSTS

A determination was made as to which levee sections comprised each alternative. Section costs were combined to provide costs for identified alternatives. Attachment 1 is a chart showing how individual cost components are added together to form structural alternatives and comprehensive plan alternatives. For ease of analysis estimates used for the initial screening and MCDA assumed a geotextile levee section would be used for all sections with elevations of 30 ft or less and a soil mix section would be used for levees with elevations over 30 ft. This process was also followed to determine the costs of additional lift construction for the alternatives. Construction Management and Relocations costs are added during this step. First lift costs and costs for additional lifts required due to settlement were provided for preliminary screening. Attachment 2 gives the description of codes used to identify alignments. Attachments 3-5 show the screening level costs.

ALTERNATIVES SELECTED FOR MCDA

After the preliminary screening, alternatives were selected to carry forward into the Multi- Criteria Decision Analysis (MCDA). Construction schedules were developed and costs by year were provided for the 50 year planning cycle. These costs included lift construction and OMRR&R costs. The final round of MCDA used the present value estimates for alternatives that included the new north shore levees. These estimates were

converted to present worth for the MCDA. The present value estimates for all structural alternatives for the two sea level rise scenarios (high sea level rise and low sea level rise) used in the MCDA are shown in attachment 6. An explanation of how costs were developed for the future sea level rise scenarios is given in the Future Scenarios section below. Present value estimates for the other components of the comprehensive alternatives can be found in the Evaluation Results Appendix.

OPERATION AND MAINTENANCE

Estimates of operation and maintenance cost were prepared based on data from existing locks, structures and barriers of similar size, maintenance requirements and operating criteria (number of hours/days/months, staffing level, 10- to 12-year major rehabilitation cycle, etc.). These costs include yearly operation and maintenance costs as well as refurbishment and major rehabilitation (every 10 to 12 years for major structures).

FUTURE SCENARIOS

For PU1 and PU2 costs were developed for five future scenarios (sea level rise alternative 1, sea level rise alternative 2, degraded coastline, degraded coastline with sea level rise altervative1 and degraded coastline with sea level rise alternative 2). Estimates of future lifts needed for the 5 future degraded coastline and sea level rise scenarios were developed based on information provided by the hydrodynamic modeling group for PU1 and PU2. Similar costs were developed for alternatives for PU3a, PU3b and PU4. Cost estimates were developed by applying a percentage to the initial construction estimates for each section. These percentages were developed based on the fully developed estimates for lifts necessary due to settlement. For purposes of the MCDA it was assumed that these costs would be spread out over a 5 year period starting at year 28 for most cases. Costs per year for each scenario for each alternative were converted into present worth for the MCDA. No future lifts were assumed for 100-yr plans for existing levees. It was assumed that those levees would be raised as needed under current authorization.

INDUCED SURGE LEVEL

Construction of the barriers in PU1 and PU2 increases the surge level in some areas of Plaquemines Parish. Costs were developed for raising levees up to provide the same level of design. These costs were added to the barrier alternatives when the 50 yr schedules were developed.

COASTAL ALTERNATIVE FEATURES

Initial costs for coastal features were developed by the Habitat Evaulation Team based on costs for previous diversion, marsh creation and shore protection projects. These costs were used in the MDCA. Subsequent to this analysis a detailed cost for each measure was developed for determining the cost for the final array of alternatives. Costs were developed for each measure contained in the coastal alternative designated as the

representative plan. Details of this plan and the development of these cost estimates can be found in Annex 1.

MITIGATION

Direct construction impact acreages for each alternative were provided to the Habitat Evaluation Team (HET). The HET developed an estimate of total wetland impacts (acres) for each alternative considered in the MCDA analysis. These acres are mitigated at a rate of 1.5 times the wetland impacts at a per acre cost of $80,000.

NON STRUCTURAL MEASURES

Estimates were developed for nonstructural components of the project alternatives. These components included stand alone nonstructural alternatives for each planning unit and complementary nonstructural measures which combine with structural alternatives. Details of these plans and their costs can be found in the Nonstructural Plan Component Appendix.

COMPREHENSIVE ALTERNATIVES

Attachments 7 to 14 show the first costs for each alternative considered for the final round of MCDA. These tables show the total first costs of the plan components (structural, nonstructural, coastal features, mitigation) for each of four scenarios (1)Low Sea Level Rise, High Employment, Dispersed Population; (2)High Sea Level Rise, High Employment, Dispersed Population; (3) Low Sea Level Rise, Business-as-Usual, Compact Population; (4) High Sea Level Rise, Business-as-Usual, Compact Population. More information on these secenarios can be found in the Evaluation Results Appendix.

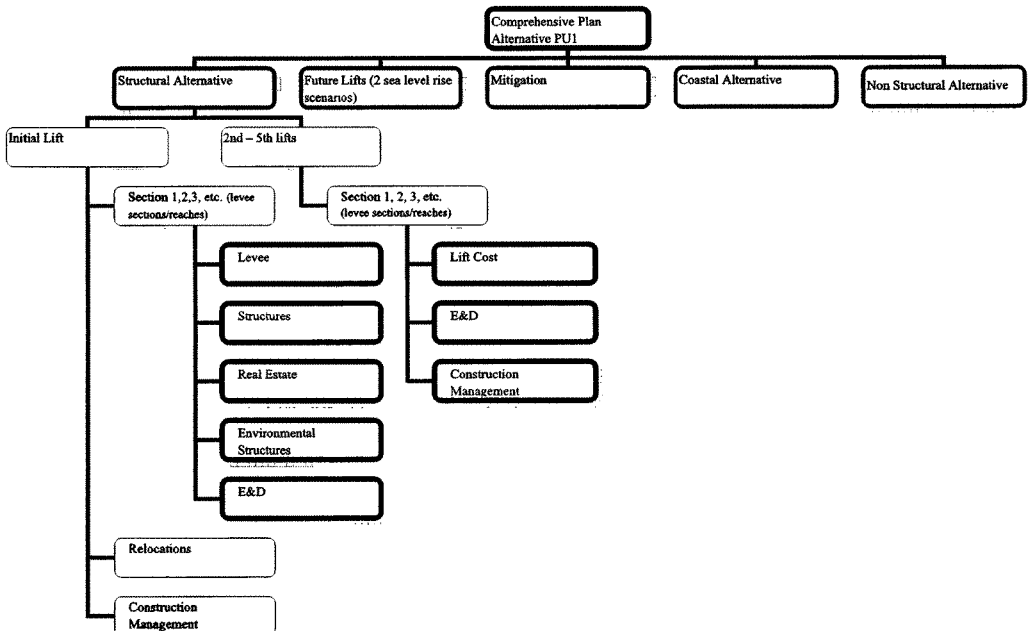
FINAL ARRAY OF ALTERNATIVES

Based on the MCDA analysis and other factors described in detail in the alternatives run through the MCDA analysis were narrowed down to a final array. Tables showing the first cost of the final array of alternatives for the 4 scenarios are attachments 15 to 22. The fully funded costs for these alternatives are shown in attachments 23 to 30. Because of the number of alternatives and measures included in the alternatives escalation was conducted on a broad basis (i.e. the initial lift costs for all structural measures were escalated based on a 14 yr construction period).

ANNEX 2

ATTACHMENTS

HOW INDIVIDUAL COST COMPONENTS ARE ADDED TOGETHER TO FORM COMPREHENSIVE PLAN ALTERNATIVES



1163

Structural Alternative Code Descriptions	
Code	Description
-100-	100-year design level
-400-	400-year design level
-1000-	1000-year design level
PU1	
-HL-	High level alignment (raise existing levees)
-LP-	Lake Pontchartrain Barrier (barrier along eastern edge of Lake Pontchartrain)
-a-	Golden Triangle alignment at the confluence of the GIWW and MRGO
-b-	Alignment at the edge of the Golden Triangle and Lake Borgne
-1	All PU1 primary alternatives include the Lake Pontchartrain and Vicinity Project levees and upper Plaquemines levees. The primary alignments for 'LP' also include a barrier-weir across the mouth of Lake Pontchartrain with a tieback to high ground east of Slidell.
-2	Primary alignment plus North Shore and West Shore levees
-3	Primary alignment plus Slidell and West Shore levees
PU2	
-WBI-	West Bank Interior Plan
-R-	Ridge Alignment Plan
-G-	GIWW Alignment Plan
-1	All PU2 primary alignments include West Bank and Vicinity levees with new sector gate and Larose to Golden Meadow levees. Primary alignments for 'R' and 'G' also include Lafitte ring levees.
-2	Primary alignment plus Boutte levee.
-3	Primary alignment plus Boutte and Des Allemands levee
-4	Primary alignment plus Boutte, Des Allemands and Bayou Lafourche levees.
PU3a	
-M-	Morganza levee alignment
-G-	GIWW alignment with Morganza Levee at 100-year design
-1	Morganza alignment with tieback to high ground west of Morgan City
-2	Morganza alignment with tieback to high ground south of Thibodaux and ring levee around Morgan City
PU3b	
-G-	GIWW levee alignment
-F-	Franklin to Abbeville alignment (inland of the GIWW)
-RL-	Ring levee alignment
-1	Primary alignment for each plan strategy
PU4	
-G-	GIWW levee alignment
-RL-	Ring levee alignment

Structural Alternative Code Descriptions	
Code	Description
-1	For the 'G' alignments, the primary alignment follows the GIWW across the planning unit boundaries.
-2	GIWW alignment with tieback to high ground near Kaplan
-3	GIWW alignment with the levee set at a height of 12 feet.

PU1 STRUCTURAL ALTERNATIVE SCREENING LEVEL DATA
(These costs include the New North Shore Alignments)

Alternative	Initial Construction Cost	Lift Construction Cost
2010 Baseline	0	
HL-1b-100-1	1,745,520,495	1,514,764,657
HL-1a-100-2	8,785,325,555	6,638,900,403
HL-1b-100-2	10,451,476,288	8,041,460,271
HL-1a-100-3	7,020,279,684	6,107,035,746
HL-1b-100-3	8,686,430,418	7,509,595,614
HL-1a-100-4	4,313,385,022	5,122,811,304
HL-1a-400-1	15,791,309,415	11,394,598,771
HL-1b-400-1	14,412,236,305	11,043,782,266
HL-1a-400-2	29,383,241,603	17,519,204,126
HL-1b-400-2	28,004,168,492	17,168,387,622
HL-1a-400-3	26,957,939,133	16,367,085,234
HL-1b-400-3	25,578,866,023	16,016,268,730
HL-1a-400-4	23,487,611,756	14,630,495,855
HL-1a-1000-1	20,001,517,951	15,401,335,856
HL-1b-1000-1	18,457,832,188	14,932,325,865
HL-1a-1000-2	35,834,701,360	23,412,571,422
HL-1b-1000-2	34,291,015,597	22,943,561,431
HL-1a-1000-3	33,016,749,826	21,978,519,543
HL-1b-1000-3	31,473,064,063	21,509,509,553
HL-1a-1000-4	29,297,994,287	19,581,622,737
LP-1a-100-1	3,913,267,880	1,266,286,707
LP-1b-100-1	5,279,572,967	3,671,592,713
LP-1a-100-2	13,082,781,122	2,314,576,101
LP-1b-100-2	14,449,086,209	4,719,882,107
LP-1a-100-3	12,392,322,793	2,259,394,192
LP-1b-100-3	13,758,627,880	4,664,700,198
LP-1a-100-4	10,572,049,851	2,114,953,320
LP-1a-400-1	17,408,404,328	5,094,366,089
LP-1b-400-1	16,095,014,421	4,580,563,907
LP-1a-400-2	31,171,401,963	8,934,582,866
LP-1b-400-2	29,858,012,056	8,420,780,683
LP-1a-400-3	29,166,432,193	8,602,634,333
LP-1b-400-3	27,853,042,286	8,088,832,150
LP-1a-400-4	26,520,710,449	7,906,969,423
LP-1a-1000-1	21,409,906,460	10,340,036,147
LP-1b-1000-1	19,932,291,750	9,910,423,985
LP-1a-1000-2	36,353,467,466	15,627,083,879
LP-1b-1000-2	34,875,852,757	15,197,471,717
LP-1a-1000-3	34,101,023,511	15,159,757,685
LP-1b-1000-3	32,623,408,801	14,730,145,524
LP-1a-1000-4	31,080,352,153	14,230,333,829

PU2 STRUCTURAL ALTERNATIVE SCREENING LEVEL DATA

Alternative	Initial Construction Cost	Lift Construction Cost
2010 Baseline	0	
WBI-1-100-1	558,079,147	0
WBI-1-400-1	8,414,926,201	7,590,582,401
WBI-1-1000-1	10,590,318,061	12,844,897,722
G-1-100-1	3,215,062,909	1,228,353,096
G-1-100-4	6,735,120,146	1,648,013,229
G-1-400-1	10,399,234,254	10,236,300,436
G-1-400-4	14,212,938,761	11,040,446,367
G-1-1000-1	14,479,468,256	8,108,646,906
G-1-1000-4	18,966,804,676	9,092,888,840
S-1-100-2	2,028,604,710	85,022,985
S-1-100-3	3,488,507,563	181,640,090
S-1-100-4	5,250,970,064	264,064,692
S-1-400-2	9,923,789,500	7,793,906,097
S-1-400-3	11,434,191,684	8,082,076,283
S-1-400-4	13,270,561,770	8,252,921,259
S-1-1000-2	12,142,175,542	13,295,249,810
S-1-1000-3	13,703,077,057	13,655,462,542
S-1-1000-4	15,576,400,935	13,840,615,785
G-1-100-1 Mod	3,834,105,224	2,246,646,259
G-1-100-4 Mod	7,354,162,461	2,666,306,393
G-1-400-1 Mod	11,795,497,304	11,704,146,647
G-1-400-4 Mod	15,609,201,810	12,508,292,578
G-1-1000-1 Mod	16,505,376,386	9,888,779,256
G-1-1000-4 Mod	20,992,712,806	10,873,021,190

PU3a, PU3b and PU4 Structural AlternativeScreening Level Data

Planning Unit	Alternative	Initial Construction Cost	Lift Construction Cost
3a	PU3a-M-100-1	8,834,444,198	7,599,927,934
	PU3a-M-400-1	18,069,117,800	10,393,265,664
	PU3a-M-1000-1	26,998,357,687	11,037,462,960
	PU3a-R-100-1	7,369,391,402	7,395,065,187
	PU3a-R-400-1	15,756,864,767	8,544,050,766
	PU3a-R-1000-1	23,524,427,673	7,994,583,399
	PU3a-R-400-2	9,747,971,436	9,555,950,789
	PU3a-R-1000-2	10,436,693,037	10,598,592,632
3b	PU3b-G-100-1	5,775,808,324	5,350,086,236
	PU3b-G-400-1	9,541,382,432	9,309,375,473
	PU3b-G-1000-1	13,575,608,293	12,942,754,321
	PU3b-F-100-1	5,749,115,798	4,568,436,658
	PU3b-F-400-1	9,965,622,237	9,386,033,450
	PU3b-F-1000-1	13,669,716,330	12,974,456,701
	PU3b-R-100-1	4,989,399,196	3,682,421,491
	PU3b-R-400-1	7,644,559,713	7,344,209,324
	PU3b-R-1000-1	10,033,658,198	10,030,286,233
4	PU4-G-100-1	5,660,473,640	2,673,206,892
	PU4-G-400-1	8,310,791,762	4,844,934,065
	PU4-G-1000-1	11,332,561,787	6,286,248,875
	PU4-G-100-2	5,514,130,296	2,873,115,070
	PU4-G-400-2	8,017,760,861	5,125,563,727
	PU4-G-1000-2	11,015,781,189	6,653,758,502
	PU4-G-400-3	5,479,989,968	2,638,928,792
	PU4-G-1000-3	5,681,635,608	2,784,026,967
	PU4-R-100-1	1,449,786,807	162,548,182
	PU-R-400-1	1,825,499,941	467,024,587
	PU-R-1000-1	2,012,766,425	620,455,594

Present Value Estimates for LACPR Structural Alternatives Carried Forward for MCDA Analysis

This table shows the present value estimates for those alternatives for which the 50-yr cost were developed (see attachment 25 and attachment 35)

Present value estimates are presented for two scenarios for each alternative - degraded coastline with a low future relative sea level rise and degraded coastline with a high future sea level rise

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost of Structural Measures $ (Billions)
1	PU1-LP-a-100-1	L	7.02
		H	7.13
	PU1-LP-a-100-2	L	22.44
		H	22.58
	PU1-LP-a-100-3	L	21.09
		H	21.28
	PU1-LP-b-400-1	L	25.54
		H	25.62
	PU1-LP-b-400-3	L	45.08
		H	45.24
	PU1-LP-b-1000-1	L	33.33
		H	33.56
	PU1-LP-b-1000-2	L	59.40
		H	59.61
	PU1-HL-a-100-3	L	15.89
		H	15.93
	PU1-HL-a-100-2	L	19.19
		H	19.25
	PU1-HL-b-400-2	L	49.57
		H	49.81
	PU1-HL-b-400-3	L	44.90
		H	45.10
2	PU2-WBI-100-1	L	1.00
		H	1.02
	PU2-R-100-2	L	7.73
		H	7.77
	PU2-R-400-2	L	25.41
		H	25.52
	PU2-R-100-4	L	13.35
		H	13.42

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost of Structural Measures $ (Billions)
	PU-2-R-400-4	L	31.47
		H	31.61
	PU2-G-1000-4	L	42.34
		H	42.46
	PU2-G-100-1	L	7.60
		H	7.75
	PU2-G-100-4	L	14.52
		H	14.70
	PU2-G-400-4	L	34.75
		H	34.88
	PU2-R-1000-4	L	39.17
		H	39.77
	PU2-R-100-3	L	10.15
		H	10.20
	PU2-WBI-400-1	L	18.29
		H	18.32
	PU2-R-400-3	L	28.32
		H	28.44
3a	PU3a-M-100-1	L	21.41
		H	21.93
	PU3a-M-100-2	L	18.98
		H	19.10
	PU3a-G-400-2	L	25.21
		H	25.28
	PU3a-G-1000-2	L	27.63
		H	27.70
3b	PU3b-G-100-1	L	15.21
		H	15.24
	PU3b-F-100-1	L	13.92
		H	13.95
	PU3b-F-400-1	L	23.44
		H	23.64
	PU3b-F-1000-1	L	31.07
		H	31.09
	PU3b-RL-100-1	L	11.58
		H	11.61
	PU3b-RL-400-1	L	18.00
		H	18.02

Planning Unit	Alternative	Scenario (H=high sea level rise, L=low sea level rise)	Present Value Cost of Structural Measures $ (Billions)
4	PU4-G-100-1	L	11.99
		H	12.27
	PU4-G-100-2	L	11.78
		H	12.07
	PU4-G-400-3	L	11.73
		H	12.01
	PU4-G-1000-3	L	12.21
		H	12.49
	PU4-RL-100-1	L	2.70
		H	2.72
	PU4-RL-400-1	L	3.47
		H	3.49
	PU4-RL-1000-1	L	3.76
		H	3.80

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	9,448,607,969	9,448,607,969
NS-100	0	0	0	4,528,178,444	9,448,607,969	13,976,786,412
NS-400	0	0	0	16,540,266,776	9,448,607,969	25,988,874,745
NS-1000	0	0	0	26,978,007,723	9,448,607,969	36,426,615,692
HL-a-100-2	15,424,225,958	3,437,769,526	498,840,000	0	9,448,607,969	28,809,443,453
HL-a-100-3	13,127,315,431	2,611,728,059	434,280,000	0	9,448,607,969	25,621,931,458
HL-b-400-2	45,172,556,114	7,798,380,255	720,480,000	0	9,448,607,969	63,140,024,338
HL-b-400-3	41,595,134,753	6,663,338,700	655,800,000	0	9,448,607,969	58,362,881,421
LP-a-100-1	5,179,554,587	1,369,386,540	117,600,000	0	9,448,607,969	16,115,149,095
LP-a-100-2	15,397,357,222	4,384,071,019	496,440,000	0	9,448,607,969	29,726,476,210
LP-a-100-3	14,651,716,985	4,498,121,995	438,840,000	0	9,448,607,969	29,037,286,948
LP-b-400-1	20,675,578,328	4,615,035,804	508,560,000	0	9,448,607,969	35,247,782,100
LP-b-400-3	35,941,874,436	8,309,389,865	902,880,000	0	9,448,607,969	54,602,752,270
LP-b-1000-1	29,842,715,735	5,622,186,542	612,000,000	0	9,448,607,969	45,525,510,246
LP-b-1000-2	50,073,324,474	10,731,555,892	1,092,000,000	0	9,448,607,969	71,345,488,335
C-HL-a-100-2	15,424,225,958	3,437,769,526	498,840,000	2,032,064,365	9,448,607,969	30,841,507,818
C-HL-a-100-3	13,127,315,431	2,611,728,059	434,280,000	2,176,427,081	9,448,607,969	27,798,358,539
C-HL-b-400-2	45,172,556,114	7,798,380,255	720,480,000	2,226,391,741	9,448,607,969	65,366,416,079
C-HL-b-400-3	41,595,134,753	6,663,338,700	655,800,000	2,385,407,606	9,448,607,969	60,748,289,027
C-LP-a-100-1	5,179,554,587	1,369,386,540	117,600,000	2,715,056,941	9,448,607,969	18,830,206,036
C-LP-a-100-2	15,397,357,222	4,384,071,019	496,440,000	1,978,506,595	9,448,607,969	31,704,982,805
C-LP-a-100-3	14,651,716,985	4,498,121,995	438,840,000	2,088,918,473	9,448,607,969	31,126,205,421
C-LP-b-400-1	20,675,578,328	4,615,035,804	508,560,000	4,071,547,228	9,448,607,969	39,319,329,328
C-LP-b-400-3	35,941,874,436	8,309,389,865	902,880,000	2,219,853,100	9,448,607,969	56,822,605,370
C-LP-b-1000-1	29,842,715,735	5,622,186,542	612,000,000	4,478,468,683	9,448,607,969	50,003,978,929
C-LP-b-1000-2	50,073,324,474	10,731,555,892	1,092,000,000	2,412,785,037	9,448,607,969	73,758,273,372
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

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Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	11,290,051,065	11,290,051,065
NS-100	0	0	0	2,991,213,525	11,290,051,065	14,281,264,591
NS-400	0	0	0	10,717,863,605	11,290,051,065	22,007,914,671
NS-1000	0	0	0	20,231,577,299	11,290,051,065	31,521,628,364
G-100-1	6,080,751,483	1,040,725,735	116,315,116	0	11,290,051,065	18,527,843,400
G-100-4	10,020,468,854	2,438,283,154	268,943,346	0	11,290,051,065	24,017,746,420
G-400-4	28,117,494,389	5,640,029,388	891,803,099	0	11,290,051,065	45,939,377,941
G-1000-4	31,865,733,996	6,992,208,363	1,134,996,570	0	11,290,051,065	51,282,989,993
R-100-2	2,113,627,695	523,680,206	84,540,000	0	11,290,051,065	14,011,898,966
R-100-3	3,670,147,652	1,093,042,318	119,220,000	0	11,290,051,065	16,172,461,036
R-100-4	5,515,034,756	1,780,402,694	196,236,000	0	11,290,051,065	18,781,724,515
R-400-2	17,717,695,598	3,852,946,060	526,987,200	0	11,290,051,065	33,387,679,923
R-400-3	19,516,267,967	4,442,002,912	562,416,000	0	11,290,051,065	35,810,737,945
R-400-4	21,523,483,028	5,158,187,245	638,700,000	0	11,290,051,065	38,610,421,339
R-1000-4	29,417,016,719	5,164,843,592	814,560,000	0	11,290,051,065	46,686,471,376
WBI-100-1	558,079,147	100,454,247	0	0	11,290,051,065	11,948,584,459
WBI-400-1	16,005,508,602	3,272,297,563	442,464,000	0	11,290,051,065	31,010,321,230
C-G-100-1	6,080,751,483	1,040,725,735	116,315,116	2,142,418,805	11,290,051,065	20,670,262,205
C-G-100-4	10,020,468,854	2,438,283,154	268,943,346	1,795,574,148	11,290,051,065	25,813,320,568
C-G-400-4	28,117,494,389	5,640,029,388	891,803,099	1,607,311,035	11,290,051,065	47,546,688,976
C-G-1000-4	31,865,733,996	6,992,208,363	1,134,996,570	2,090,044,979	11,290,051,065	53,373,034,972
C-R-100-2	2,113,627,695	523,680,206	84,540,000	2,364,045,973	11,290,051,065	16,375,944,939
C-R-100-3	3,670,147,652	1,093,042,318	119,220,000	2,116,041,838	11,290,051,065	18,288,502,874
C-R-100-4	5,515,034,756	1,780,402,694	196,236,000	1,991,495,222	11,290,051,065	20,773,219,737
C-R-400-2	17,717,695,598	3,852,946,060	526,987,200	1,656,204,913	11,290,051,065	35,243,884,836
C-R-400-3	19,516,267,967	4,442,002,912	562,416,000	1,752,368,134	11,290,051,065	37,563,106,079
C-R-400-4	21,523,483,028	5,158,187,245	638,700,000	1,605,444,613	11,290,051,065	40,215,865,952
C-R-1000-4	29,417,016,719	5,164,843,592	814,560,000	1,963,085,071	11,290,051,065	48,649,556,447
C-WBI-100-1	558,079,147	100,454,247	0	2,959,561,863	11,290,051,065	14,908,146,322
C-WBI-400-1	16,005,508,602	3,272,297,563	442,464,000	2,437,692,699	11,290,051,065	33,448,013,929
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	9,637,580,128	9,637,580,128
NS-100	0	0	0	4,528,178,444	9,637,580,128	14,165,758,572
NS-400	0	0	0	16,540,266,776	9,637,580,128	26,177,846,904
NS-1000	0	0	0	26,978,007,723	9,637,580,128	36,615,587,851
HL-a-100-2	15,424,225,958	3,554,039,980	498,840,000	0	9,637,580,128	29,114,686,066
HL-a-100-3	13,127,315,431	2,682,107,320	434,280,000	0	9,637,580,128	25,881,282,879
HL-b-400-2	45,172,556,114	8,285,298,983	720,480,000	0	9,637,580,128	63,815,915,224
HL-b-400-3	41,595,134,753	7,087,199,563	655,800,000	0	9,637,580,128	58,975,714,443
LP-a-100-1	5,179,554,587	1,590,049,526	117,600,000	0	9,637,580,128	16,524,784,241
LP-a-100-2	15,397,357,222	4,667,040,399	496,440,000	0	9,637,580,128	30,198,417,749
LP-a-100-3	14,651,716,985	4,878,897,794	438,840,000	0	9,637,580,128	29,607,034,907
LP-b-400-1	20,675,578,328	4,782,852,651	508,560,000	0	9,637,580,128	35,604,571,107
LP-b-400-3	35,941,874,436	8,629,126,019	902,880,000	0	9,637,580,128	55,111,460,583
LP-b-1000-1	29,842,715,735	6,076,106,784	612,000,000	0	9,637,580,128	46,168,402,646
LP-b-1000-2	50,073,324,474	11,154,040,058	1,092,000,000	0	9,637,580,128	71,956,944,660
C-HL-a-100-2	15,424,225,958	3,554,039,980	498,840,000	2,032,064,365	9,637,580,128	31,146,750,431
C-HL-a-100-3	13,127,315,431	2,682,107,320	434,280,000	2,176,427,081	9,637,580,128	28,057,709,960
C-HL-b-400-2	45,172,556,114	8,285,298,983	720,480,000	2,226,391,741	9,637,580,128	66,042,306,965
C-HL-b-400-3	41,595,134,753	7,087,199,563	655,800,000	2,385,407,606	9,637,580,128	61,361,122,049
C-LP-a-100-1	5,179,554,587	1,590,049,526	117,600,000	2,715,056,941	9,637,580,128	19,239,841,182
C-LP-a-100-2	15,397,357,222	4,667,040,399	496,440,000	1,978,506,595	9,637,580,128	32,176,924,344
C-LP-a-100-3	14,651,716,985	4,878,897,794	438,840,000	2,088,918,473	9,637,580,128	31,695,953,380
C-LP-b-400-1	20,675,578,328	4,782,852,651	508,560,000	4,071,547,228	9,637,580,128	39,676,118,335
C-LP-b-400-3	35,941,874,436	8,629,126,019	902,880,000	2,219,853,100	9,637,580,128	57,331,313,684
C-LP-b-1000-1	29,842,715,735	6,076,106,784	612,000,000	4,478,468,683	9,637,580,128	50,646,871,330
C-LP-b-1000-2	50,073,324,474	11,154,040,058	1,092,000,000	2,412,785,037	9,637,580,128	74,369,729,697

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0		0
NS-100	0	0	0	2,991,213,525	11,312,631,167	14,303,844,693
NS-400	0	0	0	10,717,863,605	11,312,631,167	22,030,494,773
NS-1000	0	0	0	20,231,577,299	11,312,631,167	31,544,208,466
G-100-1	6,080,751,483	1,332,810,890	116,315,116	0	11,312,631,167	18,842,508,657
G-100-4	10,020,468,854	2,802,037,920	268,943,346	0	11,312,631,167	24,404,081,288
G-400-4	28,117,494,389	5,914,689,535	891,803,099	0	11,312,631,167	46,236,618,190
G-1000-4	31,865,733,996	7,242,027,428	1,134,996,570	0	11,312,631,167	51,555,389,161
R-100-2	2,113,627,695	586,214,541	84,540,000	0	11,312,631,167	14,097,013,403
R-100-3	3,670,147,652	1,184,774,710	119,220,000	0	11,312,631,167	16,286,773,530
R-100-4	5,515,034,756	1,907,384,336	196,236,000	0	11,312,631,167	18,931,286,259
R-400-2	17,717,695,598	4,050,607,170	526,987,200	0	11,312,631,167	33,607,921,135
R-400-3	19,516,267,967	4,669,872,066	562,416,000	0	11,312,631,167	36,061,187,200
R-400-4	21,523,483,028	5,422,783,801	638,700,000	0	11,312,631,167	38,897,597,997
R-1000-4	29,417,016,719	6,367,984,244	814,560,000	0	11,312,631,167	47,912,192,131
WBI-100-1	558,079,147	150,681,370	0	0	11,312,631,167	12,021,391,685
WBI-400-1	16,005,508,602	3,440,181,827	442,464,000	0	11,312,631,167	31,200,785,597
C-G-100-1	6,080,751,483	1,332,810,890	116,315,116	2,142,418,805	11,312,631,167	20,984,927,462
C-G-100-4	10,020,468,854	2,802,037,920	268,943,346	1,795,574,148	11,312,631,167	26,199,655,436
C-G-400-4	28,117,494,389	5,914,689,535	891,803,099	1,607,311,035	11,312,631,167	47,843,929,224
C-G-1000-4	31,865,733,996	7,242,027,428	1,134,996,570	2,090,044,979	11,312,631,167	53,645,434,140
C-R-100-2	2,113,627,695	586,214,541	84,540,000	2,364,045,973	11,312,631,167	16,461,059,375
C-R-100-3	3,670,147,652	1,184,774,710	119,220,000	2,116,041,838	11,312,631,167	18,402,815,368
C-R-100-4	5,515,034,756	1,907,384,336	196,236,000	1,991,495,222	11,312,631,167	20,922,781,481
C-R-400-2	17,717,695,598	4,050,607,170	526,987,200	1,856,204,913	11,312,631,167	35,464,126,048
C-R-400-3	19,516,267,967	4,669,872,066	562,416,000	1,752,368,134	11,312,631,167	37,813,555,335
C-R-400-4	21,523,483,028	5,422,783,801	638,700,000	1,605,444,613	11,312,631,167	40,503,042,610
C-R-1000-4	29,417,016,719	6,367,984,244	814,560,000	1,963,085,071	11,312,631,167	49,875,277,202
C-WBI-100-1	558,079,147	150,681,370	0	2,959,561,863	11,312,631,167	14,980,953,548
C-WBI-400-1	16,005,508,602	3,440,181,827	442,464,000	2,437,692,699	11,312,631,167	33,638,478,296
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	9,448,607,969	9,448,607,969
NS-100	0	0	0	4,258,734,736	9,448,607,969	13,707,342,705
NS-400	0	0	0	19,409,614,275	9,448,607,969	28,858,222,244
NS-1000	0	0	0	28,240,698,887	9,448,607,969	37,689,306,856
HL-a-100-2	15,424,225,958	3,437,769,526	498,840,000	0	9,448,607,969	28,809,443,453
HL-a-100-3	13,127,315,431	2,611,728,059	434,280,000	0	9,448,607,969	25,621,931,458
HL-b-400-2	45,172,556,114	7,798,380,255	720,480,000	0	9,448,607,969	63,140,024,338
HL-b-400-3	41,595,134,753	6,663,338,700	655,800,000	0	9,448,607,969	58,362,881,421
LP-a-100-1	5,179,554,587	1,369,386,540	117,600,000	0	9,448,607,969	16,115,149,095
LP-a-100-2	15,397,357,222	4,384,071,019	496,440,000	0	9,448,607,969	29,726,476,210
LP-a-100-3	14,651,716,985	4,498,121,995	438,840,000	0	9,448,607,969	29,037,286,948
LP-b-400-1	20,675,578,328	4,615,035,804	508,560,000	0	9,448,607,969	35,247,782,100
LP-b-400-3	35,941,874,436	8,309,389,865	902,880,000	0	9,448,607,969	54,602,752,270
LP-b-1000-1	29,842,715,735	5,622,186,542	612,000,000	0	9,448,607,969	45,525,510,246
LP-b-1000-2	50,073,324,474	10,731,555,892	1,092,000,000	0	9,448,607,969	71,345,488,335
C-HL-a-100-2	15,424,225,958	3,437,769,526	498,840,000	1,581,952,286	9,448,607,969	30,391,395,739
C-HL-a-100-3	13,127,315,431	2,611,728,059	434,280,000	1,742,427,542	9,448,607,969	27,364,359,000
C-HL-b-400-2	45,172,556,114	7,798,380,255	720,480,000	1,756,733,800	9,448,607,969	64,896,758,138
C-HL-b-400-3	41,595,134,753	6,663,338,700	655,800,000	2,052,801,484	9,448,607,969	60,415,682,905
C-LP-a-100-1	5,179,554,587	1,369,386,540	117,600,000	2,309,375,337	9,448,607,969	18,424,524,432
C-LP-a-100-2	15,397,357,222	4,384,071,019	496,440,000	1,482,382,332	9,448,607,969	31,208,858,542
C-LP-a-100-3	14,651,716,985	4,498,121,995	438,840,000	1,605,113,419	9,448,607,969	30,642,400,367
C-LP-b-400-1	20,675,578,328	4,615,035,804	508,560,000	4,517,615,291	9,448,607,969	39,765,397,390
C-LP-b-400-3	35,941,874,436	8,309,389,865	902,880,000	1,767,505,945	9,448,607,969	56,370,258,215
C-LP-b-1000-1	29,842,715,735	5,622,186,542	612,000,000	5,760,652,936	9,448,607,969	51,286,163,182
C-LP-b-1000-2	50,073,324,474	10,731,555,892	1,092,000,000	1,699,495,398	9,448,607,969	73,044,983,733

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	11,290,051,065	11,290,051,065
NS-100	0	0	0	3,081,075,201	11,290,051,065	14,371,126,266
NS-400	0	0	0	19,409,614,275	11,290,051,065	30,699,665,341
NS-1000	0	0	0	28,240,698,887	11,290,051,065	39,530,749,953
G-100-1	6,080,751,483	1,040,725,735	116,315,116	0	11,290,051,065	18,527,843,400
G-100-4	10,020,468,854	2,438,283,154	268,943,346	0	11,290,051,065	24,017,746,420
G-400-4	28,117,494,389	5,640,029,388	891,803,099	0	11,290,051,065	45,939,377,941
G-1000-4	31,865,733,996	6,992,208,363	1,134,996,570	0	11,290,051,065	51,282,989,993
R-100-2	2,113,627,695	523,680,206	84,540,000	0	11,290,051,065	14,011,898,966
R-100-3	3,670,147,652	1,093,042,318	119,220,000	0	11,290,051,065	16,172,461,036
R-100-4	5,515,034,756	1,780,402,694	196,236,000	0	11,290,051,065	18,781,724,515
R-400-2	17,717,695,598	3,852,946,060	526,987,200	0	11,290,051,065	33,387,679,923
R-400-3	19,516,267,967	4,442,002,912	562,416,000	0	11,290,051,065	35,810,737,945
R-400-4	21,523,483,028	5,158,187,245	638,700,000	0	11,290,051,065	38,610,421,339
R-1000-4	29,417,016,719	5,164,843,592	814,560,000	0	11,290,051,065	46,686,471,376
WBI-100-1	558,079,147	100,454,247	0	0	11,290,051,065	11,948,584,459
WBI-400-1	16,005,508,602	3,272,297,563	442,464,000	0	11,290,051,065	31,010,321,230
C-G-100-1	6,080,751,483	1,040,725,735	116,315,116	2,194,965,309	11,290,051,065	20,722,808,709
C-G-100-4	10,020,468,854	2,438,283,154	268,943,346	1,823,625,898	11,290,051,065	25,841,372,318
C-G-400-4	28,117,494,389	5,640,029,388	891,803,099	2,094,957,764	11,290,051,065	48,034,335,706
C-G-1000-4	31,865,733,996	6,992,208,363	1,134,996,570	2,449,946,391	11,290,051,065	53,732,936,384
C-R-100-2	2,113,627,695	523,680,206	84,540,000	2,386,992,423	11,290,051,065	16,398,891,388
C-R-100-3	3,670,147,652	1,093,042,318	119,220,000	2,110,551,526	11,290,051,065	18,283,012,562
C-R-100-4	5,515,034,756	1,780,402,694	196,236,000	2,011,195,484	11,290,051,065	20,792,919,999
C-R-400-2	17,717,695,598	3,852,946,060	526,987,200	2,823,354,187	11,290,051,065	36,211,034,110
C-R-400-3	19,516,267,967	4,442,002,912	562,416,000	2,508,167,425	11,290,051,065	38,318,905,370
C-R-400-4	21,523,483,028	5,158,187,245	638,700,000	2,294,124,391	11,290,051,065	40,904,545,730
C-R-1000-4	29,417,016,719	5,164,843,592	814,560,000	2,408,216,218	11,290,051,065	49,094,687,594
C-WBI-100-1	558,079,147	100,454,247	0	3,050,872,364	11,290,051,065	14,999,456,823
C-WBI-400-1	16,005,508,602	3,272,297,563	442,464,000	3,617,595,484	11,290,051,065	34,627,916,714
[redacted]						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

11177

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	9,637,580,128	9,637,580,128
NS-100	0	0	0	4,258,734,736	9,637,580,128	13,896,314,864
NS-400	0	0	0	19,409,614,275	9,637,580,128	29,047,194,403
NS-1000	0	0	0	28,240,698,887	9,637,580,128	37,878,279,015
HL-a-100-2	15,424,225,958	3,554,039,980	498,840,000	0	9,637,580,128	29,114,686,066
HL-a-100-3	13,127,315,431	2,682,107,320	434,280,000	0	9,637,580,128	25,881,282,879
HL-b-400-2	45,172,556,114	8,285,298,983	720,480,000	0	9,637,580,128	63,815,915,224
HL-b-400-3	41,595,134,753	7,087,199,563	655,800,000	0	9,637,580,128	58,975,714,443
LP-a-100-1	5,179,554,587	1,590,049,526	117,600,000	0	9,637,580,128	16,524,784,241
LP-a-100-2	15,397,357,222	4,667,040,399	496,440,000	0	9,637,580,128	30,198,417,749
LP-a-100-3	14,651,716,985	4,878,897,794	438,840,000	0	9,637,580,128	29,607,034,907
LP-b-400-1	20,675,578,328	4,782,852,651	508,560,000	0	9,637,580,128	35,604,571,107
LP-b-400-3	35,941,874,436	8,629,126,019	902,880,000	0	9,637,580,128	55,111,460,583
LP-b-1000-1	29,842,715,735	6,076,106,784	612,000,000	0	9,637,580,128	46,168,402,646
LP-b-1000-2	50,073,324,474	11,154,040,058	1,092,000,000	0	9,637,580,128	71,956,944,660
C-HL-a-100-2	15,424,225,958	3,554,039,980	498,840,000	1,581,952,286	9,637,580,128	30,696,638,352
C-HL-a-100-3	13,127,315,431	2,682,107,320	434,280,000	1,742,427,542	9,637,580,128	27,623,710,421
C-HL-b-400-2	45,172,556,114	8,285,298,983	720,480,000	1,756,733,800	9,637,580,128	65,572,649,024
C-HL-b-400-3	41,595,134,753	7,087,199,563	655,800,000	2,052,801,484	9,637,580,128	61,028,515,928
C-LP-a-100-1	5,179,554,587	1,590,049,526	117,600,000	2,309,375,337	9,637,580,128	18,834,159,578
C-LP-a-100-2	15,397,357,222	4,667,040,399	496,440,000	1,482,382,332	9,637,580,128	31,680,800,081
C-LP-a-100-3	14,651,716,985	4,878,897,794	438,840,000	1,605,113,419	9,637,580,128	31,212,148,326
C-LP-b-400-1	20,675,578,328	4,782,852,651	508,560,000	4,517,615,291	9,637,580,128	40,122,186,398
C-LP-b-400-3	35,941,874,436	8,629,126,019	902,880,000	1,767,505,945	9,637,580,128	56,878,966,529
C-LP-b-1000-1	29,842,715,735	6,076,106,784	612,000,000	5,760,652,936	9,637,580,128	51,929,055,582
C-LP-b-1000-2	50,073,324,474	11,154,040,058	1,092,000,000	1,699,495,398	9,637,580,128	73,656,440,058
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	11,312,631,167	11,312,631,167
NS-100	0	0	0	3,081,075,201	11,312,631,167	14,393,706,368
NS-400	0	0	0	19,409,614,275	11,312,631,167	30,722,245,443
NS-1000	0	0	0	28,240,698,887	11,312,631,167	39,553,330,055
G-100-1	6,080,751,483	1,332,810,890	116,315,116	0	11,312,631,167	18,842,508,657
G-100-4	10,020,468,854	2,802,037,920	268,943,346	0	11,312,631,167	24,404,081,288
G-400-4	28,117,494,389	5,914,689,535	891,803,099	0	11,312,631,167	46,236,618,190
G-1000-4	31,865,733,996	7,242,027,428	1,134,996,570	0	11,312,631,167	51,555,389,161
R-100-2	2,113,627,695	586,214,541	84,540,000	0	11,312,631,167	14,097,013,403
R-100-3	3,670,147,652	1,184,774,710	119,220,000	0	11,312,631,167	16,286,773,530
R-100-4	5,515,034,756	1,907,384,336	196,236,000	0	11,312,631,167	18,931,286,259
R-400-2	17,717,695,598	4,050,607,170	526,987,200	0	11,312,631,167	33,607,921,135
R-400-3	19,516,267,967	4,669,872,066	562,416,000	0	11,312,631,167	36,061,187,200
R-400-4	21,523,483,028	5,422,783,801	638,700,000	0	11,312,631,167	38,897,597,997
R-1000-4	29,417,016,719	6,367,984,244	814,560,000	0	11,312,631,167	47,912,192,131
WBI-100-1	558,079,147	150,681,370	0	0	11,312,631,167	12,021,391,685
WBI-400-1	16,005,508,602	3,440,181,827	442,464,000	0	11,312,631,167	31,200,785,597
C-G-100-1	6,080,751,483	1,332,810,890	116,315,116	2,194,965,309	11,312,631,167	21,037,473,966
C-G-100-4	10,020,468,854	2,802,037,920	268,943,346	1,823,625,898	11,312,631,167	26,227,707,186
C-G-400-4	28,117,494,389	5,914,689,535	891,803,099	2,094,957,764	11,312,631,167	48,331,575,954
C-G-1000-4	31,865,733,996	7,242,027,428	1,134,996,570	2,449,946,391	11,312,631,167	54,005,335,552
C-R-100-2	2,113,627,695	586,214,541	84,540,000	2,386,992,423	11,312,631,167	16,484,005,825
C-R-100-3	3,670,147,652	1,184,774,710	119,220,000	2,110,551,526	11,312,631,167	18,397,325,056
C-R-100-4	5,515,034,756	1,907,384,336	196,236,000	2,011,195,484	11,312,631,167	20,942,481,743
C-R-400-2	17,717,695,598	4,050,607,170	526,987,200	2,823,354,187	11,312,631,167	36,431,275,322
C-R-400-3	19,516,267,967	4,669,872,066	562,416,000	2,508,167,425	11,312,631,167	38,569,354,626
C-R-400-4	21,523,483,028	5,422,783,801	638,700,000	2,294,124,391	11,312,631,167	41,191,722,388
C-R-1000-4	29,417,016,719	6,367,984,244	814,560,000	2,408,216,218	11,312,631,167	50,320,408,348
C-WBI-100-1	558,079,147	150,681,370	0	3,050,872,364	11,312,631,167	15,072,264,049
C-WBI-400-1	16,005,508,602	3,440,181,827	442,464,000	3,617,595,484	11,312,631,167	34,818,381,080
[redacted]						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	19,431,425,544	19,431,425,544
NS-100	0	0	0	5,491,870,436	19,431,425,544	24,923,295,980
NS-400	0	0	0	7,474,735,521	19,431,425,544	26,906,161,065
NS-1000	0	0	0	8,192,901,293	19,431,425,544	27,624,326,837
M-100-1	16,434,372,132	1,736,750,667	585,720,000	0	19,431,425,544	38,188,268,343
M-100-2 (was R-100-1)	14,764,456,589	1,769,898,087	504,120,000	0	19,431,425,544	36,469,900,220
G-400-2	19,303,922,225	2,510,303,624	631,320,000	0	19,431,425,544	41,876,971,393
G-1000-2	21,035,285,669	2,729,355,082	797,040,000	0	19,431,425,544	43,993,106,295
C-M-100-1	16,434,372,132	1,736,750,667	585,720,000	442,655,813	19,431,425,544	38,630,924,156
C-M-100-2	14,764,456,589	1,769,898,087	504,120,000	487,883,084	19,431,425,544	36,957,783,303
C-G-400-2	19,303,922,225	2,510,303,624	631,320,000	1,060,167,530	19,431,425,544	42,937,138,923
C-G-1000-2	21,035,285,669	2,729,355,082	797,040,000	1,739,980,750	19,431,425,544	45,733,087,045

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	4,364,994,612	4,364,994,612
NS-100	0	0	0	1,843,806,485	4,364,994,612	6,208,801,097
NS-400	0	0	0	3,186,105,365	4,364,994,612	7,551,099,977
NS-1000	0	0	0	3,974,270,990	4,364,994,612	8,339,265,602
G-100-1	11,125,894,560	2,727,160,513	275,520,000	0	4,364,994,612	18,493,569,685
F-100-1	10,317,552,457	2,707,345,670	295,920,000	0	4,364,994,612	17,685,812,738
F-400-1	19,351,655,687	2,802,312,099	465,360,000	0	4,364,994,612	26,984,322,398
F-1000-1	26,644,173,031	3,654,940,201	622,560,000	0	4,364,994,612	35,286,667,844
RL-100-1	8,671,820,687	2,334,894,790	112,800,000	0	4,364,994,612	15,484,510,089
RL-400-1	14,988,769,036	2,098,139,483	204,240,000	0	4,364,994,612	21,656,143,131
C-G-100-1	11,125,894,560	2,727,160,513	275,520,000	171,091,102	4,364,994,612	18,664,660,787
C-F-100-1	10,317,552,457	2,707,345,670	295,920,000	256,312,147	4,364,994,612	17,942,124,885
C-F-400-1	19,351,655,687	2,802,312,099	465,360,000	205,281,980	4,364,994,612	27,189,604,379
C-F-1000-1	26,644,173,031	3,654,940,201	622,560,000	605,212,664	4,364,994,612	35,891,880,509
C-RL-100-1	8,671,820,687	2,334,894,790	112,800,000	605,212,664	4,364,994,612	16,089,722,754
C-RL-400-1	14,988,769,036	2,098,139,483	204,240,000	695,798,972	4,364,994,612	22,351,942,103

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1181

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	10,295,953,284	10,295,953,284
NS-100	0	0	0	1,503,157,724	10,295,953,284	11,799,111,008
NS-400	0	0	0	2,416,672,397	10,295,953,284	12,712,625,681
NS-1000	0	0	0	3,353,405,565	10,295,953,284	13,649,358,849
G-100-1	8,333,680,532	2,205,061,596	266,520,000	0	10,295,953,284	21,101,215,412
G-100-2	8,387,245,367	2,136,572,912	211,560,000	0	10,295,953,284	21,031,331,562
G-400-3	8,118,918,760	2,118,830,878	297,960,000	0	10,295,953,284	20,831,662,921
G-1000-3	8,465,662,575	2,213,201,037	298,200,000	0	10,295,953,284	21,273,016,896
RL-100-1	1,612,334,989	668,122,326	10,560,000	0	10,295,953,284	12,586,970,598
RL-400-1	2,292,524,528	843,956,072	11,400,000	0	10,295,953,284	13,443,833,884
RL-1000-1	2,633,222,019	931,596,787	11,880,000	0	10,295,953,284	13,872,652,090
C-G-100-1	8,333,680,532	2,205,061,596	266,520,000	1,337,343,944	10,295,953,284	22,438,559,356
C-G-100-2	8,387,245,367	2,136,572,912	211,560,000	1,166,677,999	10,295,953,284	22,198,009,561
C-G-400-3	8,118,918,760	2,118,830,878	297,960,000	2,449,763,253	10,295,953,284	23,281,426,175
C-G-1000-3	8,465,662,575	2,213,201,037	298,200,000	1,018,326,175	10,295,953,284	22,291,343,071
C-RL-100-1	1,612,334,989	668,122,326	10,560,000	1,019,265,296	10,295,953,284	13,606,235,895
C-RL-400-1	2,292,524,528	843,956,072	11,400,000	1,041,121,803	10,295,953,284	14,484,955,687
C-RL-1000-1	2,633,222,019	931,596,787	11,880,000	2,501,242,463	10,295,953,284	16,373,894,553

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	19,625,739,799	19,625,739,799
NS-100	0	0	0	5,491,870,436	19,625,739,799	25,117,610,236
NS-400	0	0	0	7,474,735,521	19,625,739,799	27,100,475,320
NS-1000	0	0	0	8,192,901,293	19,625,739,799	27,818,641,092
M-100-1	16,434,372,132	2,787,977,661	585,720,000	0	19,625,739,799	39,433,809,593
M-100-2 (was R-100-1)	14,764,456,589	2,004,661,651	504,120,000	0	19,625,739,799	36,898,978,039
G-400-2	19,303,922,225	2,657,465,063	631,320,000	0	19,625,739,799	42,218,447,087
G-1000-2	21,035,285,669	2,887,127,039	797,040,000	0	19,625,739,799	44,345,192,507
C-M-100-1	16,434,372,132	2,787,977,661	585,720,000	442,655,813	19,625,739,799	39,876,465,405
C-M-100-2	14,764,456,589	2,004,661,651	504,120,000	487,883,084	19,625,739,799	37,386,861,123
C-G-400-2	19,303,922,225	2,657,465,063	631,320,000	1,060,167,530	19,625,739,799	43,278,614,617
C-G-1000-2	21,035,285,669	2,887,127,039	797,040,000	1,739,980,750	19,625,739,799	46,085,173,257

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No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1183

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	4,408,644,558	4,408,644,558
NS-100	0	0	0	1,843,806,485	4,408,644,558	6,252,451,043
NS-400	0	0	0	3,186,105,365	4,408,644,558	7,594,749,924
NS-1000	0	0	0	3,974,270,990	4,408,644,558	8,382,915,548
G-100-1	11,125,894,560	2,776,494,028	275,520,000	0	4,408,644,558	18,586,553,146
F-100-1	10,317,552,457	2,781,487,033	295,920,000	0	4,408,644,558	17,803,604,048
F-400-1	19,351,655,687	3,144,298,977	465,360,000	0	4,408,644,558	27,369,959,222
F-1000-1	26,644,173,031	3,682,488,092	622,560,000	0	4,408,644,558	35,357,865,681
RL-100-1	8,671,820,687	2,401,488,601	112,800,000	0	4,408,644,558	15,594,753,846
RL-400-1	14,988,769,036	2,154,845,955	204,240,000	0	4,408,644,558	21,756,499,550
C-G-100-1	11,125,894,560	2,776,494,028	275,520,000	171,091,102	4,408,644,558	18,757,644,248
C-F-100-1	10,317,552,457	2,781,487,033	295,920,000	256,312,147	4,408,644,558	18,059,916,195
C-F-400-1	19,351,655,687	3,144,298,977	465,360,000	205,281,980	4,408,644,558	27,575,241,203
C-F-1000-1	26,644,173,031	3,682,488,092	622,560,000	320,696,539	4,408,644,558	35,678,562,221
C-RL-100-1	8,671,820,687	2,401,488,601	112,800,000	605,212,664	4,408,644,558	16,199,966,511
C-RL-400-1	14,988,769,036	2,154,845,955	204,240,000	695,798,972	4,408,644,558	22,452,298,522

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1184

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	10,501,872,350	10,501,872,350
NS-100	0	0	0	1,503,157,724	10,501,872,350	12,005,030,074
NS-400	0	0	0	2,416,672,397	10,501,872,350	12,918,544,746
NS-1000	0	0	0	3,353,405,565	10,501,872,350	13,855,277,914
G-100-1	8,333,680,532	2,780,783,620	266,520,000	0	10,501,872,350	21,882,856,502
G-100-2	8,387,245,367	2,708,490,008	211,560,000	0	10,501,872,350	21,809,167,725
G-400-3	8,118,918,760	2,689,762,306	297,960,000	0	10,501,872,350	21,608,513,415
G-1000-3	8,465,662,575	2,789,375,252	298,200,000	0	10,501,872,350	22,055,110,177
RL-100-1	1,612,334,989	705,240,233	10,560,000	0	10,501,872,350	12,830,007,571
RL-400-1	2,292,524,528	890,842,521	11,400,000	0	10,501,872,350	13,696,639,399
RL-1000-1	2,633,222,019	983,352,164	11,880,000	0	10,501,872,350	14,130,326,533
C-G-100-1	8,333,680,532	2,780,783,620	266,520,000	1,337,343,944	10,501,872,350	23,220,200,446
C-G-100-2	8,387,245,367	2,708,490,008	211,560,000	1,166,677,999	10,501,872,350	22,975,845,724
C-G-400-3	8,118,918,760	2,689,762,306	297,960,000	2,449,763,253	10,501,872,350	24,058,276,669
C-G-1000-3	8,465,662,575	2,789,375,252	298,200,000	1,018,326,175	10,501,872,350	23,073,436,351
C-RL-100-1	1,612,334,989	705,240,233	10,560,000	1,019,265,296	10,501,872,350	13,849,272,867
C-RL-400-1	2,292,524,528	890,842,521	11,400,000	1,041,121,803	10,501,872,350	14,737,761,201
C-RL-1000-1	2,633,222,019	983,352,164	11,880,000	2,501,242,463	10,501,872,350	16,631,568,995

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	19,431,425,544	19,431,425,544
NS-100	0	0	0	5,480,711,485	19,431,425,544	24,912,137,029
NS-400	0	0	0	7,369,688,257	19,431,425,544	26,801,113,801
NS-1000	0	0	0	8,373,166,632	19,431,425,544	27,804,592,176
M-100-1	16,434,372,132	1,736,750,667	585,720,000	0	19,431,425,544	38,188,268,343
M-100-2 (was R-100-1)	14,764,456,589	1,769,898,087	504,120,000	0	19,431,425,544	36,469,900,220
G-400-2	19,303,922,225	2,510,303,624	631,320,000	0	19,431,425,544	41,876,971,393
G-1000-2	21,035,285,669	2,729,355,082	797,040,000	0	19,431,425,544	43,993,106,295
C-M-100-1	16,434,372,132	1,736,750,667	585,720,000	436,881,756	19,431,425,544	38,625,150,100
C-M-100-2	14,764,456,589	1,769,898,087	504,120,000	483,576,961	19,431,425,544	36,953,477,181
C-G-400-2	19,303,922,225	2,510,303,624	631,320,000	1,282,255,156	19,431,425,544	43,159,226,549
C-G-1000-2	21,035,285,669	2,729,355,082	797,040,000	1,919,416,621	19,431,425,544	45,912,522,916

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1186

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	4,364,994,612	4,364,994,612
NS-100	0	0	0	1,893,104,451	4,364,994,612	6,258,099,063
NS-400	0	0	0	3,276,057,346	4,364,994,612	7,641,051,958
NS-1000	0	0	0	4,434,303,424	4,364,994,612	8,799,298,036
G-100-1	11,125,894,560	2,727,160,513	275,520,000	0	4,364,994,612	18,493,569,685
F-100-1	10,317,552,457	2,707,345,670	295,920,000	0	4,364,994,612	17,685,812,738
F-400-1	19,351,655,687	2,802,312,099	465,360,000	0	4,364,994,612	26,984,322,398
F-1000-1	26,644,173,031	3,654,940,201	622,560,000	0	4,364,994,612	35,286,667,844
RL-100-1	8,671,820,687	2,334,894,790	112,800,000	0	4,364,994,612	15,484,510,089
RL-400-1	14,988,769,036	2,098,139,483	204,240,000	0	4,364,994,612	21,656,143,131
C-G-100-1	11,125,894,560	2,727,160,513	275,520,000	179,145,080	4,364,994,612	18,672,714,765
C-F-100-1	10,317,552,457	2,707,345,670	295,920,000	266,253,600	4,364,994,612	17,952,066,339
C-F-400-1	19,351,655,687	2,802,312,099	465,360,000	316,602,205	4,364,994,612	27,300,924,604
C-F-1000-1	26,644,173,031	3,654,940,201	622,560,000	343,940,192	4,364,994,612	35,630,608,036
C-RL-100-1	8,671,820,687	2,334,894,790	112,800,000	636,446,508	4,364,994,612	16,120,956,597
C-RL-400-1	14,988,769,036	2,098,139,483	204,240,000	1,246,495,282	4,364,994,612	22,902,638,413
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	10,295,953,284	10,295,953,284
NS-100	0	0	0	1,504,805,790	10,295,953,284	11,800,759,074
NS-400	0	0	0	2,374,671,167	10,295,953,284	12,670,624,451
NS-1000	0	0	0	3,462,593,524	10,295,953,284	13,758,546,808
G-100-1	8,333,680,532	2,205,061,596	266,520,000	0	10,295,953,284	21,101,215,412
G-100-2	8,387,245,367	2,136,572,912	211,560,000	0	10,295,953,284	21,031,331,562
G-400-3	8,118,918,760	2,118,830,878	297,960,000	0	10,295,953,284	20,831,662,921
G-1000-3	8,465,662,575	2,213,201,037	298,200,000	0	10,295,953,284	21,273,016,896
RL-100-1	1,612,334,989	668,122,326	10,560,000	0	10,295,953,284	12,586,970,598
RL-400-1	2,292,524,528	843,956,072	11,400,000	0	10,295,953,284	13,443,833,884
RL-1000-1	2,633,222,019	931,596,787	11,880,000	0	10,295,953,284	13,872,652,090
C-G-100-1	8,333,680,532	2,205,061,596	266,520,000	1,343,742,214	10,295,953,284	22,444,957,627
C-G-100-2	8,387,245,367	2,136,572,912	211,560,000	1,887,723,808	10,295,953,284	22,919,055,370
C-G-400-3	8,118,918,760	2,118,830,878	297,960,000	2,552,818,056	10,295,953,284	23,384,480,977
C-G-1000-3	8,465,662,575	2,213,201,037	298,200,000	1,029,220,294	10,295,953,284	22,302,237,190
C-RL-100-1	1,612,334,989	668,122,326	10,560,000	1,030,010,452	10,295,953,284	13,616,981,050
C-RL-400-1	2,292,524,528	843,956,072	11,400,000	1,531,934,723	10,295,953,284	14,975,768,607
C-RL-1000-1	2,633,222,019	931,596,787	11,880,000	2,530,878,036	10,295,953,284	16,403,530,126

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	19,625,739,799	19,625,739,799
NS-100	0	0	0	5,480,711,485	19,625,739,799	25,106,451,285
NS-400	0	0	0	7,369,688,257	19,625,739,799	26,995,428,056
NS-1000	0	0	0	8,373,166,632	19,625,739,799	27,998,906,431
M-100-1	16,434,372,132	2,787,977,661	585,720,000	0	19,625,739,799	39,433,809,593
M-100-2 (was R-100-1)	14,764,456,589	2,004,661,651	504,120,000	0	19,625,739,799	36,898,978,039
G-400-2	19,303,922,225	2,657,465,063	631,320,000	0	19,625,739,799	42,218,447,087
G-1000-2	21,035,285,669	2,887,127,039	797,040,000	0	19,625,739,799	44,345,192,507
C-M-100-1	16,434,372,132	2,787,977,661	585,720,000	436,881,756	19,625,739,799	39,870,691,349
C-M-100-2	14,764,456,589	2,004,661,651	504,120,000	483,576,961	19,625,739,799	37,382,555,001
C-G-400-2	19,303,922,225	2,657,465,063	631,320,000	1,282,255,156	19,625,739,799	43,500,702,243
C-G-1000-2	21,035,285,669	2,887,127,039	797,040,000	1,919,416,621	19,625,739,799	46,264,609,128
[illegible]						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	4,408,644,558	4,408,644,558
NS-100	0	0	0	1,893,104,451	4,408,644,558	6,301,749,010
NS-400	0	0	0	3,276,057,346	4,408,644,558	7,684,701,904
NS-1000	0	0	0	4,434,303,424	4,408,644,558	8,842,947,983
G-100-1	11,125,894,560	2,776,494,028	275,520,000	0	4,408,644,558	18,586,553,146
F-100-1	10,317,552,457	2,781,487,033	295,920,000	0	4,408,644,558	17,803,604,048
F-400-1	19,351,655,687	3,144,298,977	465,360,000	0	4,408,644,558	27,369,959,222
F-1000-1	26,644,173,031	3,682,488,092	622,560,000	0	4,408,644,558	35,357,865,681
RL-100-1	8,671,820,687	2,401,488,601	112,800,000	0	4,408,644,558	15,594,753,846
RL-400-1	14,988,769,036	2,154,845,955	204,240,000	0	4,408,644,558	21,756,499,550
C-G-100-1	11,125,894,560	2,776,494,028	275,520,000	179,145,080	4,408,644,558	18,765,698,226
C-F-100-1	10,317,552,457	2,781,487,033	295,920,000	266,253,600	4,408,644,558	18,069,857,648
C-F-400-1	19,351,655,687	3,144,298,977	465,360,000	316,602,205	4,408,644,558	27,686,561,427
C-F-1000-1	26,644,173,031	3,682,488,092	622,560,000	343,940,192	4,408,644,558	35,701,805,873
C-RL-100-1	8,671,820,687	2,401,488,601	112,800,000	636,446,508	4,408,644,558	16,231,200,354
C-RL-400-1	14,988,769,036	2,154,845,955	204,240,000	1,246,495,282	4,408,644,558	23,002,994,831

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1190

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R1 (Coastal)	0	0	0	0	10,501,872,350	10,501,872,350
NS-100	0	0	0	1,504,805,790	10,501,872,350	12,006,678,139
NS-400	0	0	0	2,374,671,167	10,501,872,350	12,876,543,516
NS-1000	0	0	0	3,462,593,524	10,501,872,350	13,964,465,874
G-100-1	8,333,680,532	2,780,783,620	266,520,000	0	10,501,872,350	21,882,856,502
G-100-2	8,387,245,367	2,708,490,008	211,560,000	0	10,501,872,350	21,809,167,725
G-400-3	8,118,918,760	2,689,762,306	297,960,000	0	10,501,872,350	21,608,513,415
G-1000-3	8,465,662,575	2,789,375,252	298,200,000	0	10,501,872,350	22,055,110,177
RL-100-1	1,612,334,989	705,240,233	10,560,000	0	10,501,872,350	12,830,007,571
RL-400-1	2,292,524,528	890,842,521	11,400,000	0	10,501,872,350	13,696,639,399
RL-1000-1	2,633,222,019	983,352,164	11,880,000	0	10,501,872,350	14,130,326,533
C-G-100-1	8,333,680,532	2,780,783,620	266,520,000	1,343,742,214	10,501,872,350	23,226,598,716
C-G-100-2	8,387,245,367	2,708,490,008	211,560,000	1,887,723,808	10,501,872,350	23,696,891,533
C-G-400-3	8,118,918,760	2,689,762,306	297,960,000	2,552,818,056	10,501,872,350	24,161,331,471
C-G-1000-3	8,465,662,575	2,789,375,252	298,200,000	1,029,220,294	10,501,872,350	23,084,330,471
C-RL-100-1	1,612,334,989	705,240,233	10,560,000	1,030,010,452	10,501,872,350	13,860,018,023
C-RL-400-1	2,292,524,528	890,842,521	11,400,000	1,531,934,723	10,501,872,350	15,228,574,121
C-RL-1000-1	2,633,222,019	983,352,164	11,880,000	2,530,878,036	10,501,872,350	16,661,204,569
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	36,217,385,412	36,217,385,412
NS-100	0	0	0	5,433,814,133	36,217,385,412	41,651,199,544
NS-400	0	0	0	19,848,320,132	36,217,385,412	56,065,705,544
NS-1000	0	0	0	32,373,609,268	36,217,385,412	68,590,994,680
LP-a-100-1	6,215,465,504	1,643,263,848	141,120,000	0	36,217,385,412	44,217,234,764
C-LP-a-100-1	6,215,465,504	1,643,263,848	141,120,000	3,258,068,329	36,217,385,412	47,475,303,093
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

1192

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	12,861,436,326	10,058,418,401	22,919,854,728
WBI-100-1	669,694,977	120,545,096	0	0	10,058,418,401	10,848,658,474
C-G-100-1	7,296,901,780	1,248,870,882	139,578,140	2,570,902,566	10,058,418,401	21,314,671,769
C-WBI-100-1	669,694,977	120,545,096	0	3,551,474,236	10,058,418,401	14,400,132,710
C-R-100-2	2,536,353,234	628,416,247	101,448,000	2,836,855,167	10,058,418,401	16,161,491,049
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

1193

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	36,941,733,120	36,941,733,120
NS-100	0	0	0	5,433,814,133	36,941,733,120	42,375,547,253
NS-400	0	0	0	19,848,320,132	36,941,733,120	56,790,053,252
NS-1000	0	0	0	32,373,609,268	36,941,733,120	69,315,342,388
LP-a-100-1	6,215,465,504	1,908,059,432	141,120,000	0	36,941,733,120	45,206,378,056
C-LP-a-100-1	6,215,465,504	1,908,059,432	141,120,000	3,258,068,329	36,941,733,120	48,464,446,386
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

1194

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	12,861,436,326	10,159,002,585	23,020,438,912
WBI-100-1	669,694,977	180,817,644	0	0	10,159,002,585	11,009,515,206
C-G-100-1	7,296,901,780	1,599,373,068	139,578,140	2,570,902,566	10,159,002,585	21,765,758,138
C-WBI-100-1	669,694,977	180,817,644	0	3,551,474,236	10,159,002,585	14,560,989,442
C-R-100-2	2,536,353,234	703,457,449	101,448,000	2,836,855,167	10,159,002,585	16,337,116,435
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	36,217,385,412	36,217,385,412
NS-100	0	0	0	5,110,481,684	36,217,385,412	41,327,867,096
NS-400	0	0	0	23,291,537,130	36,217,385,412	59,508,922,542
NS-1000	0	0	0	33,888,838,665	36,217,385,412	70,106,224,077
LP-a-100-1	6,215,465,504	1,643,263,848	141,120,000	0	36,217,385,412	44,217,234,764
C-LP-a-100-1	6,215,465,504	1,643,263,848	141,120,000	2,771,250,405	36,217,385,412	46,988,485,168
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

1196

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	23,082,585,313	10,058,418,401	33,141,003,714
WBI-100-1	669,694,977	120,545,096	0	0	10,058,418,401	10,848,658,474
C-G-100-1	7,296,901,780	1,248,870,882	139,578,140	2,633,958,371	10,058,418,401	21,377,727,574
C-WBI-100-1	669,694,977	120,545,096	0	3,661,046,837	10,058,418,401	14,509,705,311
C-R-100-2	2,536,353,234	628,416,247	101,448,000	2,864,390,907	10,058,418,401	16,189,026,789

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 1 Alternatives
(First Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	36,941,733,120	36,941,733,120
NS-100	0	0	0	5,110,481,684	36,941,733,120	42,052,214,804
NS-400	0	0	0	23,291,537,130	36,941,733,120	60,233,270,250
NS-1000	0	0	0	33,888,838,665	36,941,733,120	70,830,571,785
LP-a-100-1	6,215,465,504	1,908,059,432	141,120,000	0	36,941,733,120	45,206,378,056
C-LP-a-100-1	6,215,465,504	1,908,059,432	141,120,000	2,771,250,405	36,941,733,120	47,977,628,461
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 2 Alternatives
(First Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	23,082,585,313	10,159,002,585	33,241,587,898
WBI-100-1	669,694,977	180,817,644	0	0	10,159,002,585	11,009,515,206
C-G-100-1	7,296,901,780	1,599,373,068	139,578,140	2,633,958,371	10,159,002,585	21,828,813,943
C-WBI-100-1	669,694,977	180,817,644	0	3,661,046,837	10,159,002,585	14,670,562,043
C-R-100-2	2,536,353,234	703,457,449	101,448,000	2,864,390,907	10,159,002,585	16,364,652,175
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

1199

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	6,590,244,523	0	6,590,244,523
NS-400	0	0	0	8,969,682,625	0	8,969,682,625
NS-1000	0	0	0	9,831,481,551	0	9,831,481,551
C-M-100-1	19,721,246,559	2,084,100,800	702,864,000	531,186,975	0	23,039,398,334
C-M-100-2	17,717,347,907	2,123,877,704	604,944,000	585,459,701	0	21,031,629,311
Color Code						
No Action						
Nonstructural						
Comprehensive (Struct + NS)						

1200

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives carried into MCDA)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	3,823,326,439	0	3,823,326,439
NS-1000	0	0	0	4,769,125,188	0	4,769,125,188
C-G-100-1	13,351,073,472	3,272,592,616	330,624,000	205,309,322	0	17,159,599,410
C-F-100-1	12,381,062,948	3,248,814,804	355,104,000	307,574,576	0	16,292,556,328
C-RL-100-1	10,406,184,824	2,801,873,748	135,360,000	726,255,197	0	14,069,673,770

Color Code
No Action
Nonstructural
Comprehensive (Struct + NS)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	1,803,789,269	0	1,803,789,269
NS-400	0	0	0	2,900,006,876	0	2,900,006,876
NS-1000	0	0	0	4,024,086,678	0	4,024,086,678
C-RL-100-1	1,934,801,987	801,746,791	12,672,000	1,604,812,733	0	4,354,033,510
C-RL-400-1	2,751,029,434	1,012,747,287	13,680,000	1,400,013,598	0	5,177,470,319
C-RL-1000-1	3,159,866,423	1,117,916,144	14,256,000	2,939,715,904	0	7,231,754,471
Color Code						
No Action						
Nonstructural						
Comprehensive (Struct + NS)						

1202

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	6,590,244,523	0	6,590,244,523
NS-400	0	0	0	8,969,682,625	0	8,969,682,625
NS-1000	0	0	0	9,831,481,551	0	9,831,481,551
C-M-100-1	19,721,246,559	3,345,573,193	702,864,000	531,186,975	0	24,300,870,727
C-M-100-2	17,717,347,907	2,405,593,981	604,944,000	585,459,701	0	21,313,345,588
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1203

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	3,823,326,439	0	3,823,326,439
NS-1000	0	0	0	4,769,125,188	0	4,769,125,188
C-G-100-1	13,351,073,472	3,331,792,833	330,624,000	205,309,322	0	17,218,799,628
C-F-100-1	12,381,062,948	3,337,784,440	355,104,000	307,574,576	0	16,381,525,964
C-RL-100-1	10,406,184,824	2,881,786,322	135,360,000	726,255,197	0	14,149,586,343

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

1204

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	1,803,789,269	0	1,803,789,269
NS-400	0	0	0	2,900,006,876	0	2,900,006,876
NS-1000	0	0	0	4,024,086,678	0	4,024,086,678
C-RL-100-1	1,934,801,987	846,288,279	12,672,000	1,604,812,733	0	4,398,574,999
C-RL-400-1	2,751,029,434	1,069,011,025	13,680,000	1,400,013,598	0	5,233,734,057
C-RL-1000-1	3,159,866,423	1,180,022,597	14,256,000	2,939,715,904	0	7,293,860,924
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1205

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	6,576,853,782	0	6,576,853,782
NS-400	0	0	0	8,843,625,908	0	8,843,625,908
NS-1000	0	0	0	10,047,799,958	0	10,047,799,958
C-M-100-1	19,721,246,559	2,084,100,800	702,864,000	524,258,108	0	23,032,469,467
C-M-100-2	17,717,347,907	2,123,877,704	604,944,000	580,292,354	0	21,026,461,965

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

1206

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	3,931,268,816	0	3,931,268,816
NS-1000	0	0	0	5,321,164,109	0	5,321,164,109
C-G-100-1	13,351,073,472	3,272,592,616	330,624,000	214,974,095	0	17,169,264,183
C-F-100-1	12,381,062,948	3,248,814,804	355,104,000	319,504,320	0	16,304,486,072
C-RL-100-1	10,406,184,824	2,801,873,748	135,360,000	763,735,810	0	14,107,154,383
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1207

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	1,805,766,948	0	1,805,766,948
NS-400	0	0	0	2,849,605,400	0	2,849,605,400
NS-1000	0	0	0	4,155,112,229	0	4,155,112,229
C-RL-100-1	1,934,801,967	801,746,791	12,672,000	1,612,490,657	0	4,361,711,435
C-RL-400-1	2,751,029,434	1,012,747,287	13,680,000	2,265,268,570	0	6,042,725,290
C-RL-1000-1	3,159,866,423	1,117,916,144	14,256,000	3,063,381,667	0	7,355,420,234

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

1208

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3a Alternatives
(First Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	6,576,853,782	0	6,576,853,782
NS-400	0	0	0	8,843,625,908	0	8,843,625,908
NS-1000	0	0	0	10,047,799,958	0	10,047,799,958
C-M-100-1	19,721,246,559	3,345,573,193	702,864,000	524,258,108	0	24,293,941,860
C-M-100-2	17,717,347,907	2,405,593,981	604,944,000	580,292,354	0	21,308,178,242
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						

1209

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 3b Alternatives
(First Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	3,931,268,816	0	3,931,268,816
NS-1000	0	0	0	5,321,164,109	0	5,321,164,109
C-G-100-1	13,351,073,472	3,331,792,833	330,624,000	214,974,095	0	17,228,464,401
C-F-100-1	12,381,062,948	3,337,784,440	355,104,000	319,504,320	0	16,393,455,708
C-RL-100-1	10,406,184,824	2,881,786,322	135,360,000	763,735,810	0	14,187,066,956

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

Louisiana Coastal Protection and Restoration (LACPR)

First Cost - Planning Unit 4 Alternatives
(First Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	1,805,766,948	0	1,805,766,948
NS-400	0	0	0	2,849,605,400	0	2,849,605,400
NS-1000	0	0	0	4,155,112,229	0	4,155,112,229
C-RL-100-1	1,934,801,987	846,288,279	12,672,000	1,612,490,657	0	4,406,252,923
C-RL-400-1	2,751,029,434	1,069,011,025	13,680,000	2,265,268,570	0	6,098,989,028
C-RL-1000-1	3,159,866,423	1,180,022,597	14,256,000	3,063,381,667	0	7,417,526,687

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

1211

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 1 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	64,072,239,679	64,072,239,679
NS-100	0	0	0	6,730,544,358	64,072,239,679	70,802,784,036
NS-400	0	0	0	24,584,940,856	64,072,239,679	88,657,180,535
NS-1000	0	0	0	40,099,276,103	64,072,239,679	104,171,515,781
LP-a-100-1	8,504,796,625	3,209,674,338	174,797,002	0	64,072,239,679	75,961,507,644
C-LP-a-100-1	8,504,796,625	3,209,674,338	174,797,002	4,035,576,646	64,072,239,679	79,997,084,290

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1212

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 2 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	15,930,700,901	14,311,350,500	30,242,051,401
WBI-100-1	829,511,580	235,452,451	0	0	14,311,350,500	15,376,314,532
C-G-100-1	10,468,365,066	2,439,333,665	172,887,191	3,184,425,035	14,311,350,500	30,576,361,457
C-WBI-100-1	829,511,580	235,452,451	0	4,399,001,198	14,311,350,500	19,775,315,730
C-R-100-2	3,195,753,014	1,227,442,267	125,657,641	3,513,844,801	14,311,350,500	22,374,048,223

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 1 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	65,353,684,472	65,353,684,472
NS-100	0	0	0	6,730,544,358	65,353,684,472	72,084,228,830
NS-400	0	0	0	24,584,940,856	65,353,684,472	89,938,625,328
NS-1000	0	0	0	40,099,276,103	65,353,684,472	105,452,960,575
LP-a-100-1	8,504,796,625	3,726,881,354	174,797,002	0	65,353,684,472	77,760,159,454
C-LP-a-100-1	8,504,796,625	3,726,881,354	174,797,002	4,035,576,646	65,353,684,472	81,795,736,099
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 2 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	15,930,700,901	14,454,464,005	30,385,164,906
WBI-100-1	829,511,580	353,178,677	0	0	14,454,464,005	15,637,154,262
C-G-100-1	10,468,365,066	3,123,945,494	172,887,191	3,184,425,035	14,454,464,005	31,404,086,791
C-WBI-100-1	829,511,580	353,178,677	0	4,399,001,198	14,454,464,005	20,036,155,460
C-R-100-2	3,195,753,014	1,374,015,089	125,657,641	3,513,844,801	14,454,464,005	22,663,734,550

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 1 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	64,072,239,679	64,072,239,679
NS-100	0	0	0	6,330,051,566	64,072,239,679	70,402,291,245
NS-400	0	0	0	28,849,850,213	64,072,239,679	92,922,089,892
NS-1000	0	0	0	41,976,101,187	64,072,239,679	106,048,340,866
LP-a-100-1	8,504,796,625	3,209,674,338	174,797,002	0	64,072,239,679	75,961,507,644
C-LP-a-100-1	8,504,796,625	3,209,674,338	174,797,002	3,432,584,060	64,072,239,679	79,394,091,704

Color Code
No Action
Coastal Only
Nonstructural / Coastal
Structural / Coastal
Comprehensive (Struct + NS + Coastal)

1216

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 2 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	28,591,033,949	14,311,350,500	42,902,384,450
WBI-100-1	829,511,580	235,452,451	0	0	14,311,350,500	15,376,314,532
C-G-100-1	10,468,365,066	2,439,333,665	172,887,191	3,262,528,533	14,311,350,500	30,654,464,955
C-WBI-100-1	829,511,580	235,452,451	0	4,534,722,302	14,311,350,500	19,911,036,833
C-R-100-2	3,195,753,014	1,227,442,267	125,657,641	3,547,951,694	14,311,350,500	22,408,155,116
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 1 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 1 (PU1-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
R2 (Coastal)	0	0	0	0	65,353,684,472	65,353,684,472
NS-100	0	0	0	6,330,051,566	65,353,684,472	71,683,736,039
NS-400	0	0	0	28,849,850,213	65,353,684,472	94,203,534,686
NS-1000	0	0	0	41,976,101,187	65,353,684,472	107,329,785,660
LP-a-100-1	8,504,796,625	3,726,881,354	174,797,002	0	65,353,684,472	77,760,159,454
C-LP-a-100-1	8,504,796,625	3,726,881,354	174,797,002	3,432,584,060	65,353,684,472	81,192,743,513
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 2 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 2 (PU2-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	NS Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	28,591,033,949	14,454,464,005	43,045,497,955
WBI-100-1	829,511,580	353,178,677	0	0	14,454,464,005	15,637,154,262
C-G-100-1	10,468,365,066	3,123,945,494	172,887,191	3,262,528,533	14,454,464,005	31,482,190,289
C-WBI-100-1	829,511,580	353,178,677	0	4,534,722,302	14,454,464,005	20,171,876,564
C-R-100-2	3,195,753,014	1,374,015,089	125,657,641	3,547,951,694	14,454,464,005	22,697,841,443
Color Code						
No Action						
Coastal Only						
Nonstructural / Coastal						
Structural / Coastal						
Comprehensive (Struct + NS + Coastal)						

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3a Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	8,162,946,323	0	8,162,946,323
NS-400	0	0	0	11,110,215,643	0	11,110,215,643
NS-1000	0	0	0	12,177,675,030	0	12,177,675,030
C-M-100-1	29,265,349,174	4,070,730,861	870,596,088	657,949,906	0	34,864,626,030
C-M-100-2	26,652,831,028	4,148,424,353	749,308,373	725,174,323	0	32,275,738,077
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1220

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3b Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	4,735,728,452	0	4,735,728,452
NS-1000	0	0	0	5,907,233,454	0	5,907,233,454
C-G-100-1	19,942,834,072	6,392,130,244	409,524,405	254,304,521	0	26,998,793,242
C-F-100-1	18,243,772,858	6,345,686,678	439,846,334	380,974,446	0	25,410,280,316
C-RL-100-1	15,233,606,634	5,472,707,431	167,662,430	899,569,382	0	21,773,545,878
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1221

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 4 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 1 (Low RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	2,234,247,141	0	2,234,247,141
NS-400	0	0	0	3,592,067,090	0	3,592,067,090
NS-1000	0	0	0	4,984,398,292	0	4,984,398,292
C-RL-100-1	2,499,996,449	1,565,996,906	15,696,057	1,987,786,667	0	6,069,476,080
C-RL-400-1	3,704,826,507	1,978,129,673	16,944,607	1,734,114,086	0	7,434,014,874
C-RL-1000-1	4,308,896,694	2,183,548,775	17,658,064	3,641,252,315	0	10,151,355,849
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						
Structural / Coastal						

1222

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3a Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	8,162,946,323	0	8,162,946,323
NS-400	0	0	0	11,110,215,643	0	11,110,215,643
NS-1000	0	0	0	12,177,675,030	0	12,177,675,030
C-M-100-1	29,265,349,174	6,534,678,190	870,596,088	657,949,906	0	37,328,573,359
C-M-100-2	26,652,831,028	4,698,681,396	749,308,373	725,174,323	0	32,825,995,120
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1223

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3b Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	4,735,728,452	0	4,735,728,452
NS-1000	0	0	0	5,907,233,454	0	5,907,233,454
C-G-100-1	19,942,834,072	6,507,761,960	409,524,405	254,304,521	0	27,114,424,958
C-F-100-1	18,243,772,858	6,519,464,953	439,846,334	380,974,446	0	25,584,058,591
C-RL-100-1	15,233,606,634	5,628,795,168	167,662,430	899,569,382	0	21,929,633,615
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1224

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 4 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 2 (High RSLR and Dispersed Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	2,234,247,141	0	2,234,247,141
NS-400	0	0	0	3,592,067,090	0	3,592,067,090
NS-1000	0	0	0	4,984,398,292	0	4,984,398,292
C-RL-100-1	2,499,996,449	1,652,996,734	15,696,057	1,987,785,667	0	6,156,475,908
C-RL-400-1	3,704,826,507	2,088,025,766	16,944,607	1,734,114,086	0	7,543,910,966
C-RL-1000-1	4,308,896,694	2,304,857,040	17,658,064	3,641,252,315	0	10,272,664,114
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

Fully Funded Cost - Planning Unit 3a Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	8,146,360,003	0	8,146,360,003
NS-400	0	0	0	10,954,076,640	0	10,954,076,640
NS-1000	0	0	0	12,445,615,854	0	12,445,615,854
C-M-100-1	29,265,349,174	4,070,730,861	870,596,088	649,367,527	0	34,856,043,651
C-M-100-2	26,652,831,028	4,148,424,353	749,308,373	718,773,836	0	32,269,337,590
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

Fully Funded Cost - Planning Unit 3b Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	4,869,430,293	0	4,869,430,293
NS-1000	0	0	0	6,591,011,433	0	6,591,011,433
C-G-100-1	19,942,834,072	6,392,130,244	409,524,405	266,275,704	0	27,010,764,425
C-F-100-1	18,243,772,858	6,345,686,678	439,846,334	395,751,115	0	25,425,056,985
C-RL-100-1	15,233,606,634	5,472,707,431	167,662,430	945,994,401	0	21,819,970,897
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

Fully Funded Cost - Planning Unit 4 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 3 (Low RSLR and Compact Population)

Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	2,236,696,774	0	2,236,696,774
NS-400	0	0	0	3,529,637,761	0	3,529,637,761
NS-1000	0	0	0	5,146,691,898	0	5,146,691,898
C-RL-100-1	2,499,996,449	1,565,996,906	15,696,057	1,997,296,858	0	6,078,986,270
C-RL-400-1	3,704,826,507	1,978,129,673	16,944,607	2,805,854,271	0	8,505,755,059
C-RL-1000-1	4,308,896,694	2,183,548,775	17,658,064	3,794,429,786	0	10,304,533,319
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

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Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3a Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3a (PU3a-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	8,146,360,003	0	8,146,360,003
NS-400	0	0	0	10,954,076,640	0	10,954,076,640
NS-1000	0	0	0	12,445,615,854	0	12,445,615,854
C-M-100-1	29,265,349,174	6,534,678,190	870,596,088	649,367,527	0	37,319,990,980
C-M-100-2	26,652,831,028	4,698,681,396	749,308,373	718,773,836	0	32,819,594,633
[redacted]						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1229

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 3b Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

Planning Unit 3b (PU3b-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-400	0	0	0	4,869,430,293	0	4,869,430,293
NS-1000	0	0	0	6,591,011,433	0	6,591,011,433
C-G-100-1	19,942,834,072	6,507,761,960	409,524,405	266,275,704	0	27,126,396,141
C-F-100-1	18,243,772,858	6,519,464,953	439,846,334	395,751,115	0	25,598,835,260
C-RL-100-1	15,233,606,634	5,628,795,168	167,662,430	945,994,401	0	21,976,058,634

Color Code
No Action
Nonstructural / Coastal
Comprehensive (Struct + NS)

1230

Louisiana Coastal Protection and Restoration (LACPR)

Fully Funded Cost - Planning Unit 4 Alternatives
(Fully Funded Cost of all alternatives in final array)
Scenario 4 (High RSLR and Compact Population)

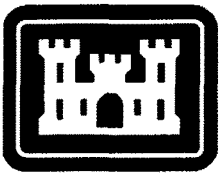
Planning Unit 4 (PU4-)	Structural Cost	Future Lift Cost (Structural Measures)	Mitigation Cost	Nonstructural Cost	Coastal Features Cost	Total Cost
No Action	0	0	0	0	0	0
NS-100	0	0	0	2,236,696,774	0	2,236,696,774
NS-400	0	0	0	3,529,637,761	0	3,529,637,761
NS-1000	0	0	0	5,146,691,898	0	5,146,691,898
C-RL-100-1	2,499,996,449	1,652,996,734	15,696,057	1,997,296,858	0	6,165,986,098
C-RL-400-1	3,704,826,507	2,088,025,766	16,944,607	2,805,854,271	0	8,615,651,152
C-RL-1000-1	4,308,896,694	2,304,857,040	17,658,064	3,794,429,786	0	10,425,841,584
Color Code						
No Action						
Nonstructural / Coastal						
Comprehensive (Struct + NS)						

1231

LOUISIANA COASTAL PROTECTION AND RESTORATION
FINAL TECHNICAL REPORT

HYDRAULICS AND HYDROLOGY APPENDIX

June 2009



U. S. Army Corps of Engineers
New Orleans District
Mississippi Valley Division

(1233)

Hydraulics and Hydrology Appendix
Volume I - Background and Methodology

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1 INTRODUCTION

1.1 Purpose

The Louisiana Coastal Protection and Restoration (LACPR) Technical Report has been developed by the United States Army Corps of Engineers (USACE) in response to Public Laws 109-103 and 109-148. Under these laws, Congress and the President directed the Secretary of the Army, acting through the Chief of Engineers, to:

- Conduct a comprehensive hurricane protection analysis and design in close coordination with the State of Louisiana and its appropriate agencies;
- Develop and present a full range of flood control, coastal restoration, and hurricane protection measures exclusive of normal policy considerations for South Louisiana;
- Consider providing protection for a storm surge equivalent to a Category 5 hurricane; and
- Submit preliminary and final technical reports.

The purpose of this appendix is to support the hydraulics and hydrology evaluation for LACPR, which is discussed in the main Technical Report.

1.2 Planning area and planning units

The LACPR planning area stretches across Louisiana's coast from the Pearl River, on the Mississippi border, to the Sabine River, on the Texas border. The planning area is divided into five planning units (see **Figure 1.1**).

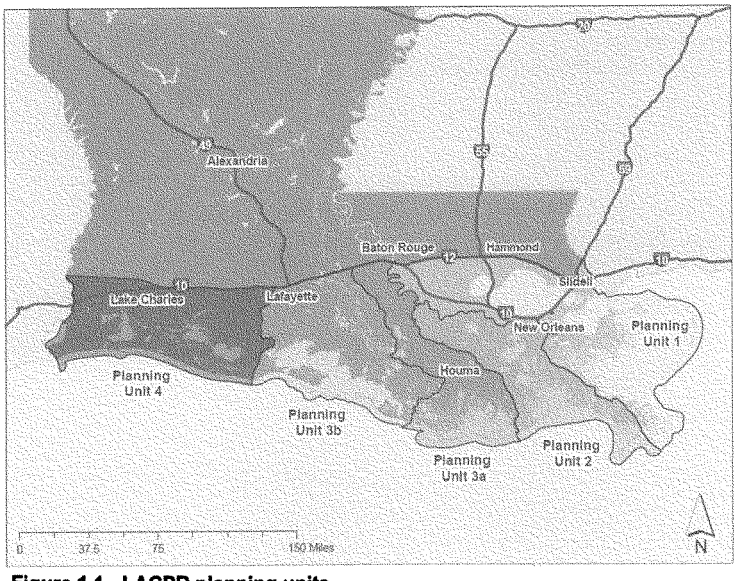


Figure 1.1 - LACPR planning units

1.3 Hydraulic evaluation

Volume I of this appendix describes the methodology for the hydraulic evaluation of the alternatives within the framework of the LACPR Technical Report. This hydraulic analysis has been visualized in **Figure 1.2**.

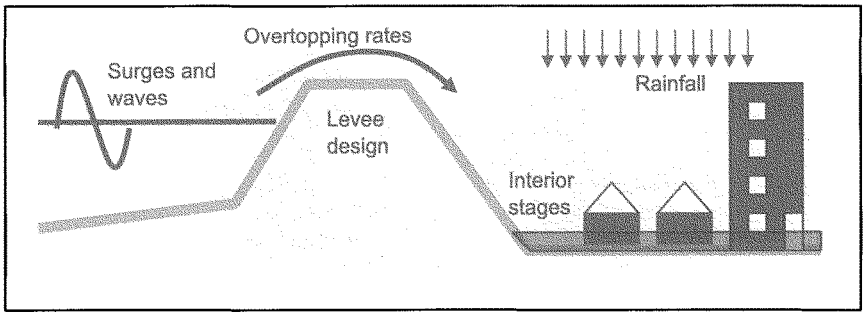


Figure 1.2 - Schematic overview of the step-wise approach in the hydraulic analysis in the framework of LACPR

In the framework of LACPR the hydraulic analysis plays a key role in the evaluation of the alternatives in the various planning units. Each levee alternative affects the surge and the waves during a storm in a different way. The differences in storm surge and wave characteristics result in varying overtopping volumes and stage frequency curves. The stage frequency curves are an important input for the economic analysis to estimate the damage in the planning subunits, and the levee heights need to be known for the cost estimates. This report describes the methodology of the hydraulic analysis that has been followed to determine the exterior and interior stages, and the levee heights. The results of this hydraulic analysis for the various planning units are described in Volume II of this appendix. The use of this data within the economic analysis and the cost estimates are described in separate appendices.

The hydraulic analysis of each alternative in LACPR consisted of the following consecutive steps:
1. Numerical computations of surge levels and wave characteristics using ADCIRC, WAM and STWAVE;
2. Frequency analysis using the JPM-OS method and the determination of exterior stage frequency;
3. Determination of the levee heights and overtopping volumes;
4. Determination of the interior stages including rainfall;

To provide a range of alternatives for evaluation and to enable the economic evaluation it was decided to evaluate each levee alignment alternative for different risk reduction levels and event frequencies. A levee design was made for three different levels of risk reduction (100-year, 400-year, 1000-design year). Given the level of risk reduction, the overtopping volumes were computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year and 2000-year event). For all alternatives, the 10-year rainfall was added to the

overtopping volume to establish the interior stage frequency curve and pumping was taken into account. The four steps in the hydraulic analysis are discussed in detail below.

Step 1: Surge levels and wave characteristics

The numerical computations for the surge levels and the wave characteristics were carried out with the numerical models ADCIRC, for surge levels, and WAM/STWAVE, for the wave characteristics. These models are state-of-the art models and have already been applied extensively during the IPET and 100-year design for the hurricane risk reduction system around New Orleans. Two basic ADCIRC modeling grids were developed to cover the southern coast of Louisiana. Several wave grids were developed for STWAVE to compute the wave characteristics.

A base set of 56 hurricane conditions have been evaluated with the modeling suite ADCIRC/STWAVE for the 2010 base condition. The modeled storms are different in terms of the hurricane tracks, minimum pressure, and radius amongst other parameters. The 2010 base condition consists of the existing condition with a levee system with a 100-year risk reduction level including the barrier at MRGO. The different levee alignments for the various alternatives (e.g. barrier plan or West Bank alignment along GIWW) have been implemented in the model grids to evaluate the behavior of the surge levels and waves. In addition, computations have been carried out to evaluate the future effects of sea level rise and marsh improvement/degradation. For all of the alternatives, the number of storms that were evaluated was reduced because of time constraints.

Step 2: Frequency analysis

Based on the results from ADCIRC and STWAVE in step 1, a frequency analysis has been carried out to determine the surge levels and wave characteristics for different return periods. The method adopted was the Joint Probability Method with Optimal Sampling (JPM-OS) that takes into account the joint probability of forward speed, size, minimum pressure, angle of approach and geographic distribution of the hurricanes. This method requires a set of 152 storms to establish the frequency curves for surge and waves. Since the various alternatives were only run for 56 or less storms, the results for the remaining storms were established using correlation techniques in order to carry out the frequency analysis with the JPM-OS method.

The frequency analysis has resulted in stage frequencies for the exterior areas, i.e. the areas that are not protected by the levees. Furthermore, this analysis has provided the surge levels and the wave characteristics for different return periods along the levee system as needed for the levee design and overtopping volumes in step 3.

Step 3: Levee design and overtopping volumes

For the levee designs the step-wise procedure that was used for the 100-year design elevations has been followed in a slightly adapted way. In short, this procedure has been applied as follows in LACPR:

- Use the surge level and wave characteristics at the levees for a given level of risk reduction (e.g. 100-year) and assume a simplified levee design for this planning effort, i.e. a levee with a wave berm at the still water level and a constant slope near the crest of the levee of 1:4.

- Determine the overtopping rate using empirical formulations. A Monte Carlo Simulation was adopted to compute the uncertainty in the overtopping rate given the uncertainties in the hydraulic boundary conditions and the empirical coefficients in the overtopping formulations.

- Establish the levee height in such a way that the overtopping rate is less than 0.1 cft/s per ft with 90% confidence. The levee heights for the various alternatives have been used as an input for the interior drainage analysis and costs estimates.

The overtopping volumes were computed using the information on the surge level hydrographs from ADCIRC. Based on a statistical analysis, a correlation was established between the duration of the surge and the maximum surge level. This correlation has been applied to compute the overtopping rate during the storm assuming that the wave characteristics are constant around the peak of the storm.

Step 4: Interior stage frequency

The final step was to determine the interior stage frequency for each planning subunit. Each subunit has been schematized as a box model for which a stage-storage curve has been established. This information has been extracted from existing rainfall-runoff models or from LIDAR data for these areas. The interior stage frequency has been based on the sum of the overtopping volume from step 3 together with the 10-year rainfall in the subunit. The effect of pumping has been taken into account if applicable. Where planning subunits join, flow of water has been allowed to occur above define thresholds.

1.4 Outline of Volume I

The outline of this volume follows the structure of the flow chart in **Figure 1.3**, which visualizes in detail the various steps to facilitate the hydraulic evaluation of the various alternatives. To evaluate various alternatives numerical modeling with ADCIRC and STWAVE was carried out to simulate the water levels and wave heights (in yellow). The water levels and the waves are used in the hydraulic analysis to determine the levee heights, the exterior stage frequency curves and the interior stage frequency curves (in blue). Additionally, economic damage assessments, levee construction cost estimates as well as risk and reliability tasks can be performed with the resulting datasets (in green).

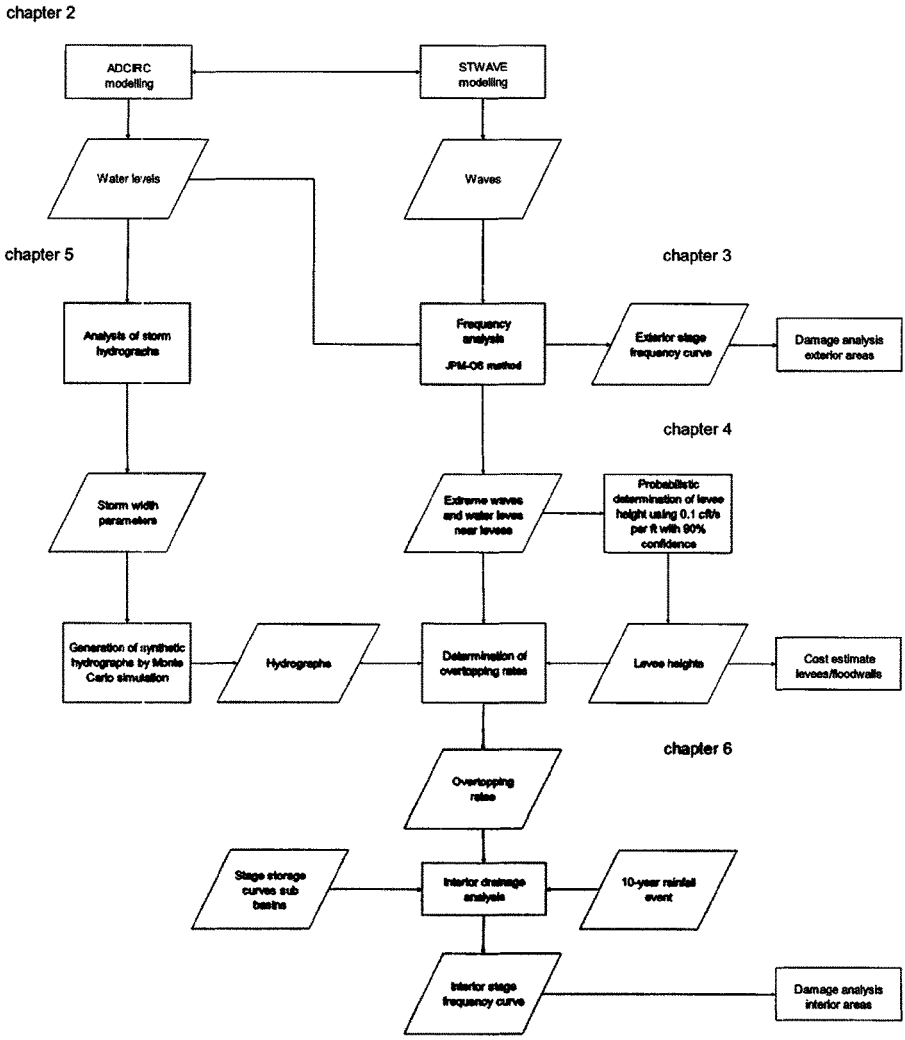


Figure 1.3 - Flow diagram of hydraulic analysis in LACPR framework (in blue)

Chapter 2 briefly describes the numerical modeling with ADCIRC and STWAVE. The background to the processes, the modeled alternatives and summaries of the model output are presented. The focus of Chapter 3 is to describe the frequency analysis undertaken to come up with the surge levels and wave characteristics for different return period events. Chapter 4 deals with the levee design procedure that has been applied within the LACPR framework and chapter 5 discusses the determination of the overtopping volumes. The development of the interior stage frequency curves is described in Chapter 6.

The work as presented in Chapter 3 through 6 of Volume I was undertaken from June to September 2007 as a joint effort between the United States Army Corps of Engineers, New Orleans District and Haskoning Inc[1]. The methods described within Volume I are limited to the hydraulic aspects only and no economic evaluation is provided. The main deliverables of this task were the stage frequency curves and the design heights of the levees. The stage frequency curves will be used for the economic evaluations and the damage studies. Note that the methodologies described within this appendix are developed to enable the relative comparison of various design alternatives. More detailed study will be needed for doing actual design.

Annex A of Volume I describes the development of relative sea level rise projections for LACPR. Annex B of Volume I describes the background and characteristics of the theoretical maximum possible intensity hurricane and some of its implications for the Louisiana coastal area.

Volume II of this appendix presents the results of the hydraulic evaluation of the LACPR alternatives.

[1] Chapter 2 is added to this report to present the complete picture of the hydraulic analysis in the framework of LACPR. The work as presented in Chapter 2 summarizes the result of a combined effort of FEMA and USACE, universities and various consultancy firms.

2 SURGE AND WAVE MODELING[2]

The surge level and wave computations with an atmospheric-hydrodynamic modeling system form the basis of the hydraulic analysis within the framework of LACPR (**Figure 2.1**). The main components and the validation of this modeling process are briefly summarized in Section 2.1 and Section 2.2. For more detailed information, the reader is referred to various earlier studies in which this modeling suite was applied (IPET, 2007; FEMA, 2007). The LACPR effort evaluates several alternative storm surge risk reduction systems using many levee alignments. For each of the main variations, a model grid has been created to model the system and provide results from which levee heights can be determined. These model grids are discussed in Section 2.3. Section 2.4 describes the selection of hurricanes that have been evaluated for the alternatives because it was impossible to run all hurricanes for all alternatives. Finally, the output locations and results of the modeling system at these points that have been used for LACPR are summarized in Section 2.5. These results are input into the frequency analysis (Chapter 3) and the determination of the overtopping volumes (Chapter 5).

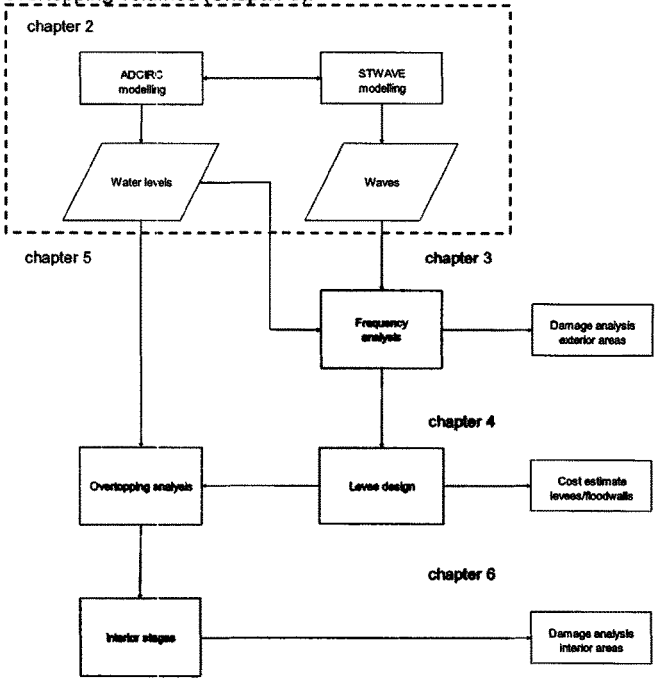


Figure 2.1 - Flow diagram of hydraulic analysis in LACPR framework (Chapter 2)

[2] The work as presented in Chapter 2 summarizes the result of a combined effort of FEMA and USACE, universities and various consultancy firms. Section 2.1 – 2.5 of this Chapter were written by the ADCIRC/STWAVE team (Joannes Westerink, Mary Cialone, Allison Sleith, John Atkinson, Jay Ratcliff).

2.1 Description of the modeled hurricanes

The LACPR authorization states that *".....the Secretary shall consider providing protection for a storm surge equivalent to a Category 5 hurricane..."*. Previously, it was believed that a single parameter, the Saffir-Simpson intensity scale (see **Table 2.1**), dictated the potential surge levels that a storm could generate. Based on this concept, previous design-storm concepts used terms such as a "Category 5 Storm" to denote a particular class of storm.

Table 2.1 - Saffir-Simpson classification

Saffir-Simpson Category	Wind speeds (m/s)	Pressure (mbar)	Historical examples at the Atlantic Ocean
1	33-42	980	Jerry (1989), Danny (1997)
2	43-49	965-979	Diana (1990), Erin (1995)
3	50-58	945-964	Roxanne (1995), Isidore (2002)
4	59-69	920-944	Galveston (1900), Betsy (1965), Iris (2001), Charley (2004)
5	> 70 m/s	< 920	Camille (1969), Katrina (2005), Rita (2005)

Recent analyses have clearly demonstrated that coastal surge levels are significantly affected by storm size as well as storm intensity (Saffir-Simpson category). It is now recognized that a small "Category 5 Storm" will generate a smaller surge than a large "Category 3 Storm" in coastal areas where the offshore slope is very small, such as along much of the Louisiana-Mississippi coastline. Thus, it is important to consider a range of storm sizes in conjunction with a fixed "Category 5" intensity, in order to represent the actual range of conditions that a "Category 5 Storm" can generate. This insight changes the manner in which a storm must be specified for planning and design purposes.

A USACE and FEMA consensus procedure was developed in order to define the relevant storms that affect Southern Louisiana (FEMA, 2007). It was agreed that the Joint Probability Method (JPM) allows for the richest storm set but that many of the storms are either irrelevant or have a very low probability of occurrence due to dependencies in the parameter space. A set of 152 storms were developed for eastern Louisiana by combining the "probable" combinations of central pressure, radius to maximum winds, forward speed, angle of track relative to coastline, and track. Tracks were defined by 5 primary tracks and 4 secondary tracks (see **Figure 2.2**). Central pressure and radius to maximum relationships were also developed that modify the storms as the coastline is approached. A storm matrix was developed based on these parameters and proposed to FEMA and USACE for concurrence. A concurrent set of 152 storms was developed for western Louisiana.

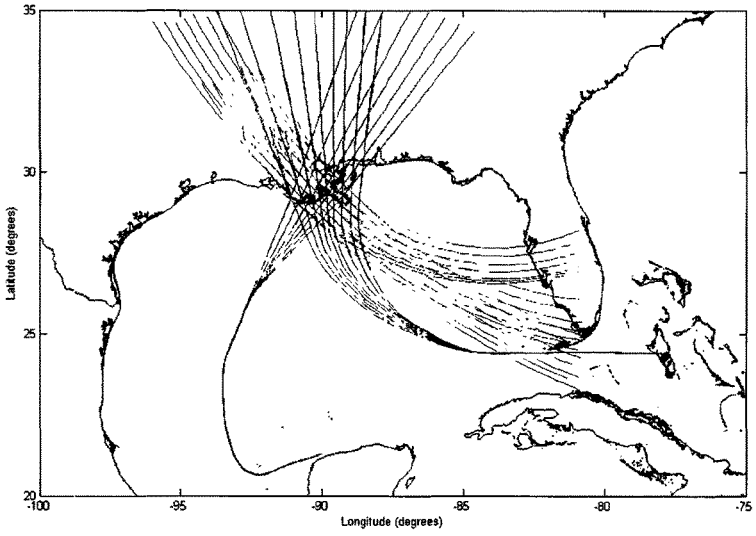


Figure 2.2 - Storm tracks for eastern Louisiana

The storm set of 152 storms contains 50 "Category 3 storms", 52 "Category 4" storms and 50 "Category 5" storms. As discussed above, these subsets of storms are classes of storms that can result in different hydrodynamic behavior depending on the storm size and other factors. The following ranges of storm sizes are considered in the set with 152 storms:

- Category 3: 11 – 35 nautical miles
- Category 4: 8 – 25 nautical miles
- Category 5: 6 – 21 nautical miles

The probability of occurrence of the 152 storms covers a frequency range between approximately 1 in 50 years and 1 in 3,500 years.

In the framework of LACPR, hydraulic events with different return periods have been chosen as a basis for evaluation (and not the Saffir-Simpson Scale). These events are: 100-year event, 400-year event, 1,000-year event and 2,000-year event. The 100-year event has been chosen because that return period serves as a basis for the current design effort for the Greater New Orleans Hurricane Storm Damage and Risk Reduction System. The 400-year event is a proxy for Katrina, because this is the estimated return period for this hurricane (see Resio et al., 2007). The 1,000-year and 2,000-year event are chosen based on practical considerations. The maximum frequency of the storm set is approximately 3,500 year. The 1,000 year and 2,000 year events were considered appropriate choices to have enough coverage in the storm set. Note that all these events cover different "Cat 5 hurricanes" with increasing storm size. Apart from this, an additional

analysis was done regarding the so-called maximum possible intensity hurricane. Annex B describes the background and characteristics of the theoretical maximum possible intensity hurricane and some of its implications for the Louisiana coastal area.

The storm set of 152 storms has been used as a starting point to analyze the surge levels and the waves at the Louisiana coastline. The setup and interaction between the various atmospheric and hydrodynamic models to compute the actual surge levels and waves is the topic of the next section.

2.2 Atmospheric-hydrodynamic modeling system

In purse of a common technical framework of al Federal Agencies involved in assessing hurricane related threats to coastal communities, an atmospheric-hydrodynamic modeling system has been implemented. The goal of this hydrodynamic model development has been to implement a simulation capability that represents the basic physics of the system as it is observed and that does not require ad hoc tuning. Therefore the hydrodynamic models should define the physical system as it exists and should consider wind, atmospheric pressure, short period wind waves, tides, and riverine flows in a comprehensive way. In order to achieve the required accuracy, a sequence of state of the art, well verified and validated wind, short period wind wave and coastal circulation models were coupled together as an atmospheric-hydrodynamic modeling system and applied to Southern Louisiana and Mississippi.

The modeling suite consists of four major components:

- Wind and pressure model (PBL)
- Surge model (ADCIRC)
- Deep water wave model (WAM)
- Shallow water wave model (STWAVE)

The coherence and interaction between these models is visualized in **Figure 2.3**.

The first component in the modeling sequence is the wind and atmospheric pressure field model. For hindcasting historical storms, kinematic H*WIND and IOKA models that use data assimilation methods in order to define wind fields and pressure decay relationships in conjunction with observational data were employed. For synthetic hurricanes in the statistical storm set, a dynamic wind model, the Planetary Boundary Layer (PBL) model was applied. A comparative analysis was done between the PBL and Hurricane Boundary Layer (HBL). Models were run to determine the best fit for this analysis and the PBL was selected. An example of one storm track and its wind speed distribution is given in **Figure 2.4**.

It should be recognized that the wind forcing is not based on the ADCIRC grid geometry. Thus the surge responses that maintain similar topography in different grids (see section 2.4) will then be almost exactly the same from the same storm. The final maximum peak surge levels are the direct results of the wind forcing which is exactly the same in the base as well as the other geometry simulations. An introduction of levee barriers can and does produce non-similar results near and far from the geometry change. These changes are clearly seen in the analysis point locations,

especially where surges are greatly reduced with a barrier in place. The surge results at point locations within and outside of alternative levee configurations are analyzed and used to quantify the economic benefits and also compute levee heights.

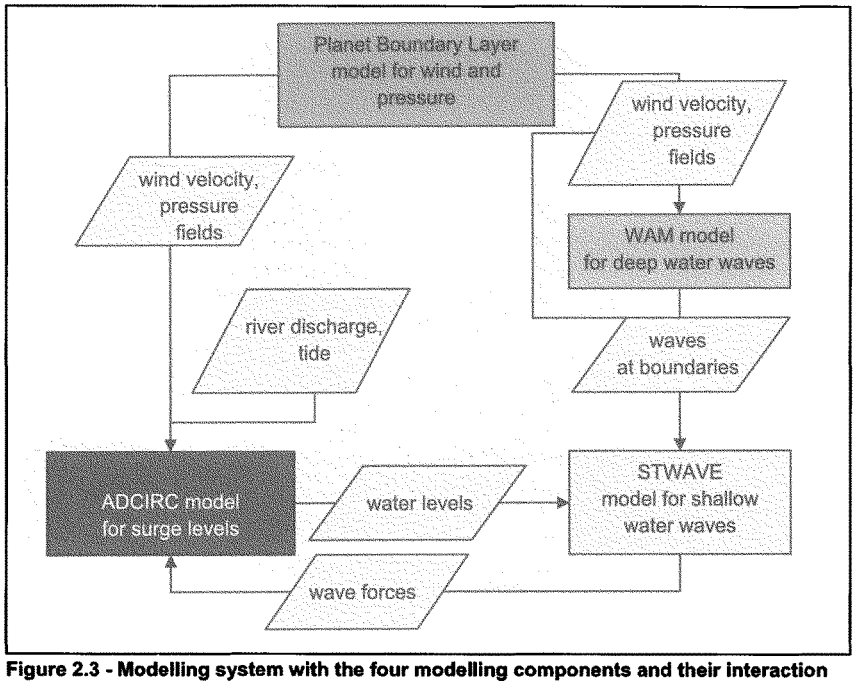


Figure 2.3 - Modelling system with the four modelling components and their interaction

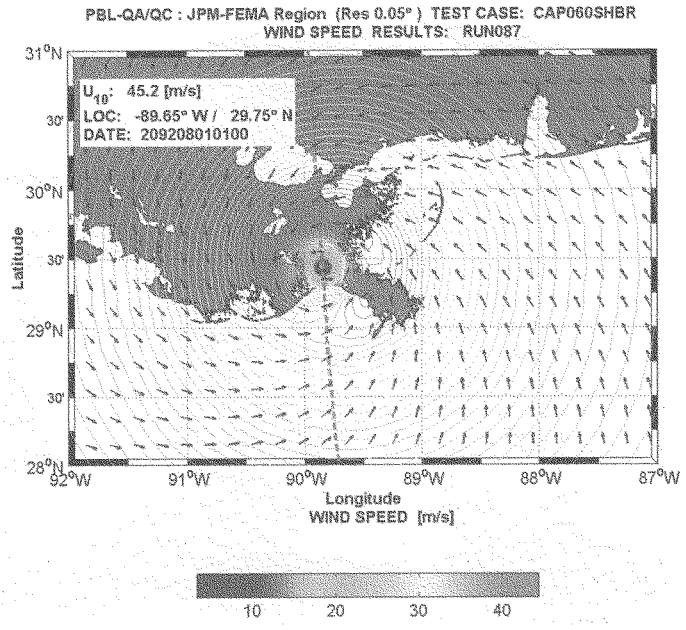


Figure 2.4 - Example of the wind field of one storm at landfall from the entire suite of 152 storms

Note: This specific storm has a minimum pressure of 900 mbar and a maximum radius of 17 nautical miles.

Once the winds are generated, the basin scale WAve prediction Model (WAM) is run in order to generate deep water waves in a Gulf of Mexico domain. These results are then applied as boundary conditions in a finer scale regional WAM model that covers the continental shelf in Southern Louisiana and Mississippi. The regional scale WAM results were then applied as boundary conditions in four to five regional finer scale STWAVE models that provide comprehensive coverage in Southern Louisiana (see **Figure 2.5**). The STWAVE computations also included water levels obtained from ADCIRC (see **Figure 2.6**). The last component to be applied was the ADCIRC hydrodynamic model, which is forced with wind and atmospheric pressure, wind wave radiation stresses from STWAVE, riverine flows and tides for hindcast cases.

There is significant interaction between the various component models. The wind models produce marine winds that are reduced for overland areas depending on the upwind roughness length scales and the existence of canopies. However, once an area is inundated, the physical

roughness elements are subject to immersion, and the nominal roughness length scales are subsequently reduced. Upon full immersion of the physical roughness elements, marine winds are again applied.

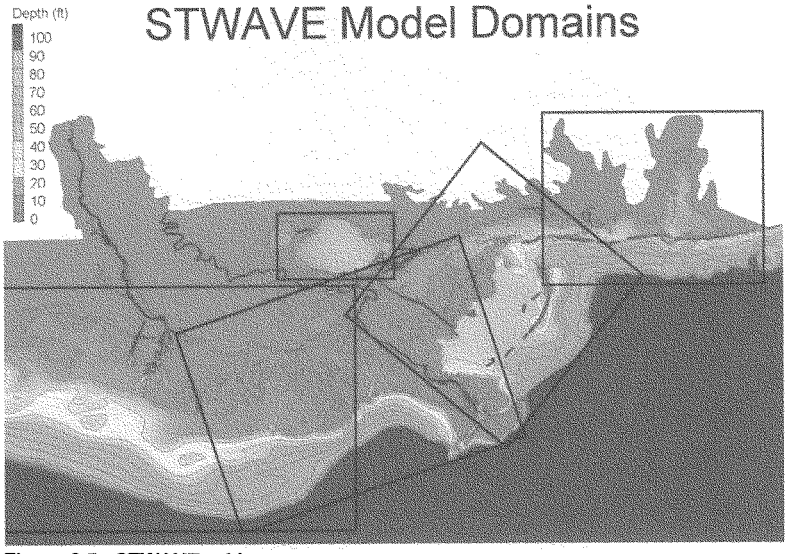


Figure 2.5 - STWAVE grid coverage

In addition to quantifying the wind waves themselves, wind-waves influence surge height with wind-wave radiation stress forcing, modify bottom friction as well as influence the sea surface roughness. Wind-waves reach shore prior to the peak surge driven by the strongest hurricane winds, so combined wind and wind-wave surge builds up earlier than solely wind driven surge. Furthermore, draw-down caused by winds coming from shore tends to be reduced by waves that are still coming into shore. In this modeling system, the interaction between the wind-waves and the surge is considered by applying wave radiation stress forcing. The effect on bottom friction or the influence of waves on surface roughness as they affect air-sea interaction, are not included since these effects are currently not well understood for hurricane conditions.

ADCIRC computations are forced with wave radiation stresses from the four to five localized STWAVE grid domains for western Louisiana, west of the Mississippi river, east of the Mississippi river, south of the Mississippi-Alabama coasts and within Lake Pontchartrain. The STWAVE computations themselves were made with boundary forcing information from the regional WAM model (which is forced with the Gulf wide WAM solution) as well as preliminary water level and current information from ADCIRC. The preliminary ADCIRC simulations included all forcing functions with the exception of the wave radiation stresses.

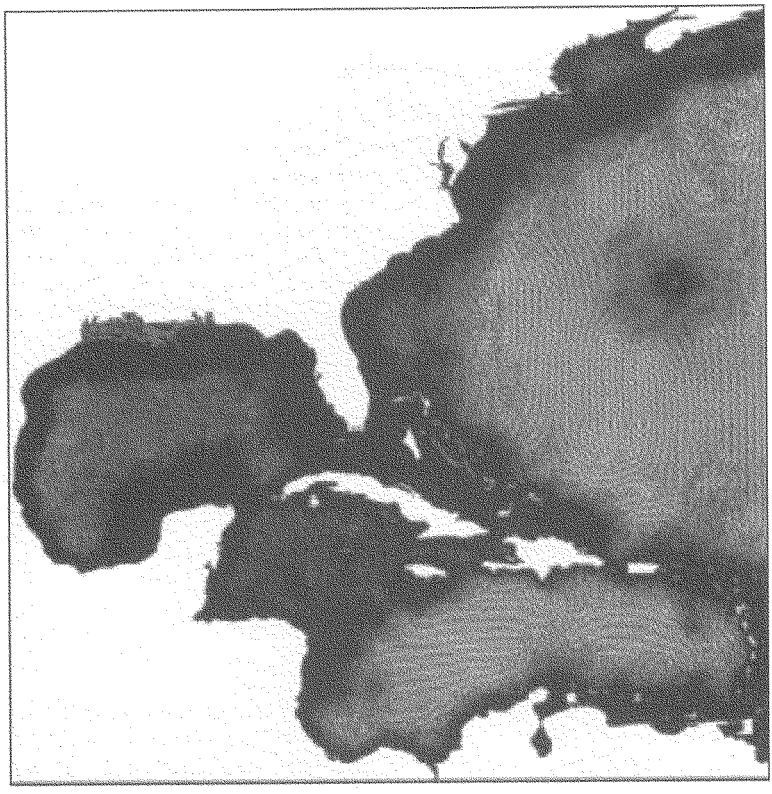


Figure 2.6 - ADCIRC grid coverage

In addition to the effects of waves, there can be significant effects on surge due to coastal tides and riverine currents. Because the tide range in Southern Louisiana is limited (about a 1.5 ft range), the nonlinear impact on the high water is limited. Therefore when looking at the statistical high water studies, tides can be linearly added in most areas without incurring significant error. However previous studies indicate that the shape of the tides themselves is significantly affected by the surge and therefore for purposes of model validation it is of significant interest to include them.

Finally it is noted that significant currents flow through the Mississippi and Atchafalaya Rivers and that these river currents strongly interact with tides and surge. For example, tides are substantially attenuated as they propagate up the Mississippi River for high flow/stages compared to low flow/stage. The level of interaction for storm surge wave propagating up the river is unknown but it may be important given the depth of the river and the magnitude of the currents. ADCIRC

computations were therefore made simultaneously including wind, atmospheric pressure, riverine flows, wave radiation stresses and for hindcast studies tides so that all significant coastal and riverine currents could fully interact nonlinearly in the computation.

2.3 Modeling validation and datum considerations

The effects of subsidence have created issues with the vertical datum that has plagued the engineering and surveying community in Southern Louisiana. Fortunately, in the last few years the National Geodetic Survey has developed a new method for updating vertical datum epochs to take into account subsidence and movement in control monuments. This new method has led to the creation of Vertical Time Dependent Positioning which requires the datum and its epoch to be listed together. At the time the LACPR effort began, the vertical datum in use was the NAVD 88 (2004.65) where 2004.65 is the datum epoch. This epoch has been superseded by a 2006.81 adjustment but to maintain continuity, the 2004.65 epoch will continue to be used for this effort. There are still many problems associated with trying to convert historical data such as gauge data, high water mark data, etc. into the new datum and epoch since the historical data is tied to older datum spanning numerous leveling epochs. The NAVD 88 (2004.65) datum will be used as the reference datum for all elevations in this report unless otherwise noted.

Design elevations referenced in this report were created using the same modeling, methodology, and data used to design the Greater New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS) work and to perform the FEMA flood insurance studies for South Louisiana (reference USACE/FEMA Louisiana and Texas Joint Coastal Storm Surge Study). Elevations used in developing the models incorporated the latest information on the relationship between water level reference surface (local mean sea level) and geodetic datum. Details can be found in draft reports done for the Joint Surge effort. All elevations are provided in NAVD88 (2004.65) which is the datum currently being used for all HSDRRS work.

Future detailed design and construction will be done using the most current HSDRRS design procedures and standards. During the design phase, gaging requirements will be established and gage(s) will be installed as required. The gage(s) will be used for determining the tidal datum local mean sea level prior to construction. Additional temporary gages may be required depending on vertical accuracy requirements. The gage(s) can also be used to monitor future hydrologic conditions in the area. The datum of the gage(s) has been established to comply with criteria contained in the Vertical Control Requirements for Engineering, Design, Construction, and Operation of Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects (Engineering Division Policy Memo #2).

The relationship between NAVD88 2004.65 and local mean sea level for the gage(s) will be reevaluated and reviewed by NOAA every 5 years (or more frequently if warranted based upon rate of subsidence). Vertical Datum Reports for each current HSDRRS polder are currently being prepared and will contain specific information on the gage network and the relationship between local mean sea level and NAVD 88 2004.65 for the project area. As new areas with HSDRRS projects are added reports for those areas will be produced.

The atmospheric-hydrodynamic modeling system was extensively validated. A brief summary of the validation is given here. For complete documentation on system validation, the reader is referred to earlier studies in which the modeling suite was applied (IPET 2006, FEMA 2007). The surge model was validated for Hurricanes Katrina and Rita. These storms were selected due to the unprecedented quality of the system definition, storm data, resulting high water marks (HWM) and the vertical leveling information. In addition the extent of inland inundation was unprecedented allowing for a unique opportunity to validate the effectiveness of modeling the effects of topography, overland resistance, and decreases in overland wind speeds. The offshore wave model was validated with data from Hurricanes Rita, Ivan, Camille, Katrina, and Andrew and the wind model was validated with data from these storms plus Hurricane Betsy. The nearshore wave model was compared to available data in Lake Pontchartrain acquired during Hurricane Katrina.

For the surge model, maximum surge levels were compared to between 80 and 204 open water and inland HWM's. Estimated model errors are based on these comparisons and the estimated accuracy of the HWM's themselves. As an example, **Figure 2.7** presents a comparison between the HWM's and the ADCIRC results for Katrina. The resulting modeling system error standard deviations, which include inaccuracies in the kinematic wind models, air-sea momentum transfer, wave radiation forcing, system definition and the hydrodynamic model itself, are estimated to be 1.47 ft (IPET HWM for Katrina), 1.36 ft (FEMA HWM for Katrina), and 1.21 ft (FEMA HWM for Rita). This indicates that about 68% of the predictions can be expected to be within 1.3 ft and 95% of the predictions can be expected to be within 2.6 ft of accuracy.

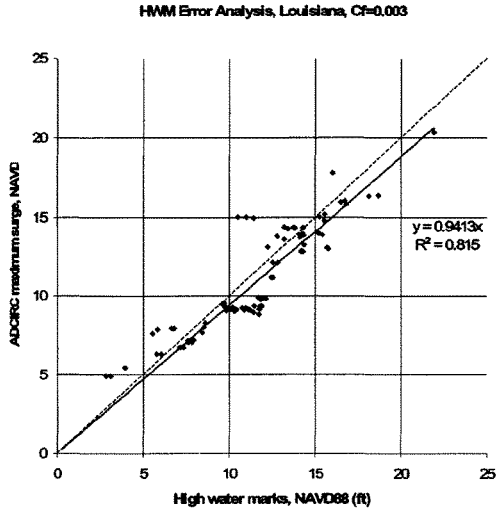


Figure 2.7 - Comparison between high water marks and ADCIRC results for Katrina

Winds were verified to point source measurements. The WAM model was validated to data from NOAA NDBC buoys for Hurricanes Rita, Katrina, Ivan, and Andrew and data from the Ocean Data Gathering Project for Hurricane Camille. The waves were validated against peak wave conditions and time variations during a storms passage. Few data are available to validate the nearshore wave model for hurricanes in Louisiana. However, two small wave buoys were deployed in Lake Pontchartrain during Katrina and results compared favorably to these data.

After completion of the surge and wave modeling, an independent analysis examined results from several nearshore wave models and a variety of conditions with a focus on wave energy dissipation effects. Careful review of simulated wave heights at some locations inshore of coastal marsh areas indicates that the with-friction STWAVE results may underestimate the wave height. In the interest of conservatism and in the absence of field-verified values for friction coefficients due to bottom and vegetation interaction, the design process applied STWAVE simulations without frictional dissipation. Uncertainty in future location and density of coastal marshes, in part due to local subsidence and lack of appropriated funding for marsh restoration, provides additional rationale for excluding the effects of friction in the nearshore wave simulations. Future planned efforts to obtain the necessary field data along with more accurate estimates of future wetland conditions should provide improved quantitative estimates of friction coefficients suitable for design purposes.

2.4 Description of grids and/or conditions modeled

Various hydraulic conditions were modeled either for sensitivity analyses or to apply the results to the LACPR alternatives. The evaluated ADCIRC grids and/or conditions and the number of modeled storms are summarized in **Table 2.2**. These grids/conditions are discussed in further detail in the paragraphs following the table.

Table 2.2 - Description of grids/conditions and the number of storms used to model each

Grid/conditions	Short description	No. of storms
2007 Base Conditions (East and West grids)	Update to the 2005 hindcast grid. Represents the Greater New Orleans Hurricane and Storm Damage Risk Reduction System that existed in the summer of 2007.	152 (East) 152 (West)
2010 Base Conditions (East and West grids)	LACPR base condition for comparison to alternatives. The East grid models the Greater New Orleans levee system in 2011 with barrier at MRGO/IHNC and 100-year levee heights around the entire system. The 2010 West grid is the same as the 2007 West grid (no existing levees).	56 (East) 152 (West)
East A grid	Full closure of Lake Pontchartrain along US90; full closure of IHNC/GIWW along west shore of Lake Borgne; full closure West Bank from between Belle Chasse to Larose along GIWW.	48
East B grid	Weir closure of Lake Pontchartrain along US90 with structures in Chef and Rigolets tidal passes; full closure of IHNC/GIWW along west shore of Lake Borgne; weir closure West Bank from Belle Chasse to Larose along GIWW.	42 / 152
East C grid	Weir closure of Lake Pontchartrain along US90 without structures in Chef and Rigolets tidal passes; full closure of IHNC/GIWW along west shore of Lake Borgne; weir closure West Bank from Belle Chasse to Larose along GIWW.	48
East D grid	Isolating Lakes Pontchartrain and Borgne from each other by building a levee across Lake Borgne from Verret to Slidell; full closure West Bank from Belle Chasse to Larose along GIWW.	40

West A grid	Non-overtopping levee alignment from Larose to Golden Meadow and along GIWW.	28
West B grid	100-year level alignment from Larose to Golden Meadow; a non-overtopping levee along the ridge and a ring levee alignment in the western part.	28
West C grid	Non-overtopping levee alignment from Larose to Golden Meadow and along the ridge, and an overtopping levee along GIWW with a ring levee around Lake Charles.	28
Plaquemines Parish	Option 1: Two spillways in the levee system along Plaquemines Option 2: Full removal of levee system along Plaquemines to river embankment level (for sensitivity analysis only)	17 17
Landscape Conditions	1) Degraded marshes 50 years from now without increased action 2) Restored marshes 50 years from now based on a hypothetical alternative (for sensitivity analysis only)	174 46
Sea level Rise Sensitivity Analysis	1) +1 ft sea level rise 2) +2 ft sea level rise 3) +3ft sea level rise	9 9 9
Barrier Islands Sensitivity Analysis	1) No barrier island 2) Restored Island 3) Post-Katrina with forest 4) Restored island with forest	15 15 15 15

2007 Base Conditions

In order to update the 2005 hindcast grid to the system that existed in the summer of 2007, levee definitions were updated to reflect the system upgrades that were implemented as part of the USACE Task Force Guardian and by the USACE Hurricane Protection Office and New Orleans District. This system was then run with the 152 storms for eastern Louisiana and 152 storms for western Louisiana in order to define 100- and 500-year water levels and corresponding wave conditions. This information served as a base condition for various sensitivity analyses that were performed prior to the development of the 2010 base grids. This grid was then replaced with the 2010 East and West base grids for evaluation of the LACPR alternatives as described below.

2010 Base Conditions

In addition to evaluating the 2007 system, the proposed system improvements anticipated for 2010 (now 2011) were investigated. This included raising levees across the system as well as a closure of the combined MRGO/GIWW east of Paris Road. Note that the area west of Larose to Golden Meadow is the same in both the 2007 and 2010 base case.

LACPR East Levee System Configurations in Planning Units 1 and 2

In order to understand the performance and implications of a variety of levee system improvements as developed by the USACE and the State of Louisiana's Coastal Protection and Restoration Authority, four east levee configurations were evaluated for Planning Unit 1 and 2. The modeled levee configurations are the so-called East A, B, C and D grids. The east configurations included a variety of alignments and elements that are summarized in **Table 2.3**. **Figure 2.8** and **Figure 2.9** present the levee alignments for East A, B, C, D.

LACPR West Levee System Configurations in Planning Units 3a, 3b, and 4

Similar to the LACPR East alignments, three different west alignments were examined with various configurations including a solid line of levees that runs north of the bays and lakes of western Louisiana as well as more localized ring levees that locally protect the population centers. The west configurations considered included a variety of alignments and elements that are summarized in **Table 2.4**. **Figure 2.10** and **Figure 2.11** present the levee alignments for West A, B and C.

Plaquemines Parish River and Back Levees (Plaquemines-1 and Plaquemines-2)

In order to understand the influence of the Mississippi River levees and adjacent back levees in lower Plaquemines Parish, spillways were incorporated into these levees (option 1) and the levees were entirely eliminated (option 2). This study component was designed to understand how surge builds up along these levees from Breton Sound and propagates towards New Orleans and Baton Rouge in the Mississippi River. In addition, the effectiveness of building localized ring levees to provide a higher level of risk reduction in Lower Plaquemines Parish can be ascertained.

Landscape Conditions

The landscape conditions included a predicted wetland definition 50 years into the future for two cases: 1. no action and 2. a restored/improved marsh condition (for sensitivity analysis only).

The no action condition was developed as part of the Coastal Louisiana Ecosystem Assessment and Restoration (CLEAR) Program. The forecasting model developed by CLEAR predicts physical processes, geomorphic features, water quality, and ecological succession. Geomorphic/bathymetric changes are based on the likelihood of discretized regions changing from open water to marsh or marsh to open water. The future condition of coastal Louisiana predicted by CLEAR, referred to as the degraded condition, in fact does predict degradation in southern Louisiana, but also predicts growth in the Atchafalaya basin and Plaquemines Parish. The CLEAR future condition bathymetry was applied to the model grids and mesh and a series of storm simulations was made.

The restored condition was developed by Engineer Research and Development Center's Coastal and Hydraulics Laboratory (ERDC-CHL) as a sensitivity analysis. The District provided ERDC-CHL with marsh creation locations and type, freshwater diversion locations, and the volume of sediment diverted for a hypothetic alternative. ERDC-CHL implemented these restoration features into a marsh creation program and modifications were made to the bathymetry, Manning's n values, and directional roughness lengths. These changes were applied to the model grids, mesh, and frictional files and a series of storm simulations was made.

Sea Level Rise Effects (Sensitivity Analysis)

Annex A describes an analysis of the relative sea level rise (includes subsidence) in detail for the Chenier Plain, the Delta Plain and the Pontchartrain Basin. The relative sea level rise is estimated at 1 – 3 ft based on two future projections. Based on this analysis, it was decided to evaluate the effect of sea level rise by applying a 1, 2, and 3-ft change in the vertical datum.

Influence of Barrier Islands (Sensitivity Analysis)

A sensitivity analysis was performed to assess the impact of bathymetric and frictional resistance changes for the barrier islands on ADCIRC-simulated peak surge elevations and STWAVE-simulated waves. The sensitivity storm suite consisted of fifteen storms of varying intensities and five barrier island configurations. The barrier island configurations modeled were:

1) no barrier islands with open water Manning's n value = 0.02;
2) a restored barrier island configuration of 12 ft (NAVD88 2004.65) for Cat Island, Ship Island, Horn Island, Petit Bois Island, and Dauphin Island and 6 ft (NAVD88 2004.65) for the Chandeleur Islands;
3) the existing Post-Katrina degraded condition with a forest Manning's n = 0.15;
4) a restored barrier island configuration with a forest Manning's n = 0.15.

Table 2.3 - Overview of measures in LACPR East grid models for Planning Units 1 and 2

Unit / Model grid	Planning Unit 1 (Lake Pontchartrain Basin)	Planning Unit 2 (Barataria Basin)
Base condition	Current levee system to 100-year level of risk reduction/authorized grade (whichever is greater) in combination with barrier at MRGO	Current levee system to 100-year level of risk reduction/authorized grade (whichever is greater)
East A	Modeling a non-overtopping levee adjacent to or on US 90 to close Lake Pontchartrain Modeling a non-overtopping levee along the west shore of Lake Borgne	Modeling non-overtopping levees along the GIWW as well a closures further north Modeling a non-overtopping levee following a southern alignment from Larose to Morgan City
East B	Modeling a weir at 12.5ft adjacent to or on US 90 and closure gates in the Rigolets and Chef Menteur Passes Modeling a non-overtopping levee along the west shore of Lake Borgne	Modeling an overtopping levee at 12.5ft along the GIWW as well a closures further north Modeling an overtopping levee at 100-year level following a southern alignment from Larose to Morgan City and a non-overtopping back levee along GIWW
East C	Modeling a non-overtopping levee adjacent to or on US 90 and openings in the Rigolets and Chef Menteur Passes Modeling a non-overtopping levee along the west shore of Lake Borgne	Modeling a non-overtopping levee following the US90 alignment with a central overtopping weir Modeling an overtopping levee at 100-year level following a southern alignment from Larose to Morgan City and a non-overtopping back levee along GIWW
East D	Modeling a non-overtopping levee across Lake Borgne from Verret to Slidell	Modeling non-overtopping levees along the GIWW as well a closures further north Modeling a non-overtopping levee following a southern alignment from Larose to Morgan City

Table 2.4 - Overview of measures in LACPR West grid models for Planning Units 3a, 3b, and 4

Unit / Model grid	Planning Unit 3a	Planning Unit 3b	Planning Unit 4
Base condition	2007 situation	2007 situation	2007 situation
West A	Non-overtopping levee (10 m) along Larose to Golden Meadow alignment	Non-overtopping levee (10 m) along GIWW	Non-overtopping levee (10 m) along GIWW
West B	Levee at 100-year elevation along Larose to Golden Meadow alignment with non-overtopping back levee along GIWW	Non-overtopping levee along ridge north of GIWW	Non-overtopping (10 m) ring levee alignment Lake Charles, Vinton, Kaplan, and Gueydan
West C	Non-overtopping levee (10 m) along Larose to Golden Meadow alignment (similar to West A)	Non-overtopping levee along ridge north of GIWW (similar to West B)	Non-overtopping (10 m) ring levee alignment Lake Charles in combination with overtopping levee along GIWW alignment

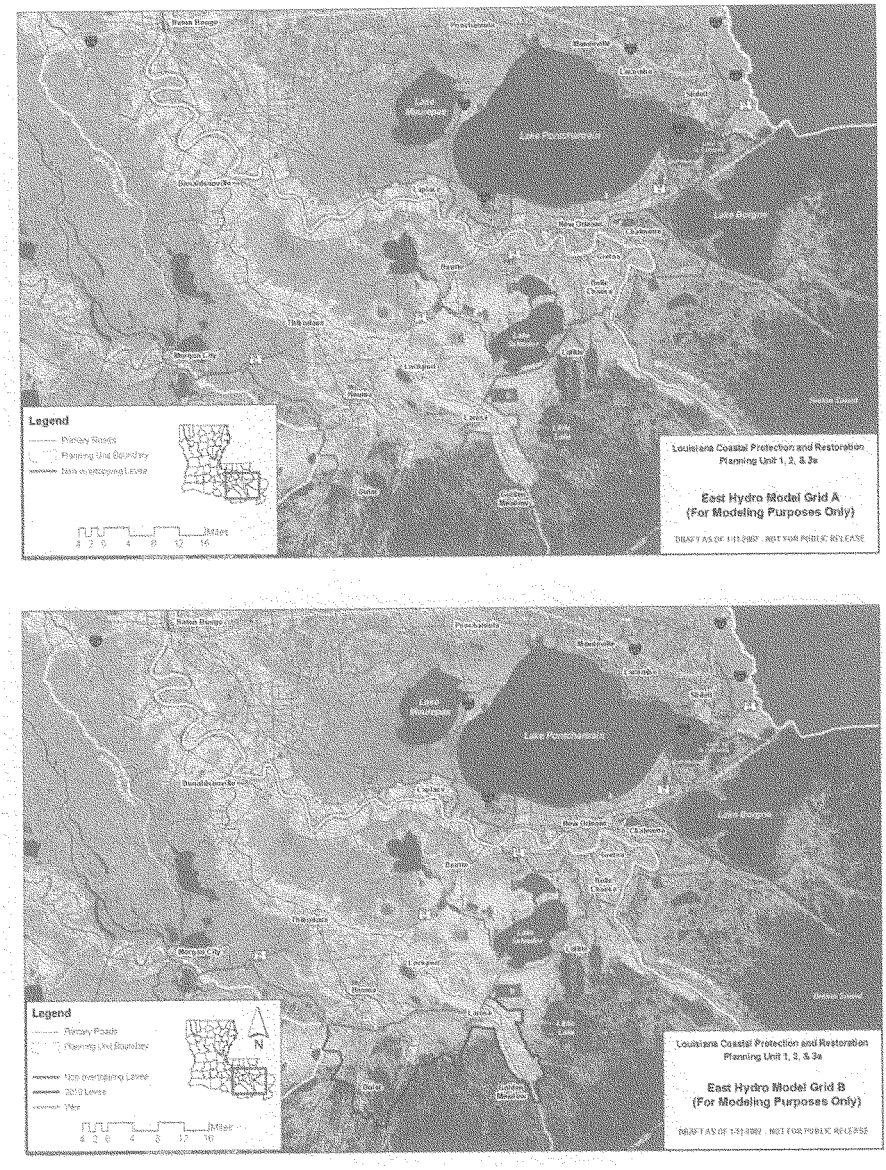


Figure 2.8 – Levee alignments in the East A and East B grids

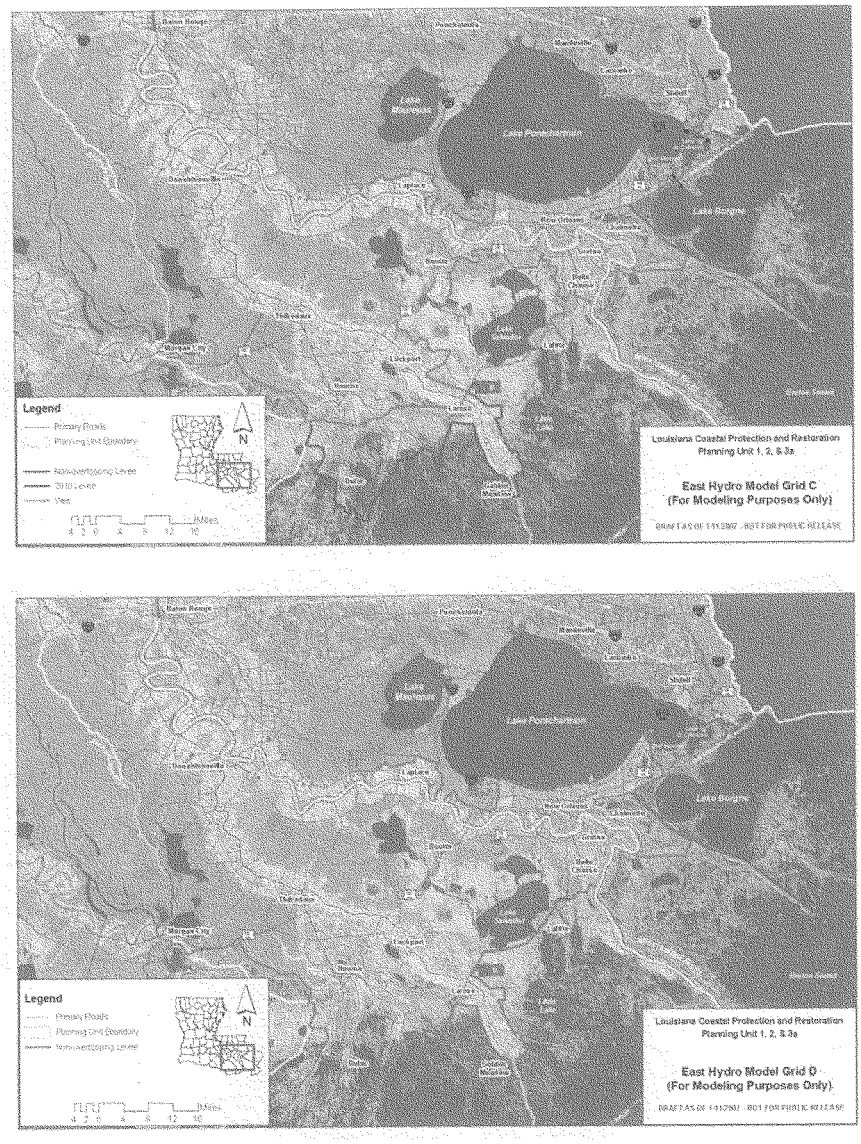
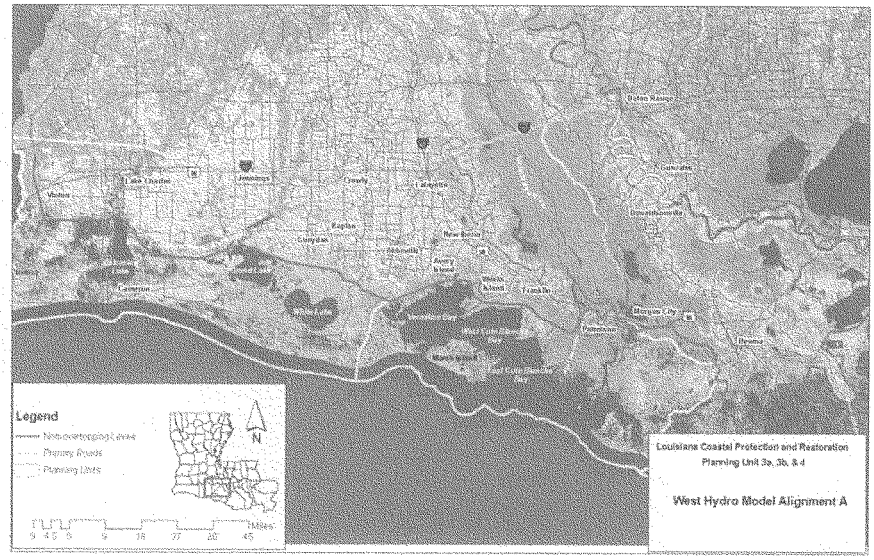


Figure 2.9 – Levee alignments in the East C and East D grids



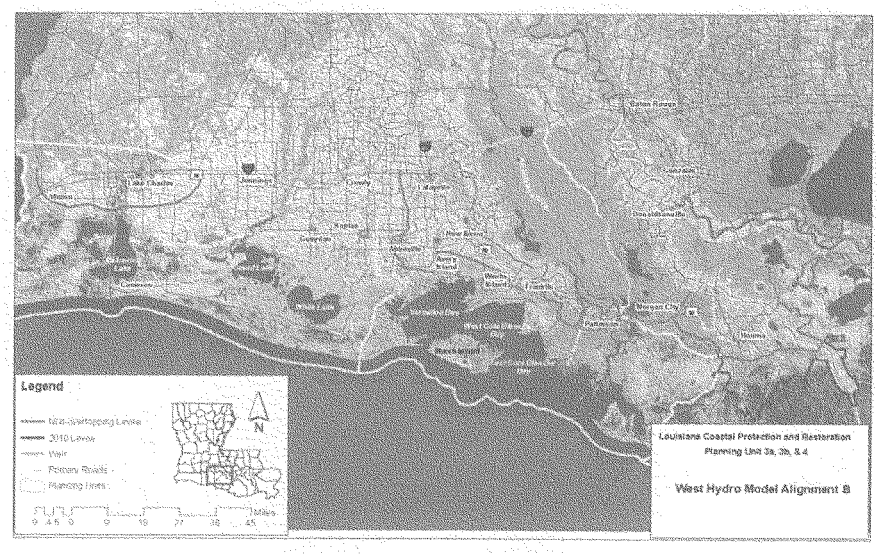


Figure 2.10 – Levee alignments in the West A and West B grids

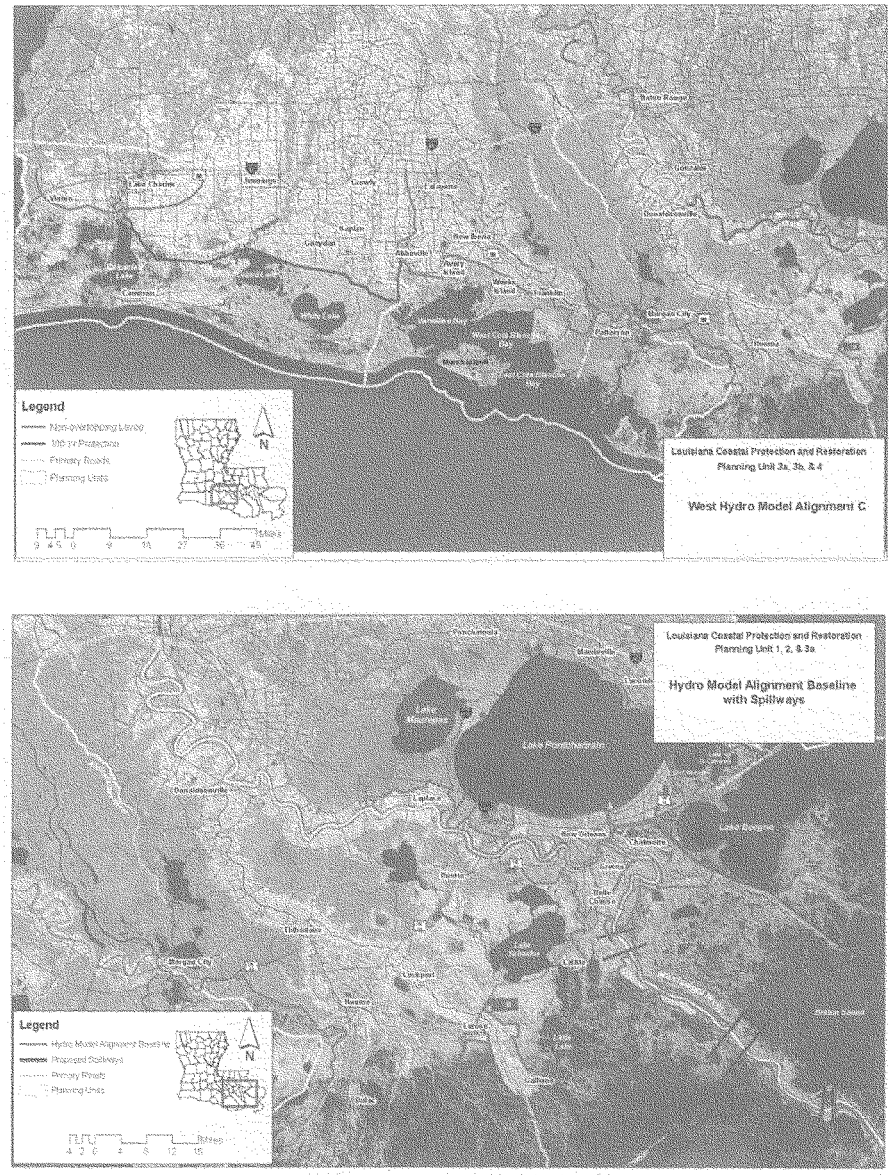


Figure 2.11 – Alignments in the West C grid (upper panel) and Plaquemines 1 grid (lower panel)

2.5 Storm selection

Given time constraints and potentially significant computation requirements, not all 152 storms could be simulated for the numerous hydraulic conditions. Thus, only a subset of the 152 storms was simulated for each condition. The selected subset was created by selecting storms whose tracks and characteristics spanned the range of parameter space defined in the JPM-OS methodology. Additionally, the subset of storms was based on the degree and location of the changed geometry for each condition.

For example, 56 storms were selected for the 2010 baseline conditions based storm characteristics as well as geographic areas they affected and the 2010 geometry that was different than the 2007 geometry.

Table 2.5 lists the storms simulated for the East grids. The storms are ordered in groups as defined in the JPM-OS White Paper (see Resio et al, 2007). These storms characteristics cover most of the important range of parameter space and thus provide a confident response surface generated from the JPM-OS code. As an example, two storm tracks of this set are shown in **Figure 2.12**. Similarly, the storms for the West grids are listed in **Table 2.6**.

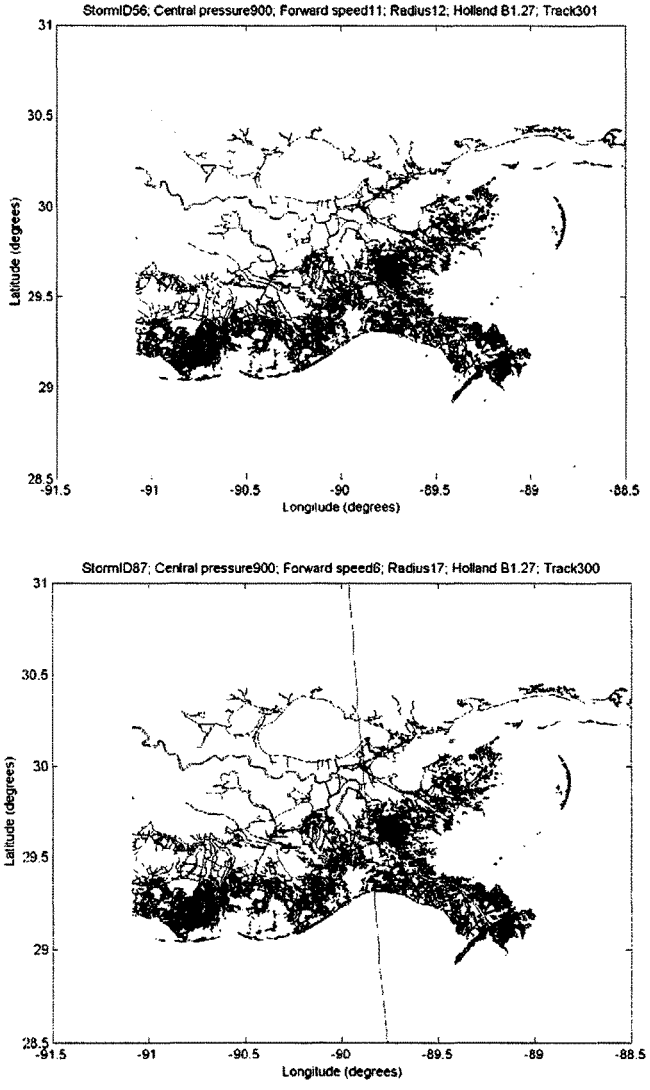


Figure 2.12 - Examples of storm tracks (storms 56 and 87)

Table 2.5 - Selected storms for the East grids

East A (48)		East B* (42)		East C (48)		East D (50)		Plaq Spill 1 (17)	Plaq Spill 2 (17)	Barrier islands (15)	Marsh degraded (24)	Marsh restored (24)
5	69	5	75	5	69	5	75	14	14	29	2	2
6	71	6	77	6	71	6	77	15	15	32	3	3
8	73	8	83	8	73	8	83	17	17	35	5	5
9	75	9	85	9	75	9	85	18	18	36	6	6
14	77	14	86	14	77	14	86	23	23	38	8	8
15	82	15	87	15	82	15	87	24	24	41	9	9
17	83	17	88	17	83	17	88	26	26	44	11	11
18	84	18	89	18	84	18	89	27	27	45	12	12
23	85	23	92	23	85	23	92	52	52	59	14	14
24	86	24	93	24	86	24	93	53	53	61	15	15
26	87	26	94	26	87	26	94	56	56	87	17	17
27	88	27	95	27	88	27	95	57	57	88	18	18
32	89	33	97	32	89	33	98	69	69	89	20	20
33	90	35	98	33	90	35	99	73	73	90	21	21
35	91	36	99	35	91	36	100	77	77	91	23	23
36	92	49	100	36	92	49		32	32		24	24
48	93	52		48	93	52		35	35		26	26
49	94	53		49	94	53					27	27
52	95	56		52	95	56					29	29
53	97	57		53	97	57					30	30
56	98	60		56	98	60					32	32
57	99	61		57	99	61					33	33
60	100	69		60	100	69					35	35
61		71		61		71					36	36
67		73		67		73						

Table 2.6 - Selected storms for West grids

West A	West B	West C	Marsh degraded	Marsh restored
203	203	203	202	202
205	205	205	205	205
208	208	208	211	211
209	209	209	214	214
212	212	212	217	217
214	214	214	220	220
217	217	217	223	223
218	218	218	226	226
221	221	221	229	229
223	223	223	232	232
226	226	226	235	235
227	227	227	238	238
230	230	230	241	241
232	232	232	244	244
235	235	235	267	267
236	236	236	269	269
239	239	239	271	271
241	241	241	273	273
244	244	244	275	275
245	245	245	277	277
315	315	315	279	279
316	316	316	281	281
317	317	317		
318	318	318		
319	319	319		
320	320	320		
321	321	321		
322	322	322		

2.6 Point sets

A number of different model output point sets have been developed to present the results from the ADCIRC and STWAVE modeling. **Table 2.7** lists the three different point sets that have been used within the LACPR technical evaluation.

Table 2.5 - Overview of point sets used

Point set	Purpose
L 274	To select data for levee height design and overtopping rates for the east grid
W 177	To select data for levee height design and overtopping rates for the west grid
Q 835	To evaluate potential impacts of alternatives on the Mississippi coastline

These point sets are visualized in **Figure 2.13** (east grid) and **Figure 2.14** (west grid) below.

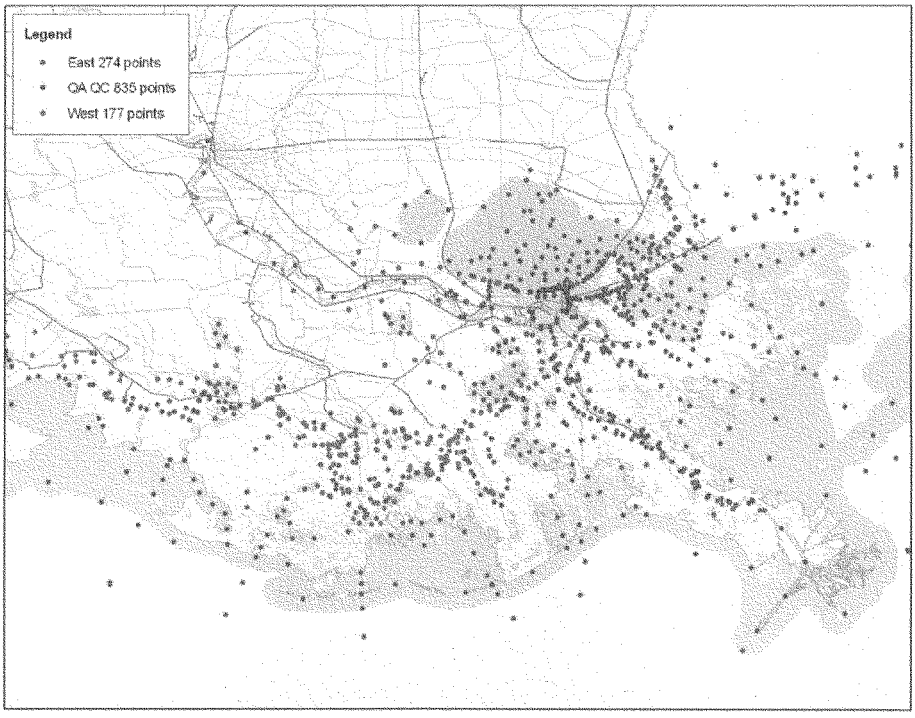


Figure 2.13 - Point sets (east grid)

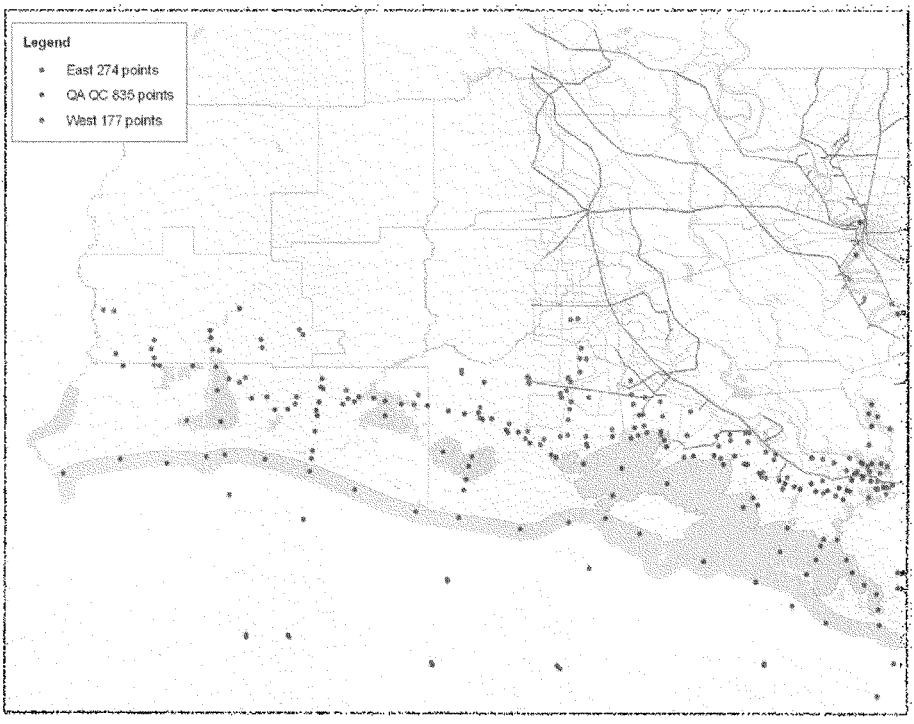


Figure 2.14 - Point sets (west grid)

3 FREQUENCY ANALYSIS WITH JPM-OS METHOD

This chapter describes the development of the frequency statistics for the surge level and wave characteristics. Inputs for this analysis are the results from the ADCIRC and STWAVE computations (**Figure 3.1**). The key element in this frequency analysis is the Joint Probability Method with Optimal Sampling (JPM-OS method). The background of this method is briefly summarized in Section 3.1. The JPM-OS method requires a full set of 152 storms to compute the frequency statistics for surge and wave characteristics. However, for all LACPR levee alignments less storms were simulated due to time constraints. To use the method for these levee alignments a fitting procedure was developed for surge levels (Section 3.2) and wave characteristics (Section 3.3) so as to create values for all 152 storms. To check the validity of this procedure, a check has been carried out for one specific alternative, see Section 3.4. The frequency statistics are used as input into the determination of the levee heights (Chapter 4) and the overtopping volumes (Chapter 5), and they are also used to provide stage frequency results for areas outside of a levee system.

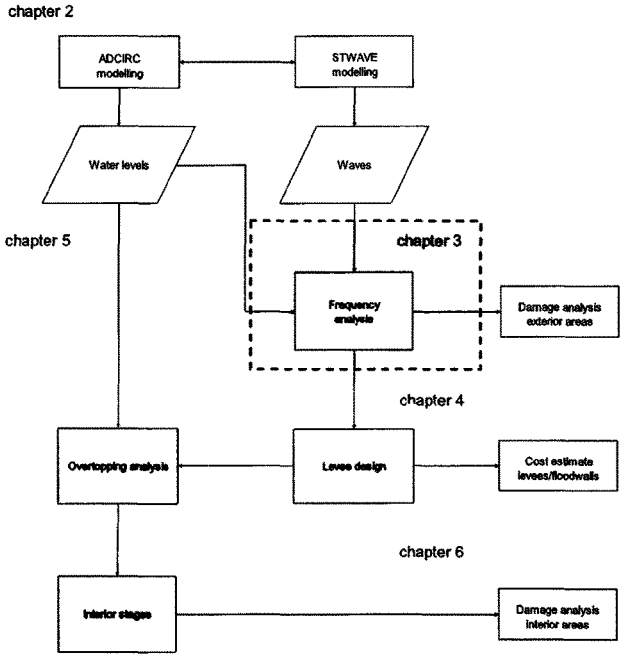


Figure 3.1 - Flow diagram of hydraulic analysis in LACPR framework (Chapter 3)

3.1 JPM-OS method

In 2006 and 2007, a team from the Corps of Engineers, FEMA, NOAA, private sector, and academia developed a new process for estimating hurricane inundation probabilities, the Joint Probability Method with Optimal Sampling process (JPM-OS). This work was initially begun for the Louisiana Coastal Protection and Restoration technical evaluation (LACPR), but now is being applied to Corps work under the 4th supplemental appropriation, the Interagency Performance Evaluation Team (IPET) risk analysis, and FEMA Base Flood Elevations for production of DFIRMs for coastal Mississippi, Louisiana, and Texas.

For most Joint Probability Methods, several thousand events are evaluated. With the JPM-OS method, optimal sampling allows for a smaller number of events to be used. The JPM-OS method computes the frequency of occurrence of surges at specific geographic points or stations. For each of these points a surge response from each of 152 specific storms is required. The JPM-OS method has been used to derive the still water elevation, wave height, and wave period frequency curves at specific points using output from ADCIRC and STWAVE. JPM-OS takes into account the joint probability of forward speed, size, minimum pressure, angle of approach and geographic distribution of the hurricanes. For more details, the reader is referred to Resio et al. (2007), see Annex B.

The output from the ADCIRC and STWAVE models used in the JPM-OS analysis are the maximum still water elevation and maximum wave characteristics (significant wave height, peak period, and wave direction) at specified points. An example of the model output at two locations is shown in **Figure 3.2**. The wave characteristics along Lake Pontchartrain are typically wind-generated and depth-limited waves. There is a high correlation between the wave height and the wave period and between the surge level and wave height for this area. In contrast, the results at the MRGO are much more scattered. The relationship between the surge level and the wave height is less evident, and the wave period strongly varies as a function of the wave height. Long wave periods are observed for a few storm conditions. The long wave periods are related to swell waves from the ocean.

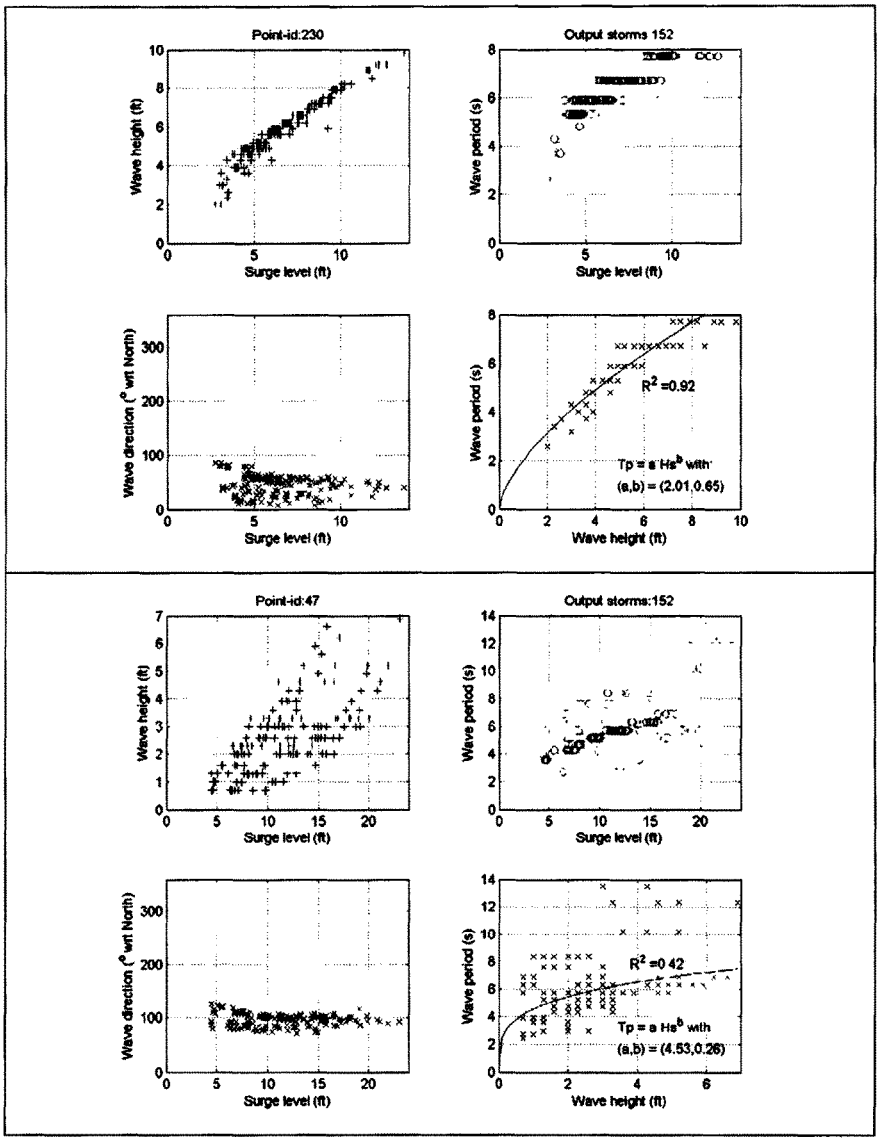


Figure 3.2 - Numerical results at Lake Pontchartrain (upper panel) and MRGO (lower panel) from ADCIRC and STWAVE (2007 base condition)

Note: The marks (o, x, +) represent 152 storm results.

Surge level frequency curves can be estimated from output from the 152 storms. Along the West Bank, there were instances where there was no output from the 152 storms because these points are dry for a specific storm. In this case, estimates were made of the surge elevation for the missing output so that the frequency analysis continued to be based on 152 values. For the nodes which formed the ADCIRC grids, the topographic elevation was modified for all cases where the surge value for all storms did not produce a surge at that node. After this multiplication, an additional check was made and if the surge was less than 0.0, the surge was set to 2.5 feet.

The original set of 152 storms was selected in such a way that it covered the probabilities in the range of 1/50 – 1/3,500 per year with main emphasis on the range 1/50 – 1/500 year. In the framework of LACPR, the frequency analysis with the JPM-OS method ranges from 1/100 per year to 1/2,000 per year. The 1/2,000 year return period is near the upper end of the original storm set limits and it can be expected that the results for the upper end are more uncertain than the results for the 1/100 – 1/1,000 year range. Nevertheless, we believe that the results can be used after careful checks within the LACPR evaluation because the main purpose is a relative comparison between the various alternatives during these events rather than an exact determination of the hydraulic boundary conditions for these extreme events.

3.2 Fit procedure for surge levels

The LACPR analysis evaluates alternative storm surge risk reduction systems using many levee alignments. ADCIRC grid geometry was created to model the system and provide results from which levee heights can be determined. As described in Section 2.5, a subset of storms was selected from the suite of 152 storms for simulation on the appropriate geometry. For instance, only 48 storms were computed for the 2010 LACPR East A grid.

In order to use the JPM-OS software to create statistical files to compare against the original 152 storms modeled for the 2007 base condition, a surge value was needed for the storms not simulated for that particular geometry. Commonly there is a relationship between the original results from the 2007 base condition and the results for the other conditions (2010 base condition, East A grid, East B grid, etc.). If no variances exist in a specific area, one may expect similar results for the 2007 condition and another condition. If changes to the nearby coastal hydrodynamic features have occurred however (e.g. adding a barrier), one may expect an altered response in the distinctive condition surge levels. To find a relationship between the surge level effect of a specific condition (i.e., East A grid) and the original surge level (2007 base), we examined the results for a few cases (**Figure 3.3** and **Figure 3.4**) in the New Orleans area. Note that the 2010 case includes the barrier at the entrance of GIWW in the New Orleans East area.

Based on inspection of various plots, we have chosen to use the following relationship between the effect on the maximum surge level and the original maximum surge level of 2007:

$$\Delta\zeta_{2010-2007} = a_1\zeta_{2007} + a_2\zeta_{2007}^2 \qquad \text{Equation (1)}$$

where:
$\Delta\zeta$: difference in maximum surge level [ft]

ζ_{2007} : maximum surge level 2007 [ft]
a_1, a_2 : coefficients [-, 1/ft]

The coefficients a_1, a_2 are fitted using the data of the storms available using a MATLAB routine.

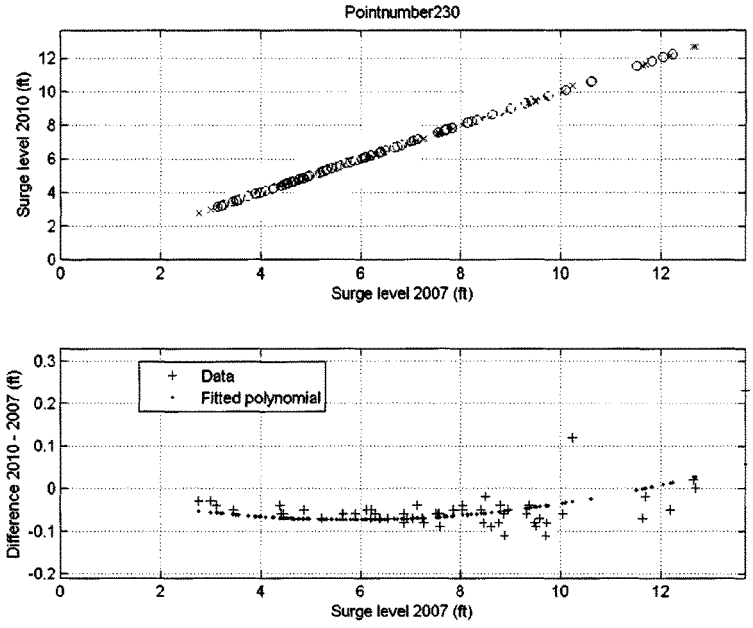


Figure 3.3 - Correlation between maximum surge levels at Lake Pontchartrain for 2010 base condition and 2007 base condition

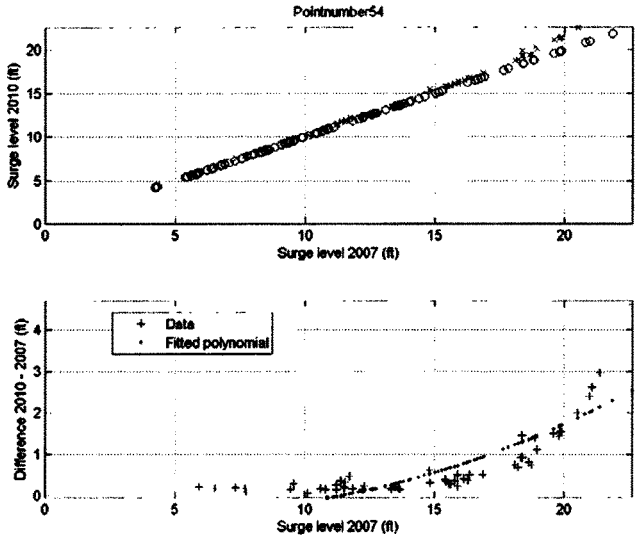


Figure 3.4 - Correlation between maximum surge levels at Lake Pontchartrain for 2010 base condition and 2007 base condition

The final step for the surge levels is to compute the 152 storm results for the new situation (2010, 2010 LACPR East A, 2010 LACPR East B, etc.). For this purpose, the fitted line according to Eq. (1) has been used for all storms (including the storms that were originally run for the new situation). The 152 results for the new situation are used as input for the probabilistic JPM-OS method to obtain the frequency curves.

For specific cases, the correlation of the fit is relatively low. One example is shown in **Figure 3.5**. This plot shows the effect on the surge levels for 2010 LACPR East A condition (i.e. full closure of Lake Pontchartrain). As can be observed, the correlation between the surge level of the base case (2007 conditions) and the effect of the surge level between 2010 LACPR East A and 2007 conditions, according to Eq. (1) is not very good. Despite this low correlation, we have produced 152 storm results based on this fit and computed the frequency curve using the JPM-OS method. Note, that points of no-data (-99999) are discarded and not used in the created polynomial curve fit between the datasets.

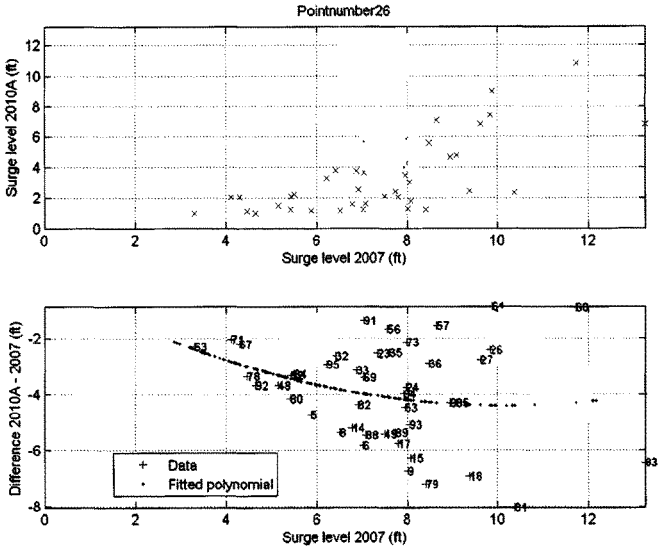


Figure 3.5 - Effect of maximum surge level at Lake Pontchartrain (2010 East A grid) with fit for all storms

Due to the low correlation we have investigated whether the various storm tracks could explain this low correlation. In the **Figure 3.6** the colored dots indicate the various hurricane tracks: black = track 1, green = track 2, red = track 3, blue = track 4. It can be observed that the relationship between the surge level of 2007 conditions and the effect of the surge level for the data points of track 1 (black points) is very good. The relationships for the other tracks are not as strong. Nonetheless they are considerably better than the fit based on all storms. Because of this, we have produced fits for each storm track separately using Eq. (1) and computed surge levels for the 152 storms applied to the 2010 LACPR East A scenario using the track information of each storm. These results have been used to compute the frequency curves for the surge levels.

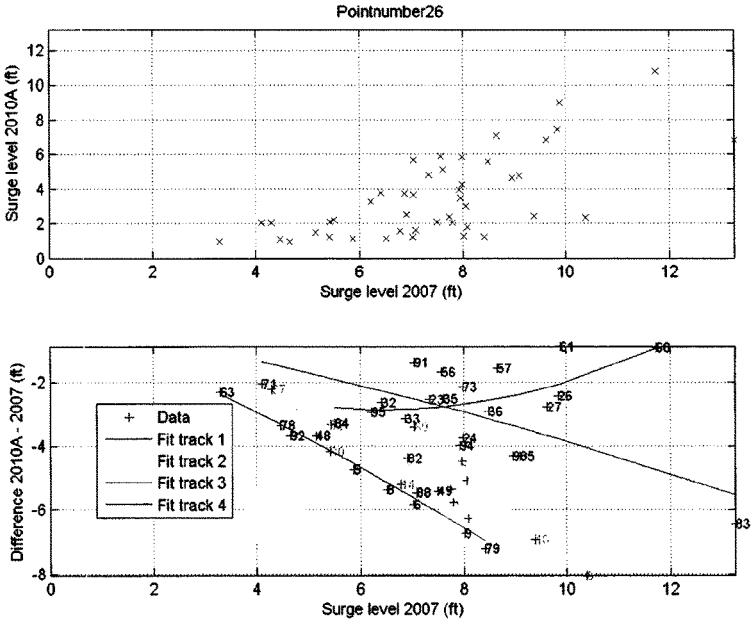


Figure 3.6 - Effect of maximum surge level at Lake Pontchartrain (2010 East A grid) with fit for different storm tracks

A comparison was made between the 1% surge levels based on both methods, viz. the fit with multiple curves based on storm tracks and the results from a single curve with without regard to storm track. Although the data fits appear to be much better for each track separately, it appears that the final 1% surge levels differ less than 0.5ft. Nevertheless, the fitting procedure based on the multiple tracks has been applied throughout the entire LACPR evaluation.

3.3 Fit procedure for wave characteristics

Similarly to the surge levels, the wave characteristics of the various conditions (2010 base, 2010 LACPR East A, 2010 LACPR East B, etc.) are also likely to be related to wave characteristics of the base case (2007 conditions). However, the relationship between the waves from 2007 and the other conditions (2010, 2010 LACPR East A, etc.) appears to be much less strong than for the maximum surge levels. This has to do with the sensitivity of the wave characteristics to small water level changes. Another issue is that the roughness formulation in STWAVE has been changed for the 2010 conditions (and other LACPR alternative conditions). The STWAVE model was executed with no bottom friction formulation for the 2007 conditions. Especially near the levee, the roughness influence is relatively high because of the limited water depth. Therefore, a good fit

between the original 2007 condition wave results and the new 2010 condition wave results cannot be expected.

To circumvent this problem, we have chosen to make a fit between the surge level and the wave characteristics for each alternative condition. Based on plots we have adopted the following relationships (see also **Figure 3.7** and **Figure 3.8**):

$$H_s = a_1 \zeta^{b_1}$$
$$T_p = a_2 \zeta^{b_2}$$

Equation (2)

where:
H_s : significant wave height [ft]
T_p : peak period [s]
ζ : Maximum surge level [ft]
a_1, b_1, b_2, a_2: coefficients

The coefficients a_1, a_2, b_1 and b_2 were fitted using the data of the storms available using a MATLAB routine.

The final step for the wave characteristics was to compute the 152 storm results for the new situation (2010, 2010 East A, East B, etc.). For this purpose, the fitted line according to Eq. (2) has been used for all storms (including the storms that were originally run for the new situation) using the fitted surge levels for that specific grid. Note, similarly to maximum surge levels, point locations with no-data were discarded and not used to create the fit.

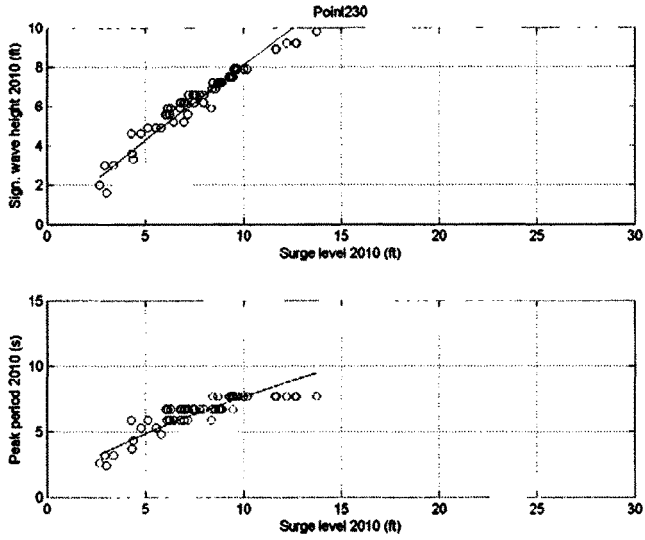


Figure 3.7 - Wave characteristics at Lake Pontchartrain for 56 storms

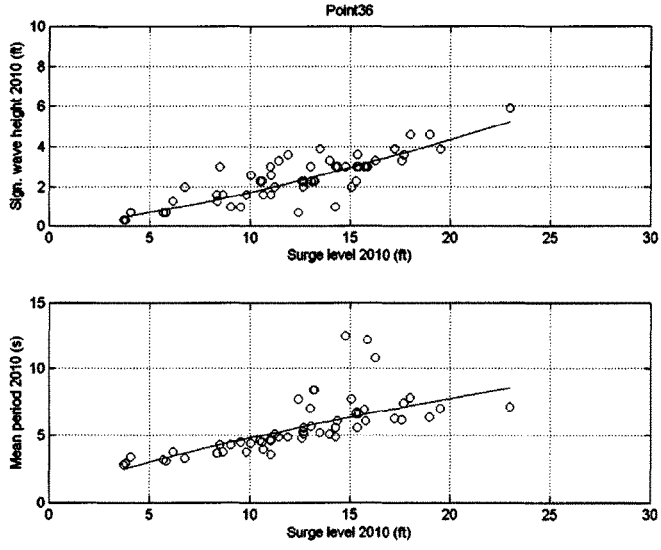


Figure 3.8 - Wave characteristics at MRGO for 56 storms

Summarizing, frequency statistics are computed for surge, wave heights, and wave periods, for all LACPR alternative conditions. For each LACPR alternative condition, using the curve fitting methodology described in this and the previous section, point files were created for surge, for wave heights, and for wave periods. Each file contains 152 values derived as above. These files are then input to the probabilistic JPM-OS software to obtain the frequency curves for the surge, wave heights, and wave periods. In this manner, 50-year, 100-year, ..., 1950-year, 2000-year values are obtained for surges, as well as wave heights, and wave periods. In addition the standard deviation of the surge level as a function of frequency is also obtained.

3.4 Validity of fit procedure

As listed in **Table 2.2**, a subset of 42 storms was selected for simulation for the East B grid. In order to validate the surge fitting model previously described, the remaining 112 storms of the full suite of 152 storms were simulated for the East B levee configuration. Statistics were computed for all of the point sets using the JPM-OS code to produce the full range of returns from the 50- through 2000-year return values. The L274 point group was selected for initial evaluation. A table of differences was computed for the 100 -year surge values between the fitted model results and the full suite of 152 storm results. **Figure 3.9** shows the 100-year return values resulting from the analysis of East B using the full 152 storm suite versus the 40 storm suite.

As can be seen in **Figure 3.9** the results are almost identical and follow a straight line which indicates they are the same. It appeared that approximately 30% (91 points) of the 100-year surges for the 152 storm set were between 0.1 to 1.3 feet less than the fitted model values. Approximately 24% (67 points) were exactly the same 100-year surge elevations. The remaining 56% were between 0.1 to 1.4 feet above the fitted model values. There were 2 outliers of 7.8 and 4.2 feet. These were located away from the coast, towards the upper Pearl River Basin. The larger differences for these 2 points are most likely due to data processing errors. For the 100-year surge levels, a standard deviation of 0.63 feet was computed for the absolute difference between the results based on the full storm suite and the 40 storm suite. Also, if the 2 outlier points are disregarded, the standard deviation is 0.34 feet. Thus, the fitted model procedure results agree relatively well with the full 152 storm suite results.

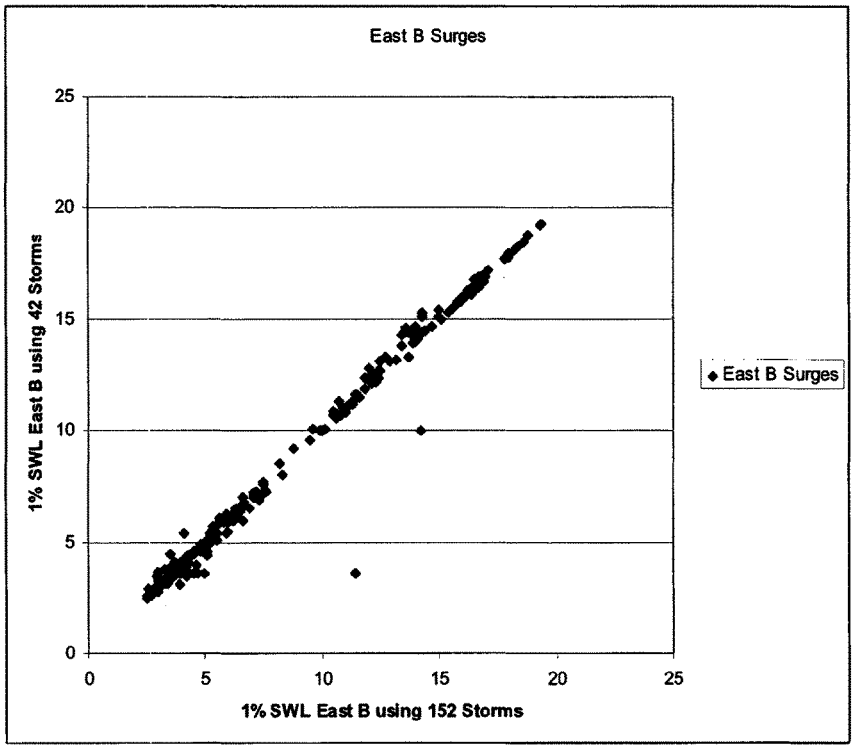


Figure 3.9 - Comparison between the 1% still water levels based on the full storm set of 152 storms and based on the 42 storms using the fit procedure

4 DETERMINATION OF LEVEE HEIGHTS

This chapter gives an overview of the design approach for the levee heights. The frequency results of the various hydraulic variables are the inputs for this analysis (**Figure 4.1**). The design procedure adopted herein has been developed in the framework of the current 100-year levee and floodwall design effort for the Greater New Orleans Hurricane and Storm Damage Risk Reduction System at the New Orleans District. Several simplifications have been applied to make this procedure applicable for LACPR.

The outline of this chapter is as follows. First, the step-wise approach for the levee design is presented and the simplifications in the LACPR technical evaluation are described (Section 4.1). Next, the general assumptions of the levee design approach are discussed in more detail (Section 4.2). Finally, the procedure to account for uncertainties in the levee design procedure is briefly explained (Section 4.3). The final levee heights are input into the determination of the overtopping volumes (Chapter 5) and for producing construction cost estimates. The cost estimates of the levee designs are described in a separate report.

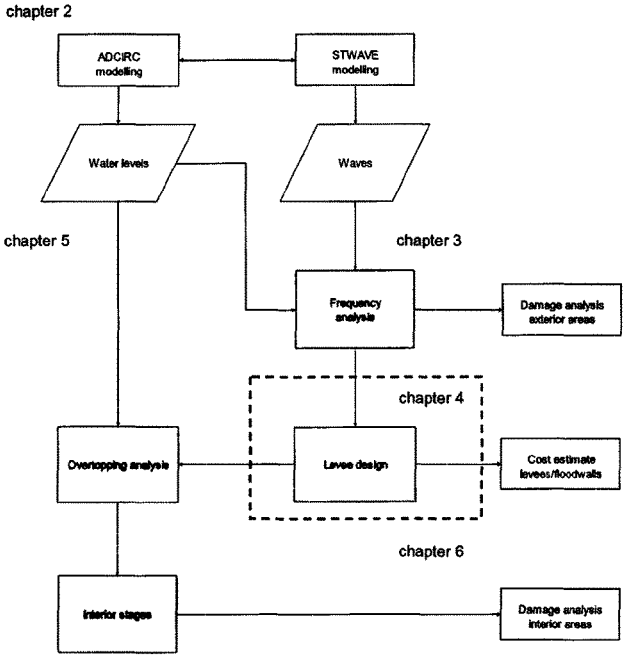


Figure 4.1 - Flow diagram of hydraulic analysis in LACPR framework (Chapter 4)

4.1 Step-wise design approach

The design procedure below gives a step-wise approach for determining the levee height, within the framework of LACPR, from a hydraulic perspective. The step-wise approach is intended to be used for each section that is more or less uniform in terms of hydraulic boundary conditions (water levels, and wave characteristics) and geometry (levee, floodwall, structure). The procedure has been developed within the framework of the current 100-year levee and floodwall design effort for the Greater New Orleans Storm Damage and Hurricane Risk Reduction System at the New Orleans District.

A levee design was made for three different levels of risk reduction (100-year, 400-year and 1000-year). Several simplifications have been applied in the step-wise approach to make the procedure applicable and suitable for LACPR. The step-wise design approach for a given return period has been adopted for LACPR as described in **Table 4.1**:

Table 4.1 – Step-wise approach

Step	Description
0 – Definition of reaches	For each planning subunit, the surge levels and wave characteristics are examined. Based on the variation in the hydraulic boundary conditions and the orientation, the flood risk reduction system was divided into one or more reaches. For each reach, one or more suitable output locations were selected from the LACPR point set.
1 – Water elevation	For each levee reach the surge levels from the frequency analysis were reviewed. Based on the quality of the data a suitable output point was selected. Volume II (Results) discusses in detail the selected output points for all reaches.
2 – Wave characteristics	The wave characteristics were extracted from the same output location as the surge levels. The wave height at the toe of the structure is assumed to be reduced as a result of depth-limited breaking according to $H_{smax} = 0.4$ h. The wave period has not been changed.
3 – Overtopping rates	The overtopping rate is computed using the Van der Meer formulations (see textbox). For this purpose, a simplified levee design is assumed (**Figure 4.2**). The steep sloping sections near the crest and near the toe are assumed to be 1:4. In between, a wave berm is present to reduce the amount of overtopping. For all cases, the wave berm factor (γ_b) is set at 0.7 in the Van der Meer equations and the slope equals 1:4. The other influence factors regarding wave incidence, roughness and vertical wall are all set to 1.0. Hence, we assume a perpendicular wave attack against a grass-sloped levee without a wall on top.
4 – Monte Carlo simulations	The final step is a Monte Carlo simulation to compute the overtopping rate from step 3 a large number of times (5,000). Every time, the hydraulic variables and the coefficients of the overtopping equation are changed to account for the uncertainties in these parameters. The approach is explained in detail in the Section 4.3. Based on the 5,000 results, the probability distribution of the overtopping rate is determined. A check is carried out to see if the overtopping rate does not exceed the overtopping criterion of 0.1 cfs per ft with 90% confidence. If yes, the design process is finished and the levee height is set. If not, the levee height is lowered and the calculation repeated until this criterion is reached (see also Annex B).

For the purpose of LACPR, the Van der Meer equations have been adopted to compute overtopping rates for levee sections. The overtopping formulations from Van der Meer are (see TAW document):

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

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

With:
q : overtopping rate [cfs/ft]
g : gravitational acceleration [ft/s^2]
H_{m0} : wave height at toe of the structure [ft]
ξ_0: surf similarity parameter [-]
α : slope [-]
R_c : freeboard [ft]
γ : coefficient for presence of berm (b), friction (f), wave incidence (β), vertical wall (v)

The coefficients -4.75 and -2.6 in Eq. 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 (see TAW document).

Eq. 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 eq. 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 wave length. 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 (see TAW document).

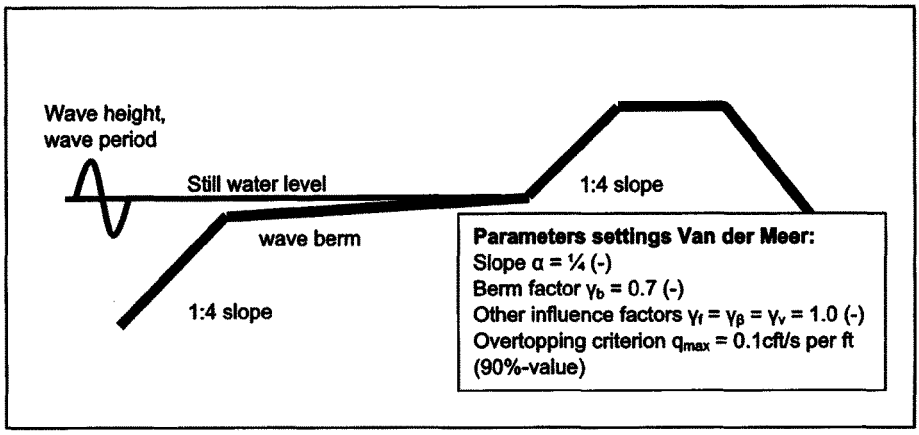


Figure 4.2 - Simplified levee cross section in LACPR evaluation

4.2 Design assumptions

This section briefly discusses the most important choices and assumptions in the design approach. These items are:

- Full dependency between surge levels and waves
- Simultaneous occurrence of maxima
- Breaker parameter
- Overtopping criteria

Full dependency between surge levels and waves

The step-wise design approach below is (partly) probabilistic in the sense that it makes use of the derived water levels and wave characteristics based on the JPM-OS method (see also Chapter 3). The procedure also includes an uncertainty analysis that accounts for uncertainties in the hydraulic parameters and the overtopping coefficients. However, the approach is not fully probabilistic because the correlation between water elevation and wave characteristics is not taken into account. This assumption is an important restriction to this approach. It is likely that the presented approach is conservative because the correlation between the surge elevation and the wave characteristics is not taken into account. Depending on the situation, the impact of this assumption on the final levee height can be minimal to significant (> 1ft).

Simultaneous occurrence of maxima

Another assumption in the design approach is that the maximum water elevation and the maximum wave height occur simultaneously. Analysis of the ADCIRC and STWAVE results shows that the time lag between the peak of the surge elevation and the wave characteristics at both

sites is small (< 1 hour). It should be noted that there are cases in which the time lag between surge and waves is larger (say 1 – 2 hours). Although this assumption may be conservative for some locations, assuming a coincidence of maximum surge and maximum waves is reasonable for most of the levee and floodwall sections in our design approach.

Breaker parameter

In the design approach we compute the overtopping rates based on empirical formulations. One of the inputs to these formulations is the wave height at the toe of the structure. This value is not known, but is estimated based on the wave results from STWAVE. Because the foreshore is generally very shallow (same order as the wave height), wave breaking will play an important role in the final 600ft before the toe of the structure (floodwall/levee height). Hence, it is not likely that the wave height at 600ft in front of the structure will be equal to the wave height at the toe of the structure, but will be lower.

To account for breaking in front of the levee/floodwall we have reduced the wave height from STWAVE. An estimate of the wave height at the toe of the structure has been made by making use of a breaker parameter. The breaker parameter is the ratio between the significant wave height and the local water depth. In the literature, the breaker parameter is often a constant or it is expressed as a function of bottom slope or incident wave. A typical range for this parameter is between 0.5 – 0.78 for engineering purposes. These values are generally obtained for situations with a mild sloping bed.

However, laboratory experiments and Boussinesq runs suggest that a breaker parameter of 0.4 is a realistic choice for a relatively long shallow foreshore, as is the case around New Orleans. This value has therefore been used in the entire design approach to translate the significant wave heights based on STWAVE to the significant wave height at the toe of the structure. The wave periods from STWAVE have been used without modification.

Overtopping criterion

Hughes (2007) carried out a literature survey to underpin the overtopping criterion value that has been used in the ongoing one-percent design for the Greater New Orleans Storm Damage and Hurricane Risk Reduction System (see USACE, 2007). The survey showed that although various numbers have been proposed, the experimental validation of these numbers is very limited. Typical values are: (see also TAW, 2002):

- 0.001 cfs/linear ft (cfs/ft) for sandy soil with a poor grass cover;
- 0.01 cfs/ft for clayey soil with a reasonably good grass cover;
- 0.1 cfs/ft for a clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope.

In spring 2007, USACE decided to make use of a maximum overtopping criterion of 0.1 cft/s per ft. This implies that the inner slope of the clay levee/floodwall has a well-maintained grass cover. An assurance criterion of at least 90% was used in accordance with the latest Corps guidelines (April

2007). In the framework of LACPR this criterion has been applied without changes for all design events (100-year, 400-year, 1000-year)[3].

4.3 Uncertainty analysis

The design criterion in the framework of LACPR is defined as follows: *the overtopping rate should be less than 0.1 cft/s per ft with 90% assurance".* To determine this overtopping rate, a Monte Carlo analysis has been carried out that accounts for uncertainties in water elevations, waves and the coefficients in the overtopping formulations. Notice that we neglect the uncertainties in the geometrical parameters. In other words: we assume that the proposed heights and slopes in this design document are minimum values achieved during construction. The text below gives a brief description of this method. For more information, the reader is referred to USACE (2007).

The probability density distributions of the hydraulic variables and the coefficients in the wave overtopping formulation are inputs into the Monte Carlo Simulation. Frequency results of the surge levels and the waves were used from the JPM-OS method. These values are the so-called "best estimates" (or mean values). An additional analysis has provided the standard deviation in the 1% still water elevation. Standard deviation values of 10% of the average significant wave height and 20% of the peak period were used; these were based on expert judgment (Smith, pers. comm.). The standard deviations of the coefficients in the Van der Meer formulations are described in the textbox in section 4.1. All uncertainties are assumed to normally distributed.

The Monte Carlo Analysis applied herein is executed as follows:

a) Draw a random number between 0 and 1 to set the exceedance probability p.
b) Compute the water level from a normal distribution using the expected value 1% surge level and standard deviation as parameters and with an exceedance probability p.
c) Draw a random number between 0 and 1 to set the exceedance probability p.
d) Compute the wave height and wave period from a normal distribution using the expected value 1% wave height and 1% wave period and the associated standard deviations and with an exceedance probability p.
e) Repeat step 3 and 4 for the three overtopping coefficients in the overtopping formula, independently, using estimates of variability (standard deviation) in each coefficient.
f) Compute the overtopping rate for these hydraulic parameters and overtopping coefficients
g) Repeat the steps 1 through 5 a large number of times (N = 5,000)
h) Compute the 50%, 90% and 95% value of the overtopping rate (i.e. q_{50}, q_{90} and q_{95})

The procedure is implemented in MATLAB. Several test runs show that 5,000 runs are sufficient to reach statistically stationary results for q_{50}, q_{90} and q_{95}. The computation time to perform this

[3] Note that the overtopping criteria have been slightly changed for the 1% design effort in August 2007 after consultation of ASCE review team (USACE, 2007). The overtopping rate should also be less than 0.01 cfs per ft at the 50% confidence limit. Additional analysis shows that this criterion is almost everywhere fulfilled with the original criterion. The LACPR methodology has therefore not been updated with this extra criterion.

analysis is in the order of tens of seconds on a current state of the art personal computer. Thus, the proposed method is straightforward and can be applied in a relatively quick way.

Figure 4.3 shows the result of this design process. The probability of non-exceedance is shown as a function of the overtopping rate. The levee design height for this specific section is 24ft. With this height, the 90% overtopping rate is 0.082 cfs per ft which meets the design criterion.

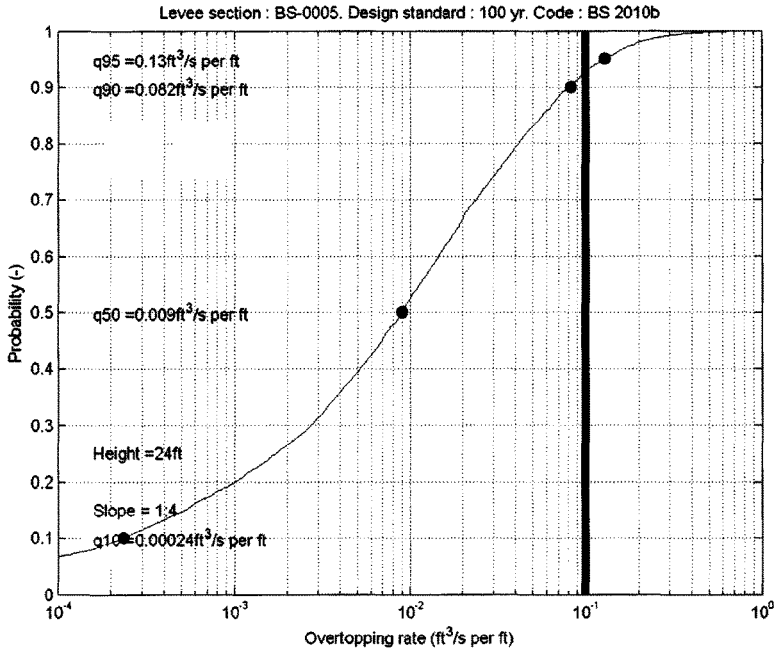


Figure 4.3 - Result for 100-year design height along MRGO levee (2010 base condition)

5 OVERTOPPING RATES

This chapter describes the determination of the overtopping rates (**Figure 5.1**). Within the framework of LACPR an estimate of the overtopping rate are needed for a given return period. The temporal variation of the hydraulic boundary conditions to compute the overtopping rate for a given return period is not easily available from ADCIRC and STWAVE. Therefore, the temporal variation of the surge level and the wave characteristics is parameterized (Section 5.1). These three load factors are used as input to the overtopping formulae, in addition to the design heights of the levees to compute the overtopping rates (Section 5.2). The overtopping rates are used as input for the interior stage analysis (Chapter 6).

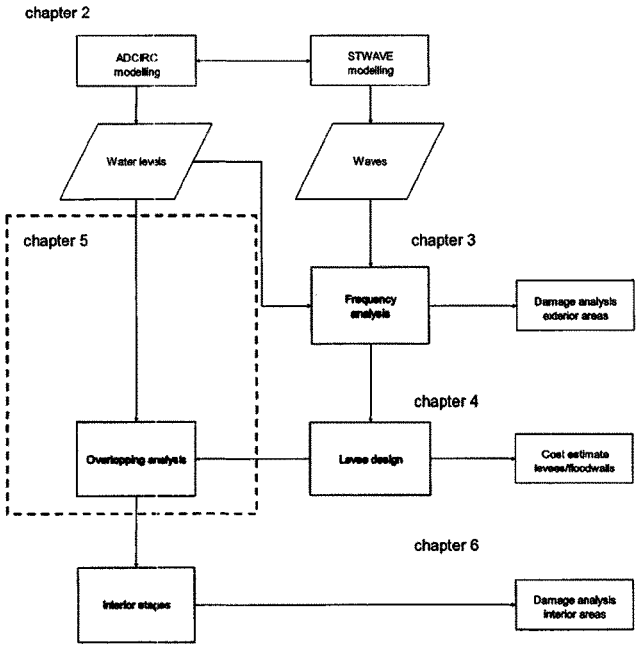


Figure 5.1 - Flow diagram of hydraulic analysis in LACPR framework (Chapter 5)

5.1 Parametrical description of hydrographs

The overtopping discharge, whether due to wave overtopping or free overflow, is determined from the variation in time of the surge level and the wave characteristics. In the framework of LACPR the overtopping rates need to be determined for a given return period. To estimate these rates, a

description is needed of the temporal variation of the hydraulic boundary conditions for that specific return period. However, these variations in time are not directly available from the numerical models ADCIRC and STWAVE. Time series are available for a specific set of storms (with a maximum of 152) for the water level, wave height and wave period. A typical hydrograph at a given point is given in **Figure 5.2**.

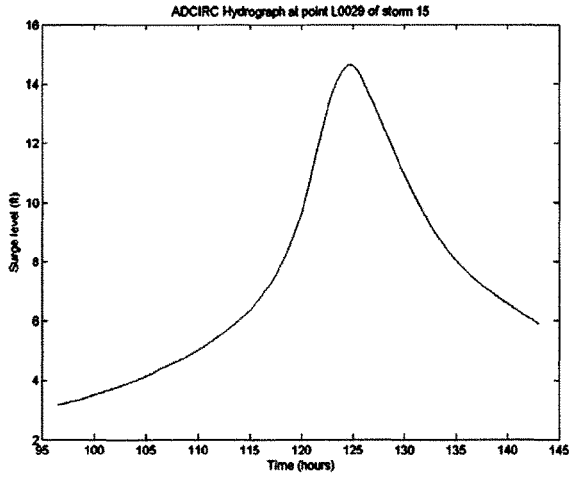


Figure 5.2 - Example of hydrograph from ADCIRC results

Within the framework of the JPM-OS method, a conditional approach has been adopted (see Resio, 2007). This implies that all parameters can be determined as a function of the surge level (η_{max}) for a given return period (1/100 years, 1/500 years, etc). Along similar lines, the shape of a hydrograph is also likely to be correlated to the maximum surge level. One may expect that a correlation can be found between the maximum surge level and the width of a hydrograph, *normalized by the maximum surge height*. This means that the shape of the hydrograph is more peaked for large surges than for a smaller surge at the same location. Although the maximum surge level of a hydrograph at a given location is much higher, the width of the normalized hydrograph will be less than the width of a normalized hydrograph corresponding to a smaller maximum surge level.

Based on these considerations, a parametric hydrograph has been developed which takes into account the variation of the shape of the hydrograph for the 152 (or less) storms. For this process, we have chosen to assume a Gaussian shape for the hydrograph:

$$\frac{\eta(t)}{\eta_{\max}} = e^{\frac{-(t-t_{\eta\max})^2}{2\sigma^2}}$$

with:
η : surge level [ft]
σ : width of hydrograph [hrs]
$t_{\eta max}$: moment with maximum surge level [hrs]

Because the hydrographs clearly show an asymmetric behavior with time, a distinction has been made between the surge level curve before the peak and after the peak. For both sides, the width of the hydrograph is estimated from the zero-th and second-order moments for the upper 30% of the normalized hydrograph (Note: the subscripts l and r refer to left-hand and right-hand side of the hydrograph):

$$\sigma_{l,r} = \sqrt{\frac{\int_{\tau(0.7*\eta_{max})_{l,r}}^{\tau(\eta_{max})} t^2 \eta(t) dt}{\int_{\tau(0.7*\eta_{max})_{l,r}}^{\tau(\eta_{max})} \eta(t) dt}}$$

with:
σ : width of hydrograph [hrs]
t : time [hrs]

Figure 5.3 presents one example of the real hydrograph from ADCIRC and the estimated Gaussian shaped hydrograph. Based on visual inspection it can be concluded that the shape of the top of the hydrograph is well represented by the fitted Gaussian formula. A more detailed view of the same comparison is shown in **Figure 5.4**. This figure presents a comparison between the real and the parameterized hydrograph at one location for one storm. The dots represent the output from ADCIRC in time along the hydrograph. It can be observed that the fit is good (R^2 = 0.99).

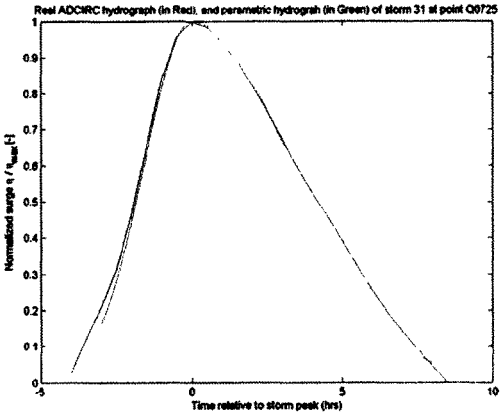


Figure 5.3 - Real hydrograph from ADCIRC (in red) and parameterized hydrograph (in red)

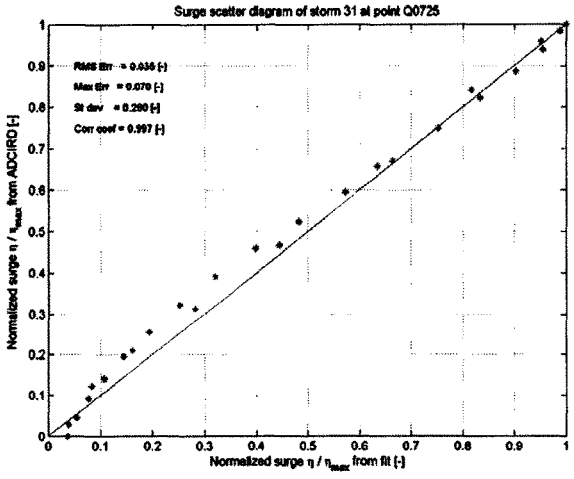


Figure 5.4 - Normalized surge from ADCIRC plotted against the normalized parameterized surge

The next step is to establish a relationship between the width of the hydrograph and the maximum surge level. A log-linear fit has been used. **Figure 5.5** presents this relationship for one output point in which the crosses represent the storms. Although the scatter is quite large, there is a visible tendency for smaller widths with higher surge levels. Furthermore, it appears that the correlation seems to be better for higher surge levels. This is related to the fact that the upper 70% of the hydrograph of a severe storm scenario has a better defined peak compared with a mild to

moderate storm. Because the scatter in the fits cannot be disregarded, this aspect has been taken into account in the uncertainty analysis of the overtopping discharges. This will be further discussed in Section 5.2.

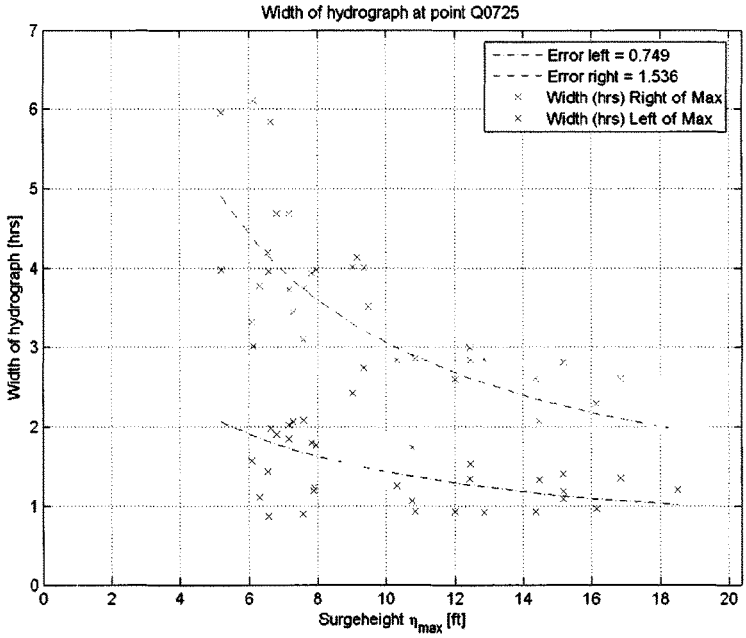


Figure 5.5 - Width of the hydrograph plotted against the maximum surge level
Note: Crosses indicate the different storms; the lines are the fit through the data points.

Apart from the surge levels, the temporal variation in the wave characteristics may also play a role in the overtopping rates. A similar approach as described above could be used for the wave characteristics as well. It appears, however, from the wave data that the variation in wave height around the peak surge is not considerable. In our approach, we have therefore used the maximum wave height for the entire surge hydrograph. A sensitivity analysis has been executed and the impact on the total overtopping volume appears to be small.

5.2 Overtopping volumes

The overtopping rates have been computed using empirical equations. In contrast with the design approach in Chapter 4, the surge level may be (far) above the crest level for some cases. For instance, a 100-year level of risk reduction in combination with a 1000-year event can easily give

surge levels higher than the crest level. Therefore, two contributions are taken into account for the overtopping rate computation: wave overtopping and free flow overtopping.

To compute the overtopping rates a distinction has been made between two cases:

- Surge level *below* the crest level : only wave overtopping
- Surge level *above* the crest level : wave overtopping and free flow

For the situation with wave overtopping only, the empirical equations from Van der Meer have been applied (see Chapter 4). If the surge level is above the crest level, both free flow and wave overtopping are taken into account (TAW, 2003):

$$q_{tot} = m(\eta - z_{crest})^{3/2} + 0.13\sqrt{gH_s^3} \qquad (5.1)$$

with:
q_{tot} : total overtopping rate (cft/s per ft)
m : weir coefficient $\left(= \frac{2}{3}\sqrt{\frac{2}{3}g} \approx 3.1 ft^{0.5}/s \right)$
g : gravitational acceleration (= 32.2 ft/s^2)
z_{crest} : crest level [ft]
η : water level [ft]
H_s : significant wave height [ft]

The first contribution in Eq. 5.1 is due to free flow, the second part is due to wave overtopping.

The overtopping rates have been computed using a Monte Carlo Simulation to account for the various uncertainties. The uncertainty in hydrograph width is initially considered, followed by the uncertainties in wave height, wave period and the coefficients of the overtopping formulation. The following procedure is followed:

a) Set the confidence level of the overtopping rate (in this case: 10%, 50% or 90%)
b) Compute the width of the hydrograph associated with this probability from step a) using the expected values and the standard deviation (assuming a normal distribution).
c) Draw a random number between 0 and 1 to set the exceedance probability p.
d) Compute the maximum water level from a normal distribution using the expected value 1% surge level and standard deviation as parameters and with an exceedance probability p.
e) Generate a hydrograph with this maximum water level and the given width of the hydrograph (10%, 50% or 90%)
f) Compute the wave height and wave period from a normal distribution using the expected value 1% wave height and 1% wave period and the associated standard deviations and with an exceedance probability p.
g) Repeat step c) and d) for the three overtopping coefficients in the overtopping formula, independently, using estimates of variability (standard deviation) in each coefficient.
h) Compute the overtopping rate for these hydraulic parameters and overtopping coefficients

i) Repeat the steps c) through h) a large number of times (N = 5,000)
j) Select the overtopping rate from the results at i) with the confidence level at step a)

The procedure is implemented in MATLAB to automate this procedure.

The above described approach results in overtopping rates with a 10%, 50% and 90% confidence. As an example, **Figure 5.6** presents the overtopping rates as a function of time for one levee design with a 100-year level of risk reduction. Every plot has a unique label at the top:

- DSX: Design standard with X-year return period (in this case 100-year)
- RPX: Hydraulic boundary condition with X-year return period (in this case 100/400/1000/2000-year)
- BSX: Base situation at location X (in this case 0001)
- BS : Evaluated situation (BS = Base situation, EA = East A, EB = East B, etc)

So, each plot represents a different return period for the hydraulic boundary conditions, viz. 100-year, 400-year, 1000-year and 2000-year. Furthermore, each plot gives the 10%, 50% and 90% overtopping rates in different colors.

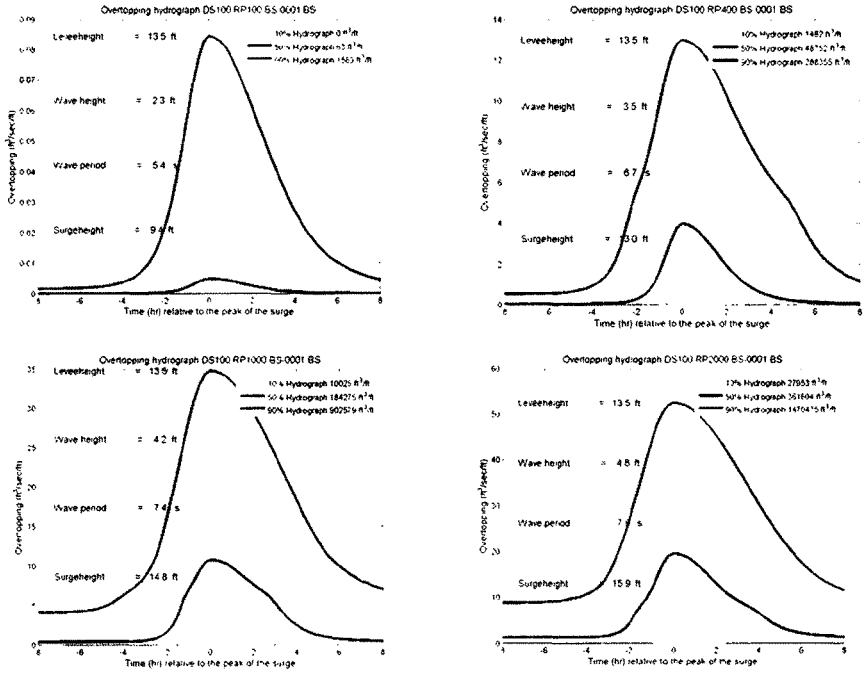


Figure 5.6 - Overtopping rates for different return periods (100-year, 400-year, 1000-year and 2000-year) at a level with a 1% design elevation

A few remarks are made regarding **Figure 5.6**:

- The maximum overtopping rate for the 100-year hydraulic situation equals about 0.1 cft/s per ft because this was the design criterion of the levee section. For the higher return periods the overtopping rates increase with several orders of magnitude.
- The 1000-year and 2000-year return period give free flow over the levee because the maximum surge level is higher than the levee crest.
- The form of the overtopping curve is not symmetrical but resembles the relatively steep front of the surge.

In total, approximately 6,000 overtopping hydrographs have been produced with the automated script for the LA-East alternatives. This number consists of 7 (Louisiana East alternatives) x 35 (planning subunits) x 2 (two levee sections per subunit on average) x 4 (hydraulic return periods) x 3 (design levels). These overtopping volumes have been used as an input into the interior drainage modeling which is discussed in Chapter 6.

6 INTERIOR DRAINAGE

This chapter discusses the process of converting overtopping into interior drainage areas into stage frequency relationships (**Figure 6.1**). First the internal drainage areas are defined (section 6.1). Next, the basic methodology for considering the interior drainage process is explained (section 6.2). Sections 6.3 to 6.5 describe the dominant processes that occur during a hurricane event which affect internal flooding: rainfall, overtopping and pumping. The interior stage frequency curves developed are used as input into the economic analysis.

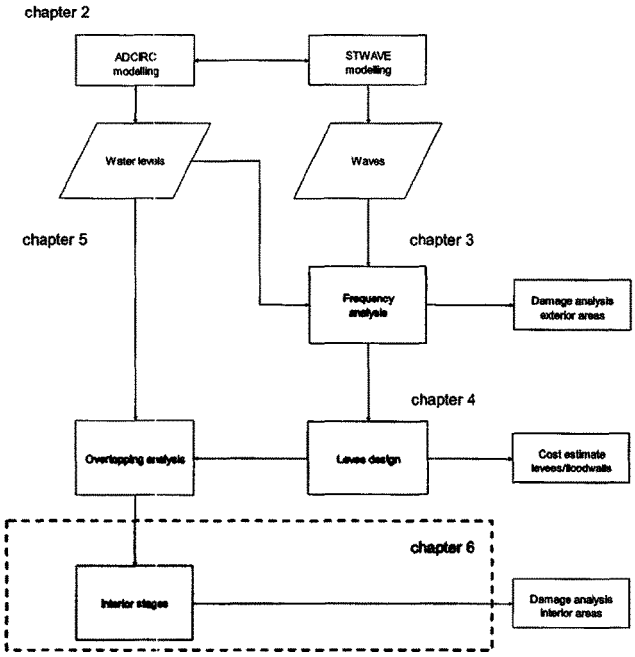


Figure 6.1 - Flow diagram of hydraulic analysis in LACPR framework (Chapter 6)

6.1 Storage areas

Within each planning unit the area within the authorized levee systems or for which levees are being planned have been sub-divided into smaller areas, called internal planning subunits. Most of the existing internal planning subunits are located in the vicinity of the city of New Orleans. New planning subunits have been developed for areas such as the north shore of Lake Pontchartrain where there currently is no levee but in one of the alternatives a levee is planned (termed semi-

internal planning subunits). Within the metropolitan areas of New Orleans the internal planning subunits have been defined either by parish boundaries or other defined features (such as raised roads or existing internal levees). For evaluation purposes the planning subunits in Planning Unit 3a, 3b and 4 are grouped into interior drainage areas. See Volume II for maps and further discussion of planning subunits.

6.2 Methodology

Each internal or semi-internal planning subunit has been schematized as a box model for which a stage-storage curve has been established. This information has been extracted from existing rainfall-runoff models or from LIDAR data for these areas. During a hurricane event the water balance is dominated by rainfall, wave or surge water overtopping and pumping (see **Figure 6.2**). The interior stage frequency has been based on the sum of the overtopping volume together with rainfall in the subunit. The effect of pumping in reducing flood volume has been taken into account if applicable. Where economic sub-basins join, a flow of water has been allowed to occur between areas above defined thresholds.

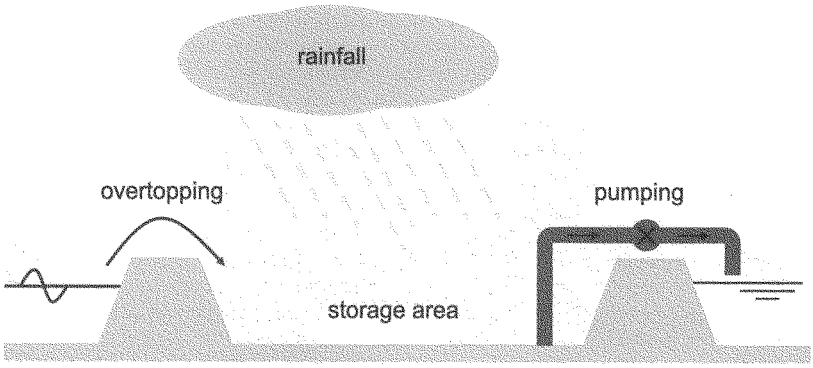


Figure 6.2 - Principle water system

The rainfall used in the evaluation was the 10-year rainfall and the development of the rainfall hydrograph is described further in Section 6.3.

For each of the overtopping edges of an internal planning subunit, overtopping hydrographs were established based on the levee design height or the current authorized levee heights, whichever was the higher. These hydrographs are described in more detail in Section 6.4.

An example of an internal planning subunit development is shown in **Figure 6.3** below. This shows the extent of an area, the stage storage relationship (in acre-ft) extracted or developed, and the overtopping lengths of the levees adjacent to the internal planning subunit.

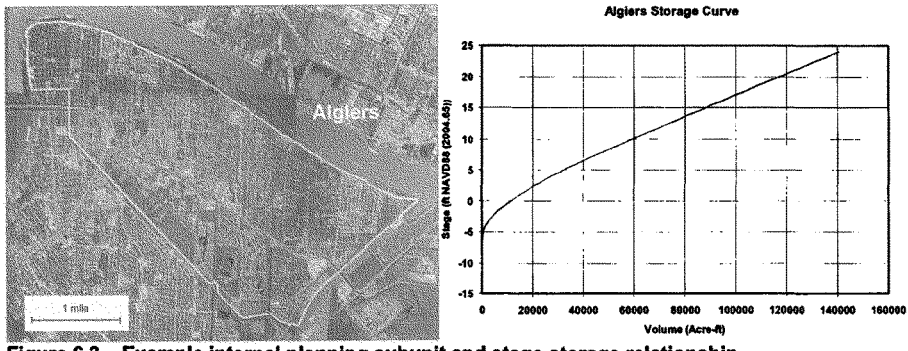


Figure 6.3 – Example internal planning subunit and stage storage relationship

6.3 Rainfall

The LACPR technical evaluation concentrates on the development of flood risk reduction systems (wetlands, levees) that protect against a range of hurricane surge events. Rainfall, however, also contributes to the interior flooding. Although this phenomenon is not the primary focus of this effort, rainfall has been taken into account. Based on earlier work, it appears that the heaviest rainfall have been from storms of less than hurricane intensity (Shoner and Molansky, 1956). In other words, it is not likely that an extreme hurricane event (100-year event, 400-year event, etc.) coincides with a rare rainfall event.

In order to evaluate the large number of LACPR alternatives on a comparable basis, a constant rainfall event was applied across all storm surge events (100-year, 400-year, etc.), confidence bands (10%, 50% and 90%) and for all planning units. Interior drainage is in essence fixed so that interior responses to overtopping over the flood risk reduction system can directly be compared from one plan to another. The rainfall event values were obtained for a range of storm durations from TP-40 documentation. These data were used because the hydrologic work done for the South East Louisiana Urban Flood Control Project (SELA) applied these data.

The basic assumption in the populated areas of New Orleans is that pumping can cope with 1" of rainfall in the first hour, and 0.5" in subsequent hours. Using this assumption, the various 10-year rainfall events (3-hour, 6-hour, 12-hour, 24-hour) were evaluated and the 6 hour duration storm was shown to give the highest rainfall rate over pumping. Since simple stage storage curves do not account for lag times, routing, etc (it is instantaneous), the shorter duration 6 hour 10-yr frequency rainfall intensity was used so that the simple stage storage curve would more closely replicate flooding as predicted with the unsteady flow HEC-RAS analysis performed by IPET.

The total rainfall is 6.5" for a 10-year rainfall event of 6 hours according to TP40 documentation. The rainfall hydrograph was calculated as a sinusoidal distribution over a six hourly period, and values were obtained in steps of 5 minutes. Note that in reality, the temporal development of rainfall events can be quite different from a sinusoidal shape. **Figure 6.4** shows the standardized

rainfall hydrograph resulting from this 10 year return period rainfall event. The rates are given in cft/s per square foot and a sinusoidal curve has been assumed. No routing of rainfall has been considered within the volume balance model. All rainfall collecting within a 5 minute time step is assumed to be available for pumping at the same time.

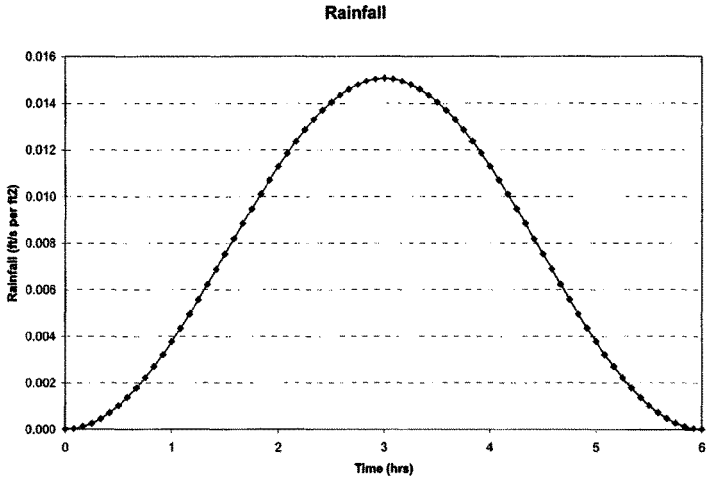


Figure 6.4 - Standardized rainfall hydrograph

6.4 Overtopping

The levee overtopping rates (10%, 50% and 90% confidence) were computed using the methods described in Chapter 5 for a number of different design standards (100, 400, and 1000-year) and the range of return periods 100, 400, 1000 and 2000 years. Examples for the 100-year design standard and 100- and 400-year storm events for the New Orleans East internal planning subunit are given in **Figure 6.5**. These show both the rainfall hydrograph and the levee overtopping hydrographs for three different defense lengths. In the same way as with the rainfall no allowance has been made for flood routing between the levees and the pumps.

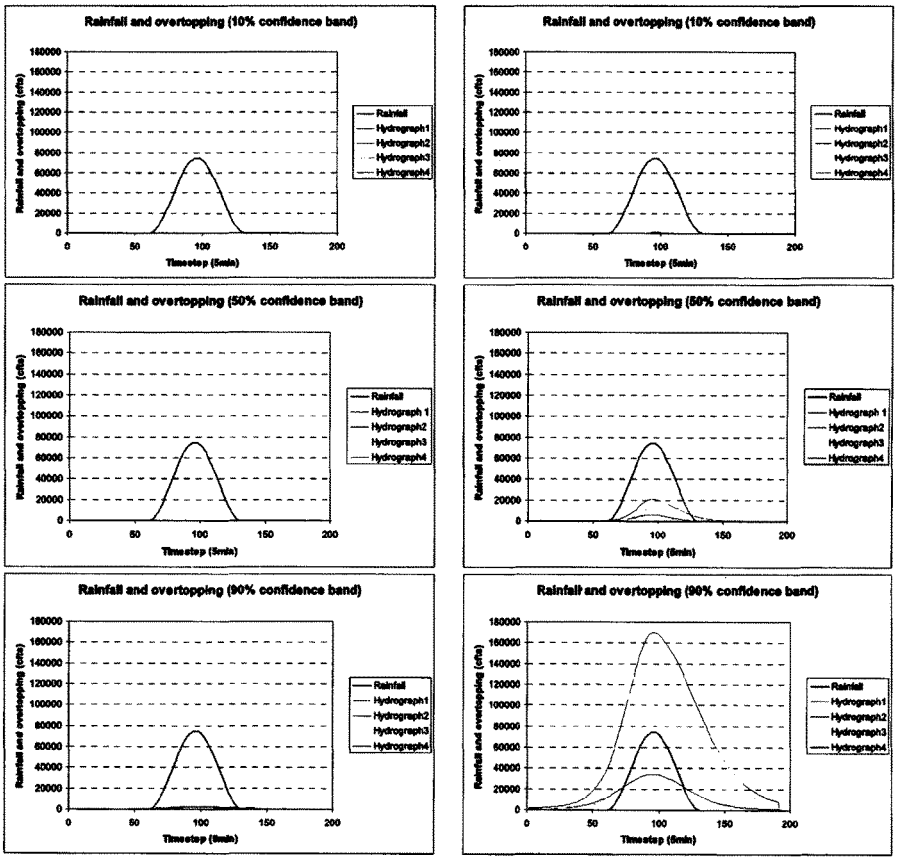


Figure 6.5 - Rainfall hydrograph of New Orleans East (100-year design standard) for a 100-year return period (left) and a 400-year return period (right)

6.5 Pumping

Pumping for each drainage area has been considered as a fixed rate of outflow. The pumping rates were obtained from the Corps for those locations where pumps were thought to exist. The values ranged from around 20,000 cft/s in East Jefferson to around 800 cft/s in St Charles Norco. Once the volume of pumping is exceeded by the inflow into the area in any 5 minute time step then flooding occurs.

New pump capacities were not estimated because of time limitations and the complexity of analyzing a very complex interior drainage system. For instance, the New Orleans area is

composed of numerous interior pumping stations that are fed by a complex system of canals and subsurface drainage systems. The primary stations along the outfall canals are operated in conjunction with the other stations. In order to increase the capacities of the outfall canal pumping stations and make sure that the added capacity is usable, one would have to redesign the complete interior delivery system to insure that flow could reach the outfall canal pump station.

6.6 Flood volumes and stage frequencies

The rate of flooding in each time step is considered by comparing the rate to the pumping rate and then if the difference is positive, recording the difference. These positive rates are then summated and multiplied by 300 to convert per second rates to totals over 5 minutes and then divided by 435,000 to convert from cubic feet to acre-feet. This gives a total volume of flooding for this condition. This is repeated for each confidence band and each design standard.

The flood stage in each internal planning subunit is established by interpolating the total flood volume into the stage storage relationship (as shown in section 6.2). The stage for each return period and design event is then compared with the levee height and the event surge height and the higher stages are capped at the higher of the levee height or surge elevation.

For those interior planning subunits which are internally connected the total flood volumes are used within separate calculations to consider the volumes flowing between adjacent storage areas and then to see whether the combined system fills above the levels of the divides or to the top level of the levees (see **Figure 6.66**).

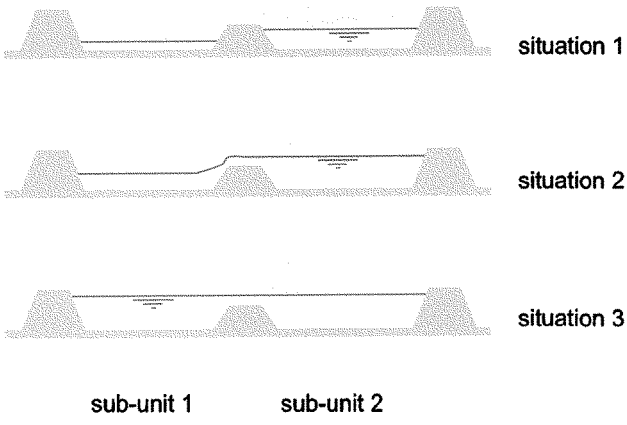


Figure 6.6 - Water volumes flowing between adjacent subunits

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Annex A
Sea level rise

Author: Kevin Knuuti, ERDC-CHL

Variations and trends in the relationship between local mean sea level (LMSL) and land elevations are important considerations in the planning and design of structures in areas that are currently tidally influenced or that could become tidally influenced in the future. In areas where the LMSL is rising relative to land elevation, the relative sea-level rise (RSLR) is often divided into a global increase in water mass (eustatic rise), a rise in local water level due to density changes in the water (steric rise), and a drop in local land elevation (subsidence). Throughout the 20th century, the global average SLR due to eustatic and steric effects has been approximately 1.8 mm/year (Meehl, 2007). Examination of tide gauges on geologically stable platforms in the northern Gulf of Mexico indicates a regional average SLR of approximately 1.8-2.0 mm/year during that same time period. Throughout coastal Louisiana, rates of subsidence exceed the rate of SLR by varying amounts. The resulting rates of RSLR throughout coastal Louisiana are significantly higher than the global average and regional average SLR rates.

Though the causes of climate change and future projections of climate change are somewhat controversial, it is well accepted that RSL has been rising across coastal Louisiana and will continue to do so in the future. Because of the difficulty associated with quantifying the rates of SLR that will occur in different areas of Louisiana, a sensitivity analysis is performed to determine how different project designs would respond to a range of SLR rates. For this sensitivity analysis, an extrapolation of the historic rates of RSLR is used as the low level for future sea-level rise and accelerated rates of rise based on National Research Council (NRC, 1987) and Intergovernmental Panel on Climate Change(IPCC) (Meehl, 2007) projections are used for higher rates of rise. Historic rates of RSLR vary across Louisiana and also vary depending on the methods used to estimate those rates. The two most commonly cited methods of estimating historic RSLR rates in Louisiana are radiometric dating of organic deposits (mostly peat) and analysis of long-term tide-gauge data. Because the RSLR rates determined from tide gauge data are based on more recent (20th century) data and because these rates are generally greater than the rates determined from radiometric dating, tide gauge RSLR rates are used for the low rate of RSLR in the sensitivity analysis.

Both the National Ocean Service and the U.S. Army Corps of Engineers have maintained long-term water-level gauges that can be used to calculate historic RSLR rates across coastal Louisiana. Because of the distance between these gauges, and the engineering difficulty associated with using numerous historic RSLR rates for analysis, coastal Louisiana was divided into different geomorphic regions for RSLR analysis. Within each geomorphic region, subsidence rates were thought to be relatively uniform due to relatively homogeneous geologic conditions. The geomorphic regions considered were based on the historic shifting of the Mississippi River's main stem and the associated delta lobes the river created, as shown in figure A.1 and as described by Penland (1990). Based on similarities in historic RSLR rates, alternative screening further grouped the regions into three primary geomorphic regions: the Chenier Plain (region 1 in figure A.1), the Delta Plain (regions 2-6 in figure A.1), and the Pontchartrain Basin (region 7 in figure A.1).

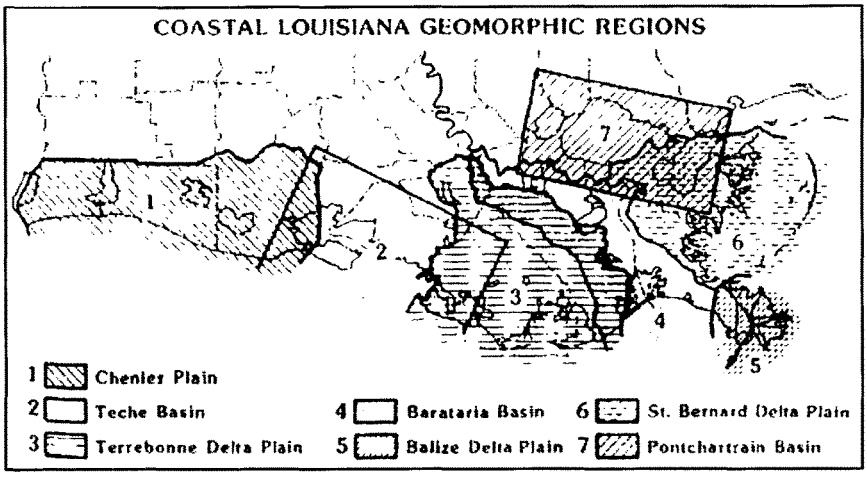

Figure A.1, from Penland, 1990

Future rates of RSLR were determined by considering both the 1987 NRC global mean SLR projections and the 2007 IPCC global mean SLR projections, along with estimates for local and regional subsidence rates across coastal Louisiana. While the 2007 IPCC projections are considered the most current and rigorous effort to estimate future global mean SLR rates there has been some criticism that these projections do not adequately consider the potential for extreme scenarios such as massive ice loss and melting from Antarctica. The 2007 IPCC mean central value estimate for global mean SLR by 2100 is 0.343 meters and the upper limit value is 0.59 meters. Due to the uncertainties associated with the IPCC estimate methods, a conservative value of 0.5 meters of rise by 2100 is used for rigorous sensitivity analysis, with the acceleration rate being the same as that described in the 1987 NRC report and modified by Knuuti (2002):

$$E(t_2) - E(t_1) = 0.0012(t_2 - t_1) + b(t_2^2 - t_1^2) \qquad \text{eqn A.1}$$

(see Knuuti, 2002, for description of variables and derivation of equation).

To account for possible extreme scenarios of global mean SLR and the associated RSLR across Louisiana, the sensitivity analysis also considered the "Curve III" value from the 1987 NRC report, which estimates a global mean SLR of 1.5 meters by 2100.

Estimates of local and regional subsidence rates were calculated by subtracting the regional historic SLR rate (2.0 mm/year) from the local and regional RSLR rates described earlier. These subsidence rates were combined with the future projections described in the previous two paragraphs to determine local and regional projections for future rates of RSLR. Table A.1 summarizes the RSLR values developed for the sensitivity analysis scenarios.

Table A.1: Relative Sea-Level Rise Values, 50-year project life

	Relative Sea-Level Rise Values between 2010 and 2060 (meters)		
Basis for value	Chenier Plain	Delta Plain	Pontchartrain Basin
Historic rate	0.2	0.4	0.2
Future scenario 1	0.4	0.6	0.4
Future scenario 2	0.8	1.0	0.8

References:

National Research Council (NRC), 1987. *Responding to Changes in Sea Level: Engineering Implications.*

Corps of Engineers policy, as described in ER 1105-2-100 (dated 22 April 2000),

Knuuti, K. (2002). "Planning for Sea-Level Rise: U.S. Army Corps of Engineers Policy" in Solutions to Coastal Disasters '02. ASCE, Alexandria, VA.

Meehl, G. (Intergovernmental Panel on Climate Change), 2007. *The Scientific Basis.*

Annex B
The maximum possible intensity and its use for coastal hazard estimation

The Maximum Possible Intensity and Its Use for Coastal Hazard Estimation

Don Resio

The Maximum Possible Intensity (MPI) of a hurricane has been recognized as a parameter that critically affects the probabilities of extreme tropical cyclone intensities at least since the late 1970's) (see for example: World Meteorological Organization, 1976 and Mooley, 1980). Even before that time, theoreticians had recognized the existence of thermodynamic and dynamic constraints on the energy available for tropical cyclone intensification, even when it is unencumbered by the proximity of land (see for example: Riehl, 1954; Miller, 1958; and Malkus and Riehl, 1960). More recently, Emanuel (1986, 1991) and Holland (1997) formulated theoretical models for estimating maximum tropical cyclone intensity. In an evaluation of the performance of these two MPI models, Tonkin *et al.* (2000) examined storms within 1) the Australian/southwest Pacific region, 2) the northwest Pacific region, and 3) the North Atlantic region. Since our primary interest is focused on the Gulf of Mexico, we will limit our discussion here to results for that region.

Figure 1 shows the geographic area encompassed within the "North Atlantic region" as defined by Tonkin *et al.* Figure 2 presents the results from Tonkin *et al.*'s application of the Emanuel Model (black dots joined by a solid line), Holland's model (dashed line), and observed intensities (open triangles joined by a solid line). This application used a climatological mean Sea Surface Temperature (SST) defined over the period 1950-1979. According to Tonkin *et al.*, Evans (1993) results suggest that there is little gain in predictive skill when actual monthly SST values are used in place of the climatological mean.

As can be seen in Figure 2 and as widely recognized from theoretical considerations, a strong relationship exists between climatological SST values and the lowest central pressures. We see that, in the range of SST values from 26° to 28° (C), the minimum central pressures of the Holland Model, the Emanuel model and the observed intensities are all in approximate agreement. Above 28° (C) the observations continue to show decreasing central pressures with increasing values of SST; whereas, the Emanuel and Holland models do not.

Figure 3, taken from Schade (2000), shows another approximation for the MPI. In this paper, Schade suggests that the effect of the SST field on tropical cyclone intensity is twofold. First, the large-scale ambient SST field "sets the stage for the tropical cyclone." Second, the intensity of a tropical cyclone is highly sensitive to the reduction of the SST in the interior region of the storm due to the response of the ocean to surface winds. Thus, whereas the concept of the MPI is well founded, some of its details are still under development.

Figure 4 shows the average August-September SST for the Gulf of Mexico for the period 1940-2006. As can be seen here, the highest average values during this part of the

year (the peak of hurricane season) have varied from as low as 28.17° C in 1984 to as high as 29.49° C in 1962. The dotted vertical line in Figure 3 shows this historical maximum plotted on top of Schade's results. The heavy solid line along the top of that Figure denotes the MPI value without consideration of any negative feedback of the type discussed by Schade; thus, it is expected to represent a maximum possible threshold for the MPI. From Figures 2, 3, and 4, we can deduce that a value of 880mb represents a very sensible (perhaps slightly conservative) value for the MPI in the Gulf of Mexico.

Once the value of the MPI is established, we can construct a set of storms that represents the envelope of worst conditions for different size storms. Since the present state of the art does not indicate a strong dependence between storm size and the value of the MPI for a given ambient SST field, we will assume here that the MPI is a fixed value, independent of storm size. If we select the storm track which produces the maximum surge values for specified size and intensity, we can make the somewhat conservative assumption that integrates all storms into that class of storm track. In this context, the probability of a storm can be estimated simply from the joint probability of size and intensity along the MPI line, i.e.

$$P(storm) = P(C_p = 880mb) \times P(R_p \mid C_p = 880mb)$$

If we select values of R_p ranging from small to large, we can estimate the maximum possible surges for any coastal site. This is an important improvement over concepts which attempt to relate storm surge maxima to the Saffir-Simpson scale, which considers only storm intensity.

Figure 5 shows the preliminary results of some runs with a radius to maximum winds of 25 nautical miles along three tracks. For the New Orleans area, the cumulative distribution function (CDF) for hurricane intensity (peripheral pressure minus central pressure) is given by

$$p(c_p \le 880mb \text{ and } R_p \ge 25nm) = \Lambda_1 \cdot \Lambda_2$$

$$\Lambda_1 = F[a_0, a_1] = \exp\left\{-\exp-\left[\frac{\Delta P - a_0}{a_1}\right]\right\} \text{ (Gumbel Distribution)}$$

$$\Lambda_2 = p(R_p \mid c_p) = \frac{1}{\sigma(\Delta P)\sqrt{2\pi}} e^{\frac{(\bar{R}_p(\Delta P) - R_p)^2}{2\sigma^2(\Delta P)}} \text{ (Conditional Normal Distribution)}$$

where

c_p is the central pressure

R_p is the scale function for the radius to maximum wind speed

$F(\square)$ is the CDF for the argument $\square$

ΔP is the pressure differential = $(p_0 - c_p)$ where p_0 is the peripheral pressure

$\bar{R}_p$ is the conditional mean value of R_p - given ΔP

σ is the conditional standard deviation of R_p - given ΔP

Using values for the Gumbel coefficients which are identical to those described in Resio *et al.* (2007) and using values for the conditional mean and conditional standard deviations capped to be no smaller than the value at 900 mb, the estimated return period for a storm with a central pressure of 880 is 2905 years. The combination of this central pressure with a size of 25 nm or larger is expected only once every 74,848 years.

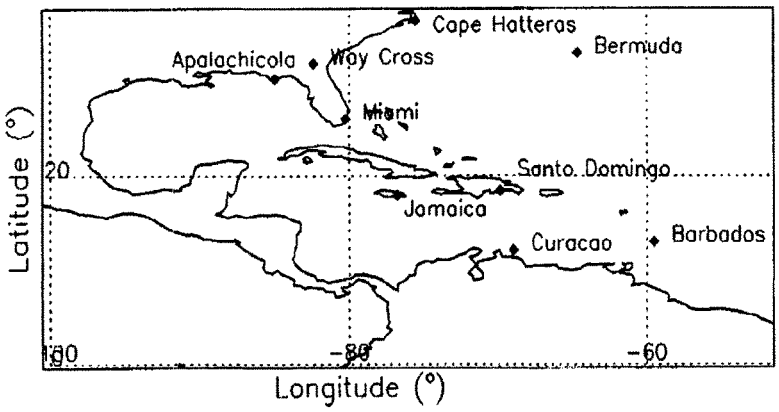

Figure 1. Geographic area considered under heading of "North Atlantic" by Tonkin *et al.* (2000).

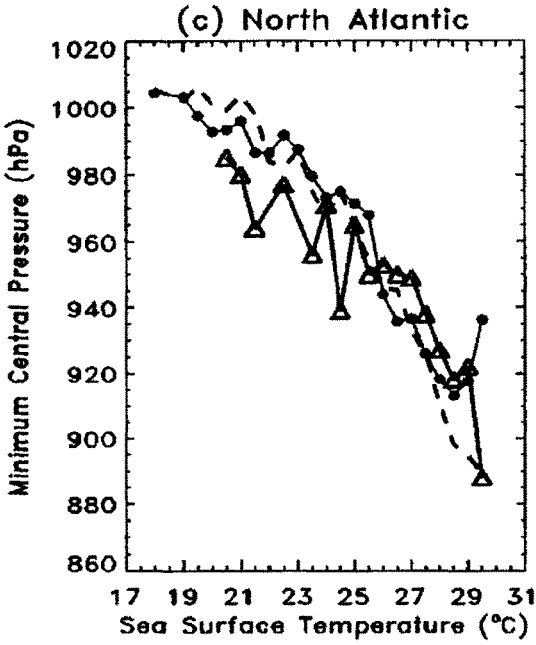

Figure 2. Relationship between observed minimum central pressures (maximum intensities) and sea surface temperature in the North Atlantic basin (from: Tonkin et al., 2000)

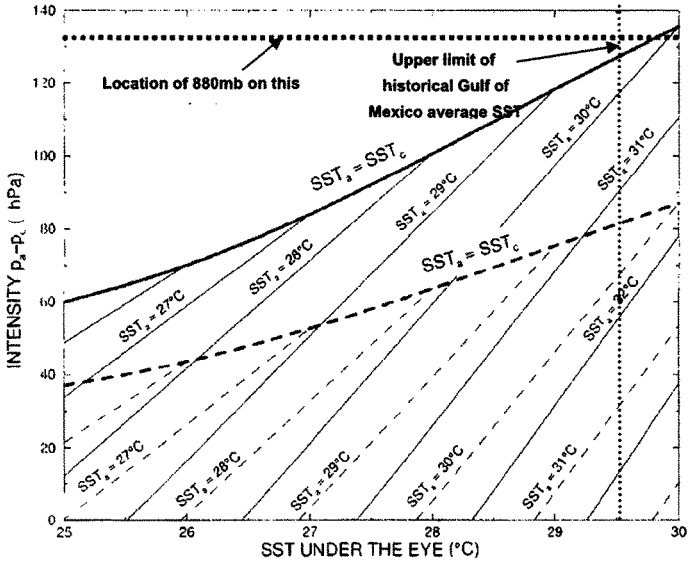

Figure 3. Estimated cyclone intensity as a function of the SST under the eye of the storm (from: Schade, 2000). The solid and the dashed lines correspond to ambient relative humidities of 75% and 85%, respectively. The heavy lines mark the maximum possible intensity that is realized neglecting (negative) SST feedback. The thin lines connect points with the same ambient SST.

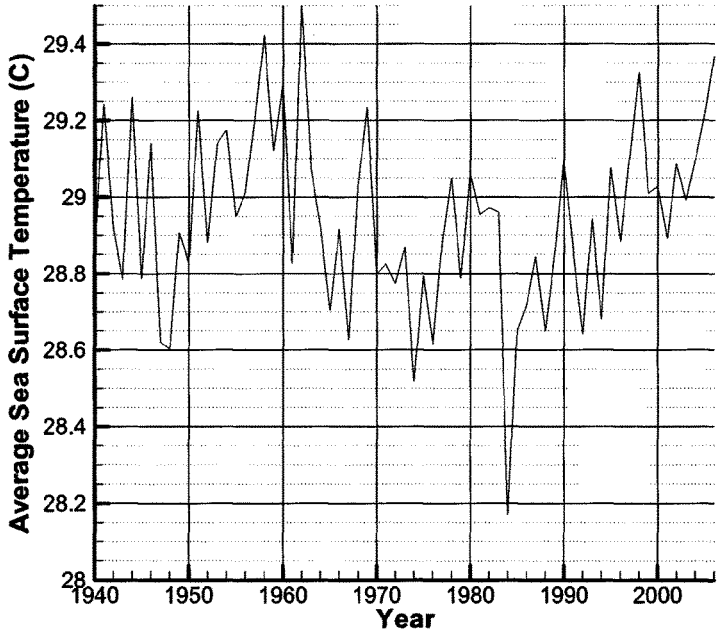

Figure 4. Variation of average annual (unsmoothed) Gulf of Mexico SST's from 1940 through 2006.

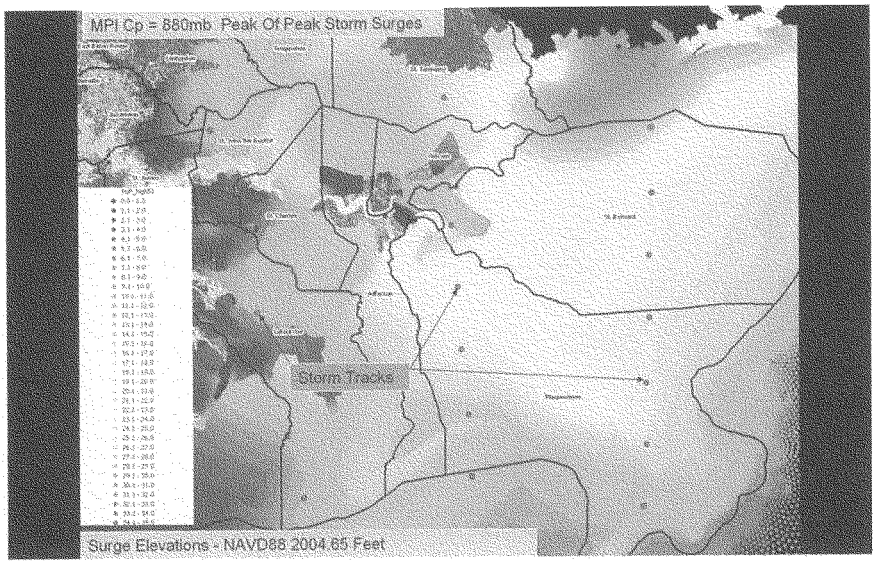

Figure 5. Preliminary results showing contours of the maximum elevation of maximum surges for three tracks (paths shown by sequences of red dots) for a storm with central pressure equal to 880 mb and a radius to maximum wind speed of 25 nautical miles. In these storms, the Holland B term and the pressure and size variations during approach to the coast were treated in the same fashion as the rest of the storms described in the White Paper by Resio et al. (2007).

Hydraulics and Hydrology Appendix
Volume II – Results

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ANNEX B Fact Sheets for Sub Basins

1 INTRODUCTION

Volume II of this appendix presents the results of the hydraulic evaluation of the LACPR alternatives. The methodology that was developed for this evaluation is discussed in Volume I.

Volume II describes the results of the processes carried out for determination of the levee heights and stage frequency curves for the five LACPR planning units. In Chapter 2 the hydrodynamic modeling (surge and waves) is discussed, being the primary input for deriving the levee heights and stage frequencies. Chapter 3 describes the process to come up with stage frequency curves for the planning subunits. Chapters 4 to 8 describe the results for each planning unit, respectively. The report ends with conclusions and recommendations in Chapter 9.

Annex A provides detailed descriptions of preliminary hydrodynamic results used either in the analysis of LACPR alternatives or for sensitivity analyses. Annex B provides maps and information related to levee heights, pumping capacity, stage storage relationships, etc. for 58 individual storage areas, or sub-basins, across South Louisiana.

1.1 Scope and limitations

The work presented in Volume II has been carried out between June and April 2008 as a combined effort of the United States Army Corps of Engineers, New Orleans District, and Haskoning Inc. The methods and results as described in this report are limited to the technical aspects only and do not include the economic evaluation.

The main deliverables of the hydraulic evaluation of the LACPR work are the design levee heights and the stage frequency curves for both interior and exterior areas. These are meant as input for the economic evaluations and the damage studies.

Note that the results described in this volume were developed to enable the relative comparison of various design alternatives. More detailed studies will be needed before doing actual design.

1.2 LACPR alternatives

Within the five LACPR planning units, various *alternatives* (sets of measures) have been developed and evaluated. An alternative may be the construction of a particular alignment of levees, it may include structures or landscape changes which reduce surge elevations further offshore, it may include nonstructural measures, such as buyout or raising of property. Most often, alternatives are proposed in combination, like the construction of levees together with the raising of property.

Alternatives (proposed sets of levees in terms of alignment and height) are designed to provide risk reduction against flooding events of a certain magnitude. The *level of risk reduction* provided by an alternative indicates the frequency of the flooding to which the specific alternative is to provide risk reduction, indicated in "once per a certain number of years." For this report, three particular levels of risk reduction have been considered. These are 100-year, 400-year, and 1000-year. The selection of these levels of risk reduction has been based on the request to provide the 100-year level of risk reduction, plus risk reduction for a range of hurricanes up to "Category 5."

For the evaluation of the alternatives, the designed levee systems have been confronted with a standard set of flooding events. Here, an *event* is the representation of surge elevation, wave

height, and wave period that might be expected to occur at any location with a defined frequency. The range of return periods used is 10, 100, 400, 1000, and 2000 years. The middle 3 events match with the proposed levels of risk reduction, whilst the 10 gives a lower limit and the 2000 year provides an event which will overtop all levels of risk reduction.

In this way, the amount of overtopping can be computed for each alternative for a certain frequency event. The amount of overtopping results in a water level (stage) in the protected area. Thus, a planning alternative which is designed to provide a 400-year level of risk reduction should not overtop when it is confronted with a 100-year event, but it will overtop in case of a 1000-year event. Details on the LACPR alternatives are given in the main technical report and in the chapters per planning unit in this report.

1.3 Planning units and subunits

Each planning unit is divided into planning subunits. These subunits are aggregates of the census blocks used for economic data collection. Where these planning subunits fall within the authorized levee systems or where levees are being planned, these areas are the subject of calculations to determine flooding with the levees in place. **Figures 4.1, 5.1, 6.1, 7.1, and 8.1** illustrate the planning subunits within each of the five LACPR planning units.

Within the existing levee systems of the metropolitan areas of New Orleans and along the West Bank of the Mississippi (planning units 1 and 2), the planning subunits were established as larger groups of census blocks and which were defined either by parish boundaries or other physical features (such as raised roads or existing internal levees). In other areas the census blocks were grouped based on flooding similarity, but limited in size by parish boundaries, proposed levee alignments and physical features.

In the west (planning units 3a, 3b, and 4) the planning subunits are derived purely from the census blocks, which are combined to form the planning subunits. These units have been split along any proposed levee alignment so that groups of planning subunits (called drainage areas) can be used within the calculations.

1.4 Hydraulic evaluation

In the hydraulic evaluation, hydrodynamic results (storm surges and waves) are used to compute the levee heights and the stage frequency curves (see **Figure 1.1**). The stage frequency curves in their turn are an important input for the economic analysis to estimate the damage in the planning subunits. In addition, the levee heights are required so that the cost of construction and ongoing maintenance can be established.

Step 1: Determination of the levee heights
The levee height is computed for three different levels of risk reduction (100-year, 400-year, 1000-year). Given the level of risk reduction, the overtopping volumes are computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). The resulting overtopping volumes are used to compute the interior frequency curves in the planning subunits (step 2).

Step 2: Determination of the stage frequency curves
For each planning subunit the frequency curves are extracted from the hydraulic results estimated for different levee alignments and scenarios. A single planning subunit can have either an internal stage frequency, an external stage frequency, or in some cases both an internal and an external stage frequency for a particular hydraulic model condition. A planning subunit has both internal and external stage frequencies in those cases where no levee currently exists in the model, but where a levee is planned in one of the alternatives. The stage frequency curves are used in the economic evaluation to estimate the damages caused by flooding.

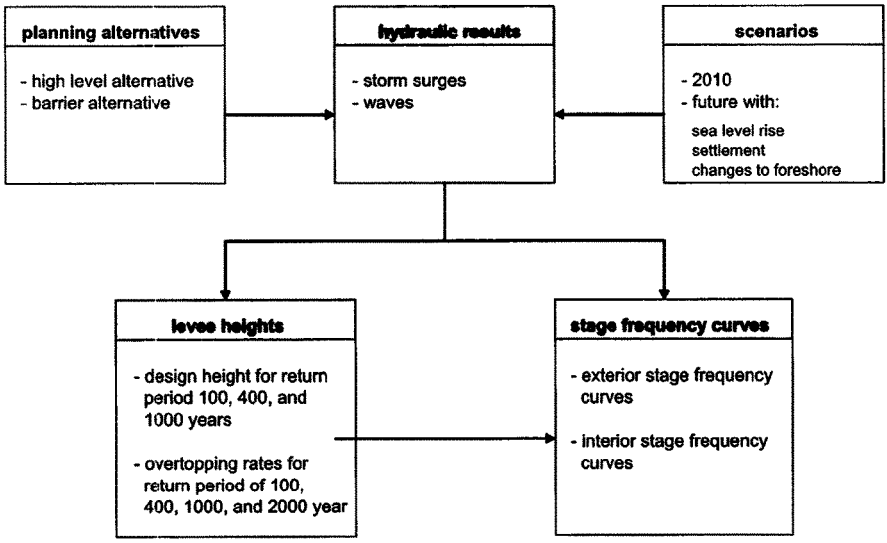

Figure 1.1 - From levee alignments and scenarios to frequency curves for plan evaluation

2 HYDRODYNAMIC RESULTS

2.1 Introduction

The purpose of the hydrodynamic modeling is to provide engineering based estimates on extreme surge and wave heights for the evaluation of both existing (base) and alternative future conditions to the levee design and stage frequency analysis required for risk assessment and economic evaluation of alternatives within the LACPR technical evaluation.

The JPM-OS statistical code, *Resio*, was developed for an optimal sampling of storms whose coverage would approximate statistical sampling of surge responses for a domain of about 3.5 degrees longitude of the central Gulf coast. Resio designed the JPM-OS statistical code to accept the maximum ADCIRC surge response at any specified point with-in the domain of coverage. The minimum optimal sampling of storms and tracks that provide the required sampling for the desired probability space combined to include 152 storms. The 152 storm sample produces reliable statistical responses for the coastal region under examination and when viewed as a statistical surface showed no crenulations along the response surface. Too few storms would manifest itself by producing crenulations along a given frequency water surface contour where a smooth response should be expected. Including more storms than the 152 storm suite would result in additional super computer time to run ADCIRC and not appreciably add to the resolution of the response surface. The probability space covered by the chosen suite of 152 storm included storm probabilities with surge responses ranging from about 50 year to about 3500 year in recurrence interval. Details about the JPM-OS can be found in the White Paper contained in the Reference Library attached to this Appendix.

Initially, two separate sets of 152 storms and tracks were used: one for the eastern part of southern Louisiana (primarily Planning Units 1 and 2); and a second set for the west (Planning Units 3a, 3b and 4). In particular Planning Unit 3a (the Morganza area) fell in the overlap area of both of the storm suites and ADCIRC model runs. When reviewing statistical results from the JPM-OS at the western edge of the eastern storm suite, and the eastern edge of the west storm suite, it was apparent that in the overlap areas, Planning Unit 3a produced inconsistent results when considered as separate suites. Rather that attempt a blending of probabilities in Planning Unit 3a, Resio modified the JPM-OS code so that it could analyze the maximum surge responses from all 304 storms from the two separate suites. With the 304 storm suite JPM-OS code, statistical results for Planning Unit 3a would include the effects of possible storms that could affect the area and the statistical surfaces generated with the 304 storm code provided reliable and smooth water surface contouring across all planning units. Statistical surfaces for the existing and other conditions can be found in the Evaluation Results Appendix.

The reader is referred to Volume I for a full description of the applied methodology, the underlying models and assumptions. An important issue regarding the wave results from STWAVE is that only the no friction results has been used ultimately in the framework of LACPR. The results from the independent analysis suggest that compared with the STWAVE results with friction, the no friction STWAVE results provide the more appropriate wave conditions for levee design (see also Annex A). Uncertainty in future location and density of coastal marshes, in part due to local

subsidence and the uncertainty of funding for marsh restoration, provides additional rationale for excluding the effects of friction in the near shore wave results.

In the post processing of the model results the maximum surge values were established by looking at each time step and developing the maximum value at each point within the model over time. This returns the maximum envelope of surge values for a particular storm event. In addition a peak of peaks surface was produced which took the highest value at each point from the range of storms computed. The ensemble of maximum values (from the suite of storms) was used to determine the expected return periods for the surge elevations. A similar process was undertaken from the STWAVE results for wave height and wave period.

The following sections give a brief overview of the hydraulic modeling results. Section 2.2 describes the 2007 base condition modeling. The chapter continues describe the models developed for the east and west part of the State, including changes to the marsh over 50 years. It briefly presents the processes undertaken to turn the model results into event data, before describing the selection of particular model grids for use in the further analysis. More detailed information about the modeling is provided in Annex A.

All figures presented in the following sections represent the maximum of maximum, MOM, ADCIRC response surfaces for all storms simulated, and for the east the results from the southeast STWAVE grid (which covers the east of New Orleans, the Mississippi Delta, Lake Borgne and the barrier islands) have been used to demonstrate the wave effects. It should be noted that the MOM plots are visually informative but care has to be exercised when screening for the maximum change in water level for a particular point since there is the potential for the maximum response to contain momentary instabilities in the ADCIRC run. However, it is apparent that the rather smooth change in surface contouring shown in these plots is unaffected by instabilities and that the MOM surfaces plots are indicative of maximum water levels produced by the 304 suite of storms, 152 in the east and 152 in the west.

2.2 2007 base condition

The 2007 base model has been built to cover the whole of southern Louisiana and represents the situation at the start of the 2007 hurricane season. It includes post Hurricane Katrina and Rita bathymetry together with levee heights and extents which match with the current state of repaired and upgraded levees. A full set of 152 storms was run for each of the east and west parts of the model to determine water levels and corresponding wave heights to use as base conditions for the comparison of future alternatives.

Figure 2.1 and **Figure 2.2** represent the maximum surge level recorded for the 152 storms simulated for each of the east and west grids for Southeastern Louisiana. These plots do not represent a single return period surge event, but are representations of the maximum recorded value at each location from all the 152 storms run through the models. The maximum results show significant surge elevations within the existing Hurricane Storm Damage Risk Reduction System in the east, but this would be expected as the range of storms considered include storms in excess of the 3,500 year return period.

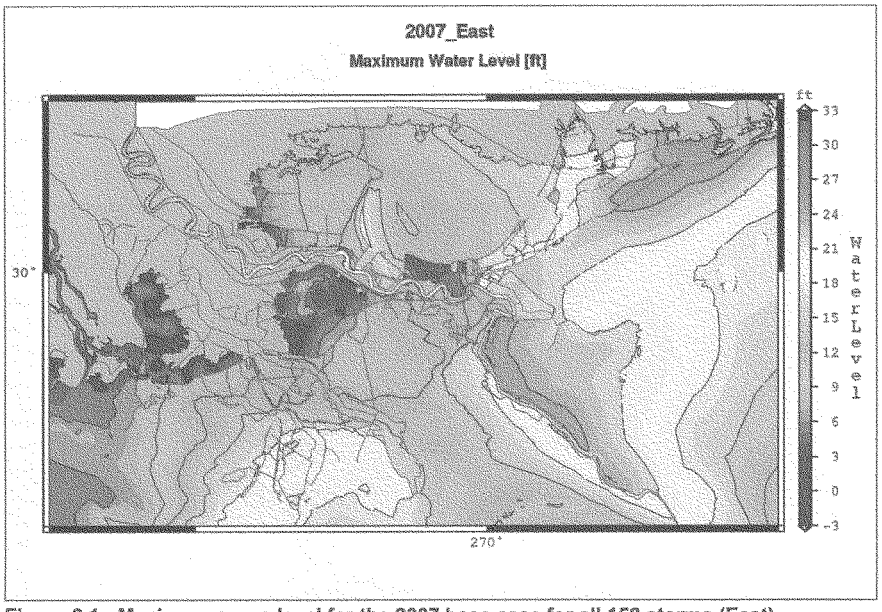

Figure 2.1 - Maximum surge level for the 2007 base case for all 152 storms (East)

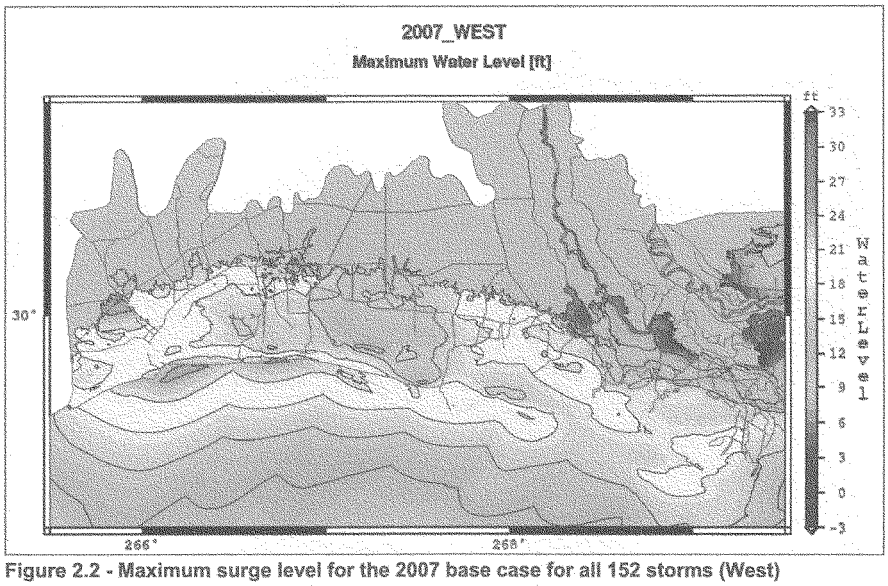

Figure 2.2 - Maximum surge level for the 2007 base case for all 152 storms (West)

Figure 2.3 and **Figure 2.4** show the maximum wave heights predicted from the range of storms run within the model for the southeastern STWAVE grid (with no friction) and the west STWAVE grid. These show the reduction in wave height as the water depth rapidly decreases at the barrier islands and in the east shows maximum heights around the existing levee system of 6 to 8 feet.

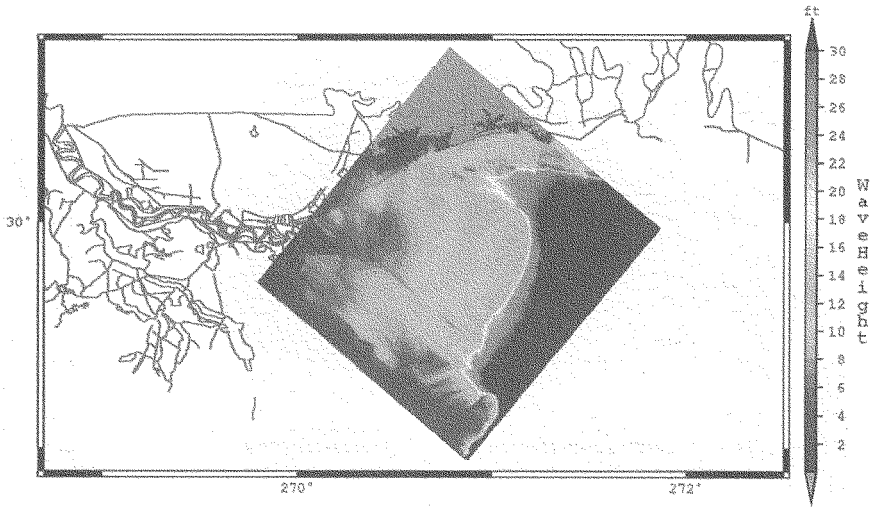

Figure 2.3 - Maximum wave height for the 2007 base case for all 152 storms (East STWAVE grid)

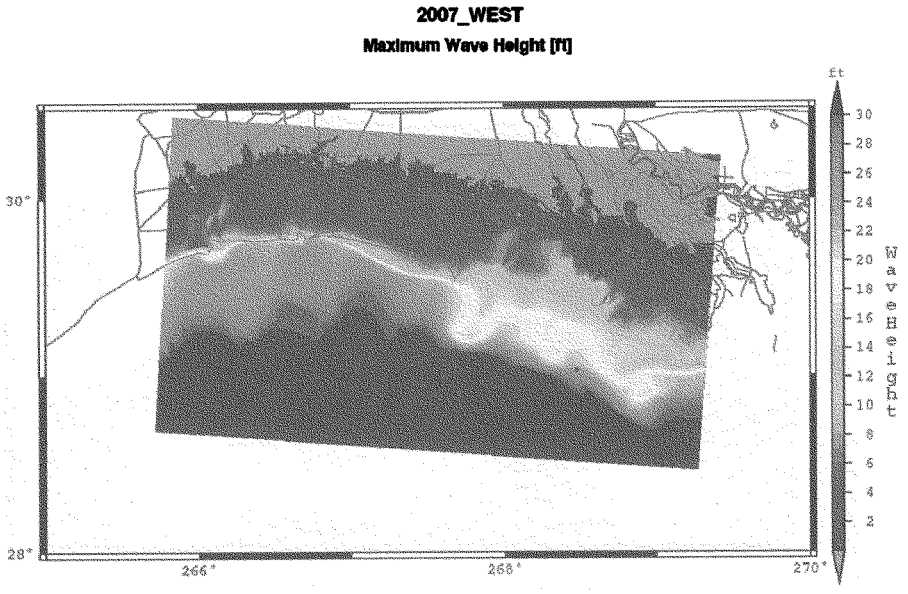

Figure 2.4 - Maximum wave height for the 2007 base case for all 152 storms (West STWAVE grid)

2.3 East models

2.3.1 2010 base condition

The 2010 base condition in the east represents the proposed improvements to the system that are expected to be completed by 2010. These include restoring the levees to their authorized levels and, in and around the metropolitan area of New Orleans, raising the levee heights to provide a 100-year level of risk reduction.

Initially, also included in the 2010 base run was the proposed levee for the Morganza to the Gulf feasibility study. Since the 2007 ADCIRC model runs had already addressed the base no levee condition for the Morganza area it was decided to include this proposed levee system to maximize design output from the ADCIRC runs. The Morganza levee is not expected to be completed by the 2010 date but was included in the ADCIRC runs to expedite reanalysis of the Morganza project which at the time was awaiting authorization for construction by the Congress. Around the Morganza area (to the west of Bayou Lafourche) the model included a non overtopping levee to represent the proposed new levee around the Morganza area. The Morganza project was considered to be sufficiently removed from the Planning Unit 1 and Planning Unit 2 basins so as to not influence surge responses in those basins. **Figure 2.5** below in fact shows that the proximity of the proposed Morganza levee has no influence on water levels in Planning Units 1 and 2.

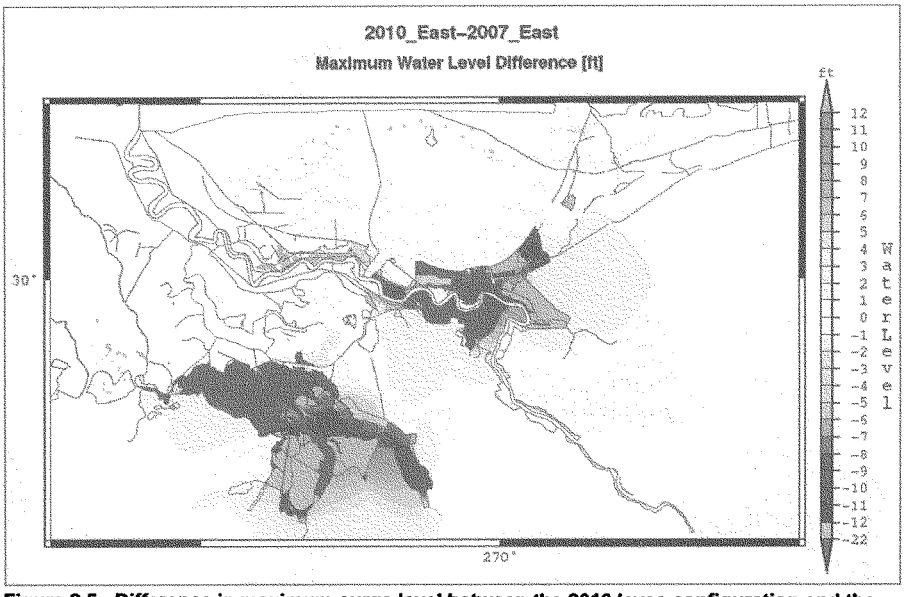

Figure 2.5 - Difference in maximum surge level between the 2010 levee configuration and the 2007 base grid for the 2010 storm suite

For this conclusion, the reader should note the large area of no change separating the two response surfaces shown in **Figure 2.5**. To avoid confusion when looking at the 2010 base in the final analysis for which results are presented in the Evaluation Results Appendix, the 2010 base condition was reanalyzed without the proposed Morganza Levee and all 152 storms in the east were run in ADCIRC. The 304 JPM-OS code was used to compute the frequency analysis for the 2010 base condition.

Figure 2.5 shows the differences in maximum water level between the existing (2007) model and the 2010 base condition. This shows the dramatic reduction in surge elevations within the levee systems where the improved levees reduce surge elevations. However it also indicates the increase in maximum surge levels outside the levee system (generally 1 to 3 feet, but with localized hot spots). The changes in maximum wave height are much less marked, other than within the levee system where the dramatic reduction in water depths reduces the wave heights to also nil. Plots of the wave height change are included in the Annex A.

2.3.2 Closure options of Lake Pontchartrain and the Barataria Basin

Four different model grids have been constructed representing four options for closing the mouth of Lake Pontchartrain (Planning Unit 1), together with three options for closing the Barataria Basin (Planning Unit 2). Note that two of the model grids used the same closure in the Barataria Basin. Of these four models, one was used for the main calculations whilst the remainders were used for screening. All four models are considered below, but more detail is given on the weir closure

option which was used for the main comparison of closure options to non-closure options. All the models used the 2010 base model as the starting point. More details of the model set ups are given in Volume I.

Full Closure along US90 and GIWW (Model grid EA)

For Lake Pontchartrain area this model represents the closure of Lake Pontchartrain along the US90 corridor with a non-overtopping levee. This option prevents any filling of Lake Pontchartrain from Lake Borgne, but it still shows elevated water levels within the Lake as a result of the high wind speeds blowing directly over the lake waters. It also includes moving the primary levee alignment to around the edge of Lake Borgne, closing the Gulf Intracoastal Waterway (GIWW) and the Mississippi River-Gulf Outlet (MRGO). Surge values generated in Lake Pontchartrain would represent the maximum reduction in water levels achievable in the lake since the non-overtopping levee blocks all inflow from Lake Borgne regardless of storm strength and surge heights generated in the Mississippi Sound and Lake Borgne area.

For the Barataria Basin the model includes a non-overtopping levee which follows the line of the GIWW from Belle Chase across to the northern end of the Larose to Golden Meadow levee system. It also includes non-overtopping levees around the Larose to Golden Meadow area.

The results of this model are discussed in the Discussion on the East model grids (see Section 2.6).

Weir Closure along US90 and GIWW (Model grid EB)

For the Lake Pontchartrain closure, this model is the same as model grid EA except that the levee across the mouth of Lake Pontchartrain has been lowered to 12 feet. This blocks the flow of water during small events and act as a weir in extreme events allowing a reduced amount of water into the Lake. **Figure 2.6** shows the difference in maximum water levels for all storms between the EB model and the 2007 base case. Reductions of the maximum surges within the Lake are in the order of 2 to 3 feet in the south and 3 to 4 feet in the north. The maximum values do increase on the outside of the weirs, particularly near the closure of the GIWW/MRGO, where the maximum levels go up by 5 to 6 feet. This rather large increase in water levels is due to the fact that the in the EB grid, levee heights are set to not overtop for the reach starting at the GIWW near South Point extending along Lake Borgne where it crosses the MRGO and continuing down around the Chalmette loop back to Carnarvon and south to it lower terminus at the east bank Mississippi River levee across from Belle Chase. Levee heights in the 2007 condition could overtop and hence maximum water levels are moderated and the differences between 2010 and 2007 are accentuated. The potential impact of this weir on the Mississippi coastline is discussed in Section 2.6.

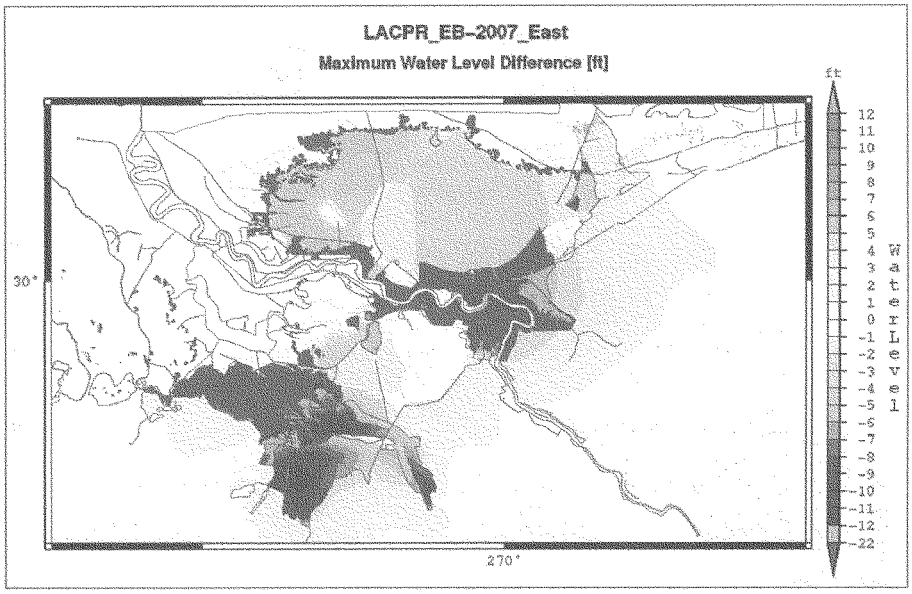

Figure 2.6 - Difference in maximum surge level between East B grid and the base grid for the East B storm suite

In the Barataria Basin area the EB model grid replaces the non-overtopping levee in model EA with a 12' weir. This weir reduces the maximum water levels in the areas north west of the weir further weir, but appears to have little impact on the maximums water levels north east of the weir along the West Bank Hurricane risk reduction levee. Offshore or on the unprotected side of the weir water levels increase by up to 4 feet adjacent to the weir.

Partial closure along US90 and Weir along US90 (Model grid EC)

The EC model uses a non overtopping levee similar to EA across the mouth of the Barataria Basin, but in addition allows for openings through the levees at the Chef Menteur Pass and the Rigolets. These opening allow water to flow between Lake Borgne and Lake Pontchartrain during storm events.

Within the Barataria Basin the EC model adopts a 12' weir along the alignment of US90, joining the levees around the Sunset Drainage District with a levee running along the east bank of Bayou Lafourche.

The results of this model are discussed in the Discussion on the East model grids (see Section 2.6).

Full Closure through Lake Borgne (Model grid ED)

This alternative considers a non-overtopping levee built across Lake Borgne, connecting the southeastern tip of the St Bernard Parish defenses with Slidell. This alternative results in noticeable increases in maximum water level on the outside of the levee and along the Mississippi coastline, but has a marked reduction in maximum levels within the Lake Pontchartrain and within the areas to the south and east of New Orleans east and St Bernard Parish.

Figure 2.7 shows the differences in maximum water level between the ED model grid and the 2007 base model.

The model in the Barataria Basin is the same as model grid with the full closure (East A) and is not considered further.

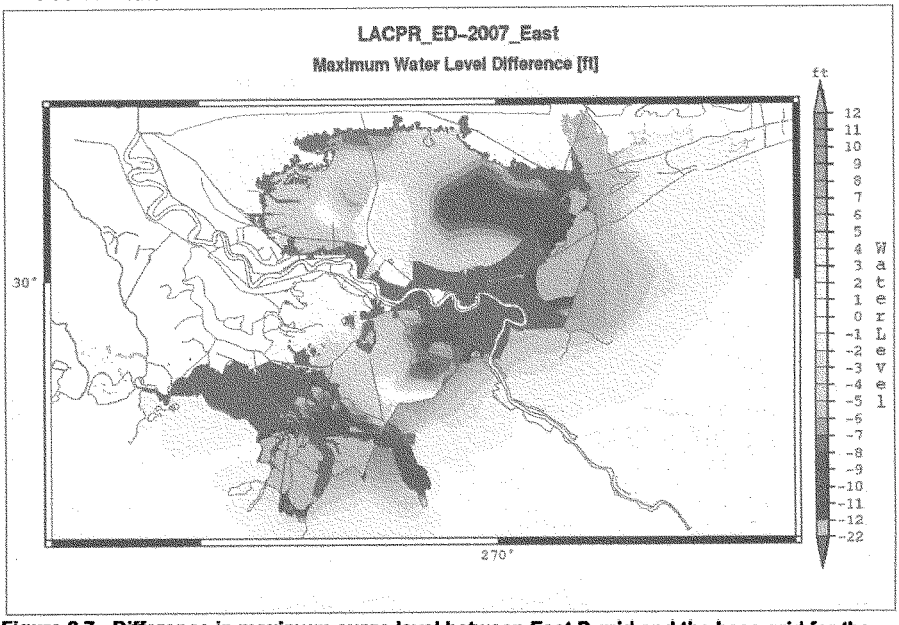

Figure 2.7 - Difference in maximum surge level between East D grid and the base grid for the East D storm suite

2.3.3 Other alternative models considered

In addition to the four major alignment grids, the following other models have been developed and run to test the impacts of changes to the system:

- Plaquemines 1 sensitivity analysis - models the introduction of three spillways across the lower Mississippi River within Plaquemines parish;

- Plaquemines 2 sensitivity analysis - models the removal of all levees along the Mississippi River within the delta which allows the relatively free flow of water across the Mississippi;
- Barrier Islands sensitivity analysis - five different models were run for a selected storm set which represented the post Katrina conditions and 4 different restored island conditions (see Annex A);
- Future Landscape Condition for evaluating LACPR alternatives - No action based on the CLEAR model - this evaluation is described in more detail below;
- Landscape Condition sensitivity analysis - Restored/improved marsh conditions;
- Sea level rise sensitivity analysis - based on 9 storms using the 2010 model grid and increasing water level by 1', 2' and 3'. The 9 storms were selected to represent the 100-year storm event in various locations (**Figure 2.8**).

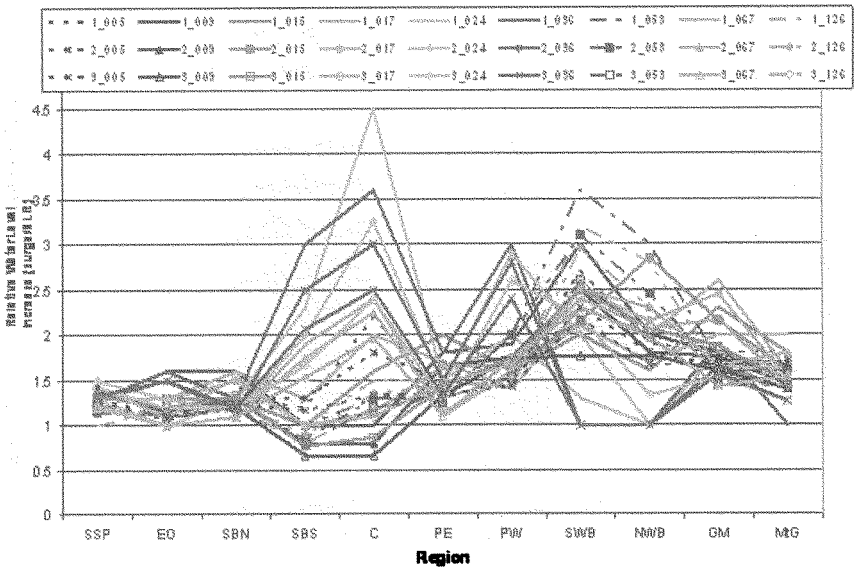

Figure 2.8 - Relative water level increases by reach.

The legend provides the sea level rise (1, 2, and 3 ft) and storm number (5, 9, 15, 17, 24, 36, 53, 67, 126). The regions are: SSP = South shore Lake Pontchartrain, EO = East Orleans, SBN = St Bernard, C = Caernarvon, PE = Plaquemines East, PW = Plaquemines West, SWB = West Bank South, NWB = West Bank North, GM = Golden Meadow, MtG = Morganza to Gulf.

Surge results indicate that the relative increase in water level for a given storm and location decreases as the sea level rise increases. For instance, the response at Caernarvon for storm 24 clearly shows this effect. A sea level rise of 1ft results in a 4.5ft rise, whereas a sea level rise of 3ft results in a 2.5ft rise. The reason is that the amount of surge for a given fetch and wind speed is inversely proportional to the depth. The West Bank and Caernarvon areas showed the largest

variability in response due to the geometric complexity in these regions. Wave results indicate that wave heights generally increase by less than 1 ft. Some areas, however, had 2-3 ft increases in wave height. As with the surge, the rate of wave height increase is less for the larger values of sea level rise.

2.3.4 Future degraded landscape

Topography, landscape features, and vegetation have the potential to reduce storm surge elevations and absorb wave energy. Land elevations greater than the storm surge elevation act as a physical barrier and create bathymetric resistance for the surge and waves. Landscape features such as wetlands also have the potential to create frictional resistance and affect storm surge and wave energy even when below the surge elevation. Understanding the interaction between hurricanes and coastal landscapes is important in planning hurricane flood risk reduction for South Louisiana. In the past, the level of risk reduction provided by wetlands has been empirically estimated with simple "rules of thumb" that state surge is attenuated at a rate of X feet per Y miles of marsh. However, the assumption of a constant attenuation rate implies a simple balance between gravity/water surface elevation gradient and friction. The actual situation is much more complex and dependant on many details including storm intensity, track, forward speed, and surrounding local bathymetry and topography.

The application of empirical surge reduction rates can be misleading because they do not account for the transient nature of forcing or the local topographic/bathymetric conditions. In addition, the empirical data on which these "rules of thumb" are based have a high degree of scatter (which is expected because of the complexity of the processes). A close inspection of the data used to establish the often quoted 1ft surge decrease per 2.75 miles of marsh (Corps of Engineers 1963 report), actually shows a range of values from no surge reduction to 1 ft reduction per 1 mile of marsh.

The surge reducing potential of a given wetland, then, is variable and may differ for different storms. An example of this can be seen from by comparing the predicted maximum water elevation across the Caernarvon marsh for Hurricanes Katrina and Rita. **Figure 2.9** and **Figure 2.10** plot the predicted peak surge elevations for Hurricanes Katrina and Rita, respectively (note that both of these simulations have been validated to high water marks). **Figure 2.9** shows a reduction in surge across the Caernarvon marsh for Hurricane Katrina. For Katrina, the water level decreased 1 ft per 4.4 miles of marsh. For Hurricane Rita, however, **Figure 2.10** shows an increase in surge elevations. Water levels increase over the marsh at a rate of about 1 ft per 8.7 miles of marsh because Rita blew near steady easterly to southeasterly winds toward the delta for a full day. In this scenario, the bathymetric and frictional resistance of the marsh did not play a dominant role in determining the surge level since the winds blew long enough and strongly enough to push and pile the water against the Mississippi River levees.

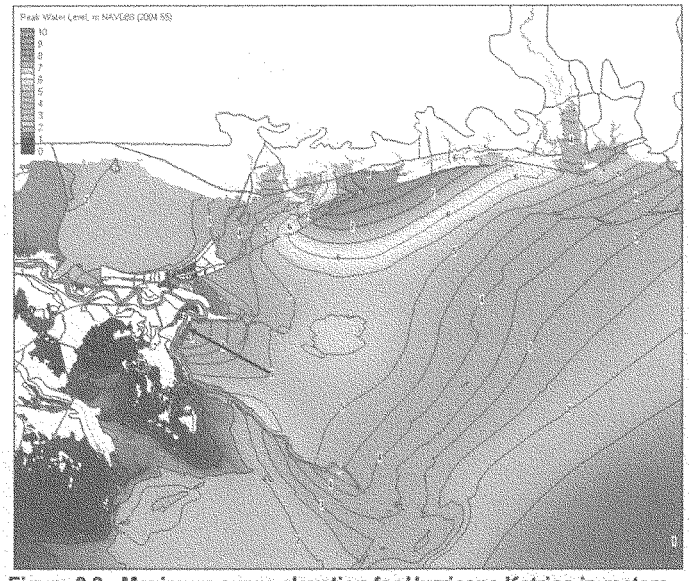

Figure 2.9 - Maximum surge elevation for Hurricane Katrina in meters

Note: The arrow indicates profile across Caernarvon marsh to illustrate surge attenuation rate.

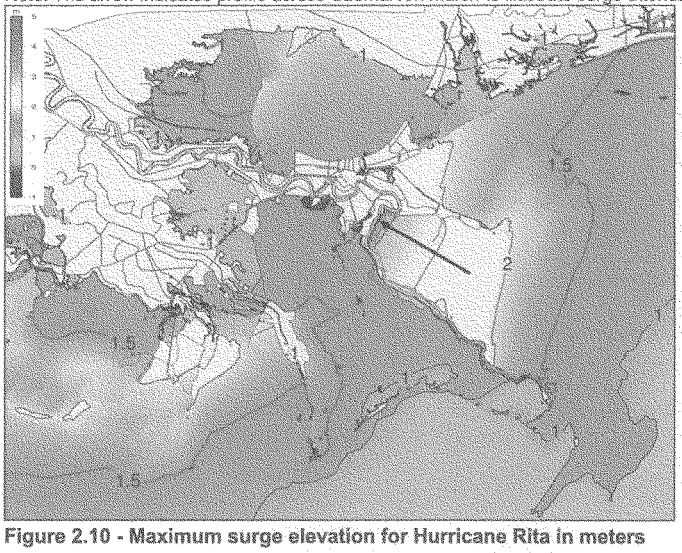

Figure 2.10 - Maximum surge elevation for Hurricane Rita in meters

Note: The arrow indicates profile across Caernarvon marsh to illustrate surge attenuation rate.

The numerical model was applied to assess the potential increase or decrease in surge and waves due to changes in coastal marshes. The landscape conditions included a restored/improved marsh condition and a predicted wetland definition 50 years into the future with no action (primarily a degraded condition). The landscape degradation and restoration is represented in the numerical models by bathymetric and frictional resistance changes. Coastal landscape features can reduce surge potential when land elevations greater than the storm surge elevation act as a physical barrier and create bathymetric resistance for the surge and waves. These features may also reduce surge potential by reducing surface winds due to higher sub-aerial surface roughness and by slowing surge propagation due to bottom friction in shallow flow at the inundation front.

The future no action condition was developed as part of the Coastal Louisiana Ecosystem Assessment and Restoration (CLEAR) Program. The forecasting model developed by CLEAR predicts physical processes, geomorphic features, water quality, and ecological succession. Geomorphic/bathymetric changes are based on the likelihood of discretized regions changing from open water to marsh or marsh to open water. The future condition of Coastal Louisiana predicted by CLEAR predicts degradation across most of Southern Louisiana, but also predicts isolated areas of growth in the Atchafalaya basin and Breton Sound. The CLEAR future condition bathymetry was applied to the model grids and mesh and a series of storm simulations was made. **Figure 2.11** plots the future (CLEAR/no action) landscape bathymetry changes from the base condition. The marsh areas were reduced in elevation by as much as 3 ft across large areas. The Manning n values were also reduced.

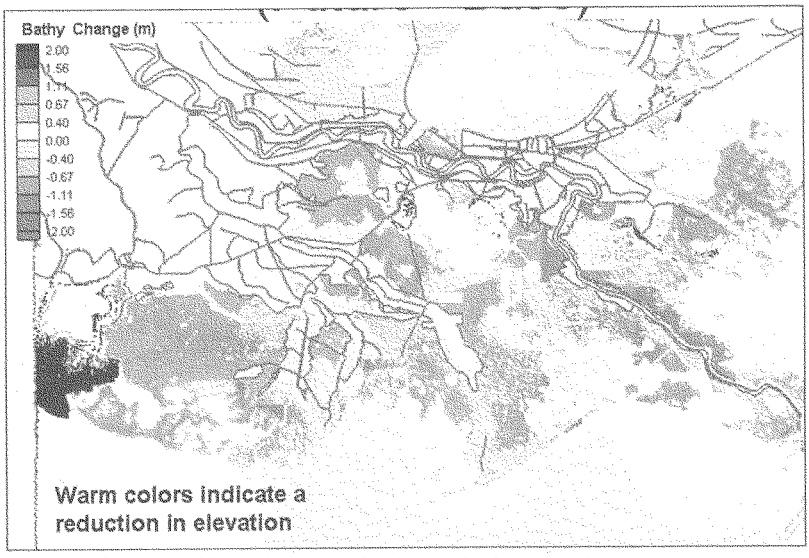

Figure 2.11 - Future CLEAR/no action landscape bathymetry changes from the base condition (future condition – base condition)

To illustrate the impact of marsh degradation, the analysis of changes in water surface elevation across three marsh areas (see **Figure 2.12**) in southeastern Louisiana will be presented. Storm 011 from the JPM-OS is used for the analysis below. Storm 011 has a central pressure of 960 mb, a radius to maximum winds of 21 nautical miles, and a forward speed of 11 knots. The storm track is plotted in **Figure 2.13**.

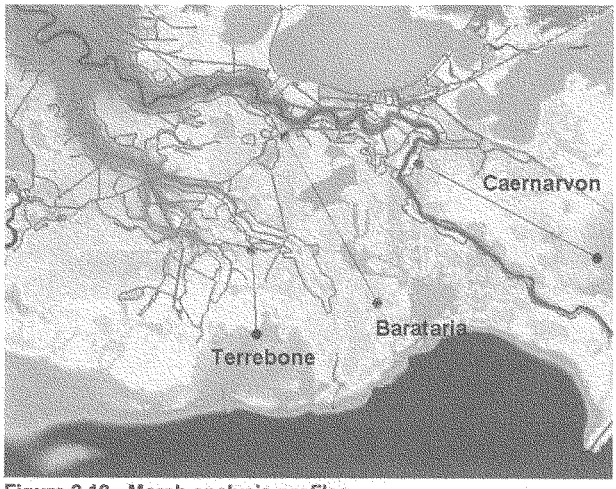

Figure 2.12 - Marsh analysis profiles

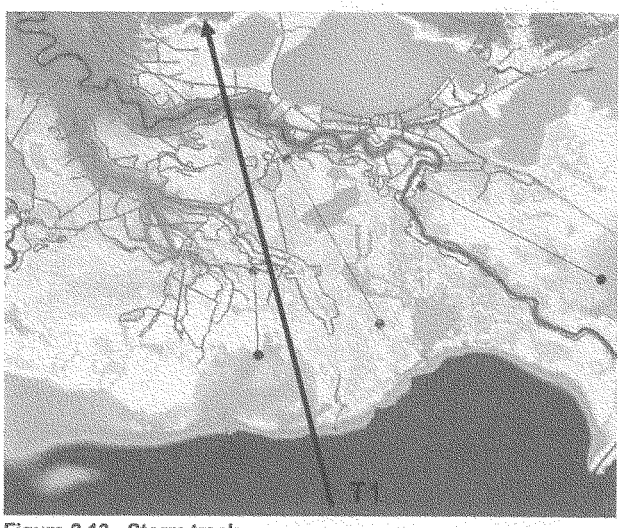

Figure 2.13 - Storm track

The variability of the surge response over different marshes for the same storm is illustrated in **Figures 2.14** through **2.16**, which plot the Storm 011 maximum water surface elevation across the Caernarvon, Barataria, and Terrebonne profiles, respectively. Both the base and future conditions are plotted. As the plots show, the surge for the base condition increases by 5 ft across 39 miles of the Caernarvon marsh, decreases by approximately 4 ft across 39 miles of the Barataria marsh, and decreases approximately 1 ft across 18 miles at Terrebonne. Results are similar for the Caernarvon and Barataria degraded cases, except that we see a greater increase in surge across Caernarvon and a smaller decrease in the surge for the degraded condition across Barataria. For Terrebonne, the degradation of the marsh results in surge nominally increasing across Terrebonne for this storm.

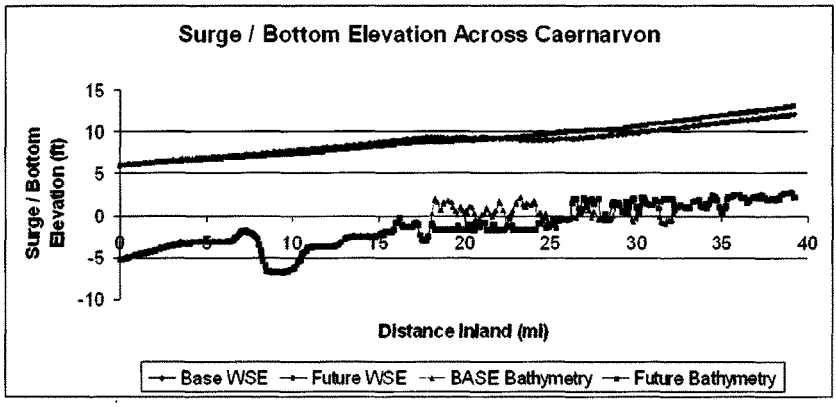

Figure 2.14 - Bottom elevation and surge for Storm 011 across Caernarvon

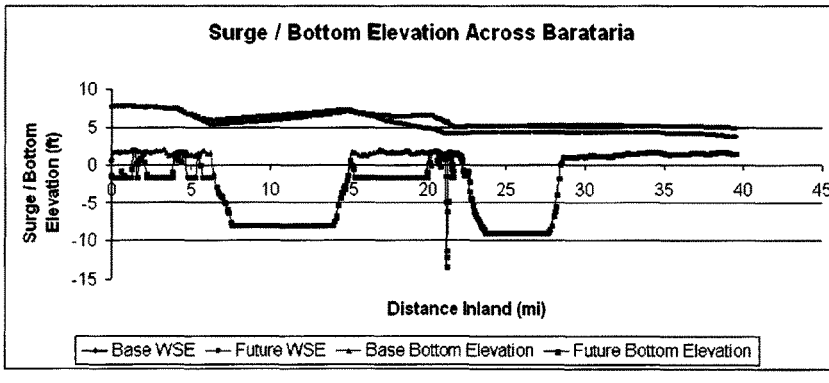

Figure 2.15 - Bottom elevation and surge for Storm 011 across Barataria

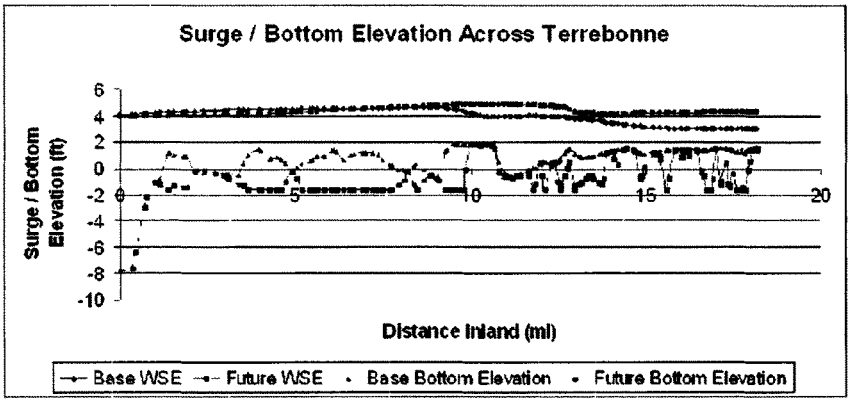

Figure 2.16 - Bottom elevation and surge for Storm 011 across Terrebonne

The difference in surge response between the base and future conditions is further illustrated in **Figures 2.17** through **2.19**, which plot the difference in both the bathymetry and the Storm 011 peak surge for the two conditions. The first thing to note in **Figure 2.17** is that the Caernarvon marsh future condition includes both degradation and land building. The area of improved marsh has little impact on surge elevation. The degraded marsh on the seaward side of the originally emergent marsh, however, results in slightly lower surge seaward of the degradation and an increase in surge landward of the degraded marsh. This occurs because the reduced frictional resistance resulting from the loss of marsh allows the surge to propagate further across the marsh during the storms passage. The marsh degradation occurs over about 8 miles of marsh and the result is about a 1 ft increase in surge. Across Barataria, there are two distinct areas of marsh degradation which both result in increased surge (see **Figure 2.18**). The first area of degradation is approximately 7 miles and surge increases by less than 1 ft. The second area of degradation is also approximately 7 miles across but the surge increase by nearly 2 ft relative to the base condition. Further inland the surge is increased for the degraded condition relative to the base condition by about 1 ft. For Terrebonne (**Figure 2.19**), the entire 18 mile profile has been degraded and the surge for the future condition is slightly lower over the seaward half of the marsh and increases relative to the base case on the landward side on the order of 1 ft.

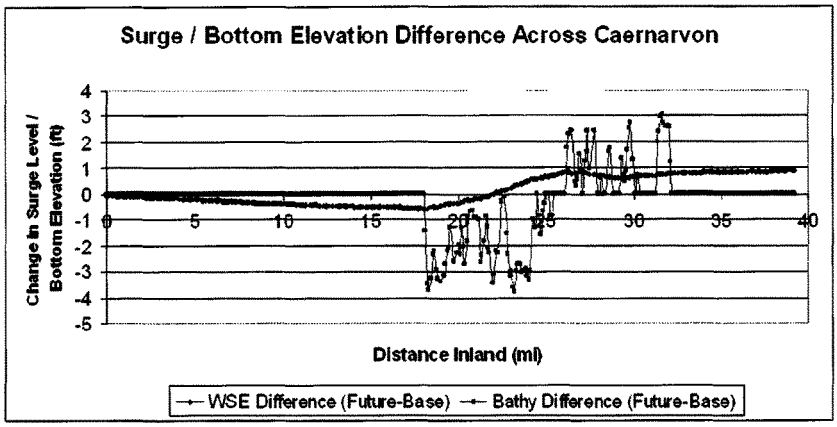

Figure 2.17 - Difference between future and base conditions in bottom elevation and surge for Storm 011 across Caernarvon

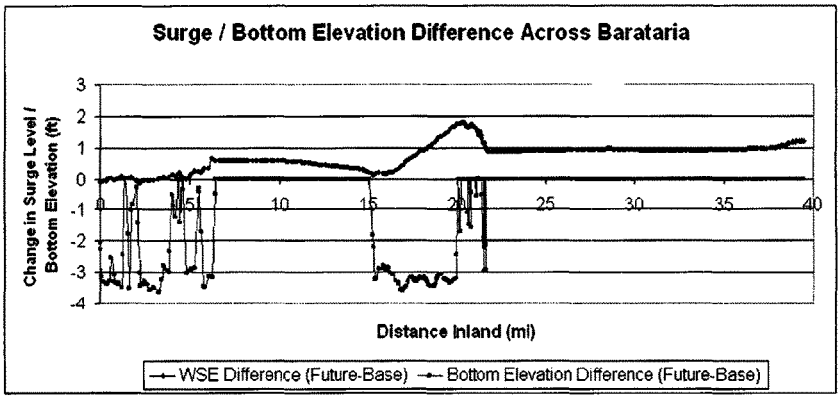

Figure 2.18 - Difference between future and base conditions in bottom elevation and surge for Storm 011 across Barataria

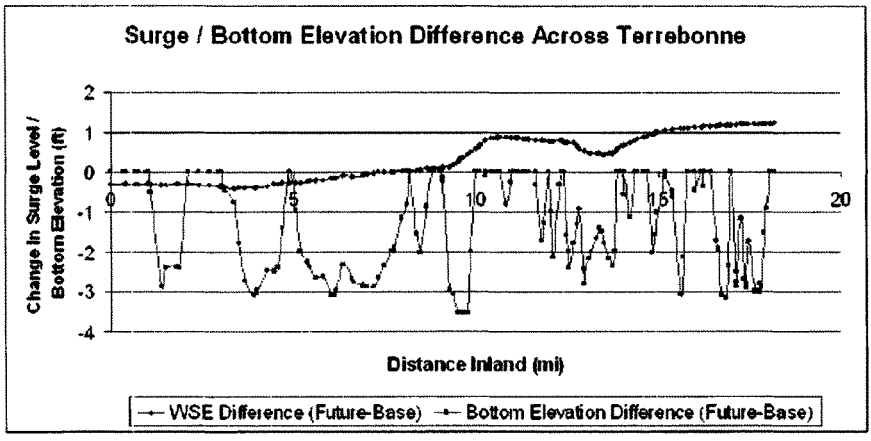

Figure 2.19 - Difference between future and base conditions in bottom elevation and surge for Storm 011 across Terrebonne

The above analysis illustrates the complexity surge response over marshes wetlands. The response of the surge to the wetland is variable. The complex, transient nature of the forcing, the local topography/bathymetry, and the full dynamics of the governing force balance preclude the application of empirically derived constant attenuation rates. The influence of wetlands on surges was evaluated stochastically by simulating a 304 storm suite for the JPM-OS statistical methodology. **Figures 2.20** through **2.22** plot the 100-year water surface elevation for both the base and the future conditions across the Caernarvon, Barataria, and Terrebonne profiles, respectively. In general, the plots indicate that the degradation that is projected to occur over the next 50 years results in relatively small changes in the 100-year water levels. Water levels for the future degraded case are typically lower over and seaward of the degraded marsh and higher landward of the degraded marsh area. These conclusions generally hold for the 400- and 1000-year water surface elevations as well. **Figures 2.23** through **2.25** plot the difference in the 100-, 400-, and 1000-year maximum water surface elevations for the two conditions across each profile. The increase in water surface elevation for all return periods is generally less than 2 ft. Reductions on the seaward side of marsh degradation are less than 1 ft. The plots show that the wetlands do have surge reduction potential but that it can be variable across the marsh. The degradation of the marsh essentially results in a redistribution of the water and typically more water is allowed to propagate further landward when the wetland is degraded.

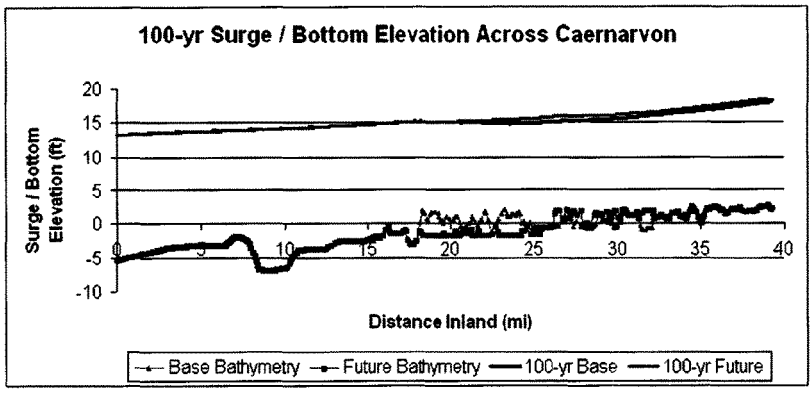

Figure 2.20 - Bottom elevation and 100-year water surface elevation across Caernarvon

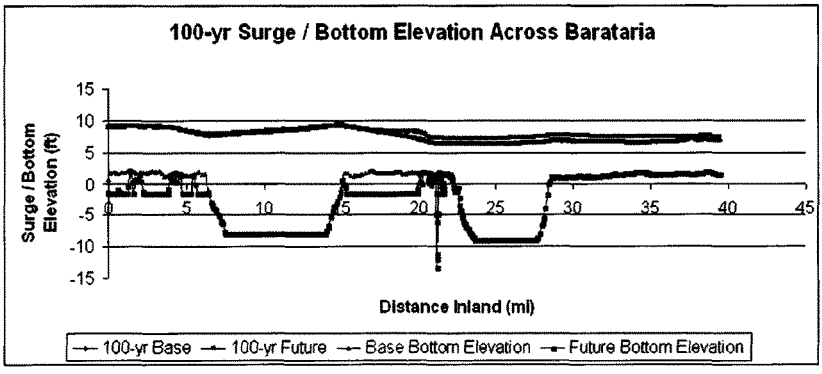

Figure 2.21 - Bottom elevation and 100-year water surface elevation across Barataria

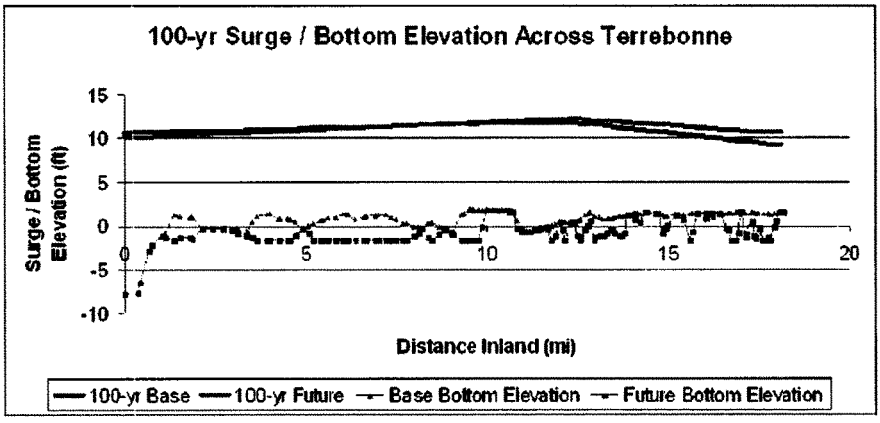

Figure 2.22 - Bottom elevation and 100-year water surface elevation across Terrebonne

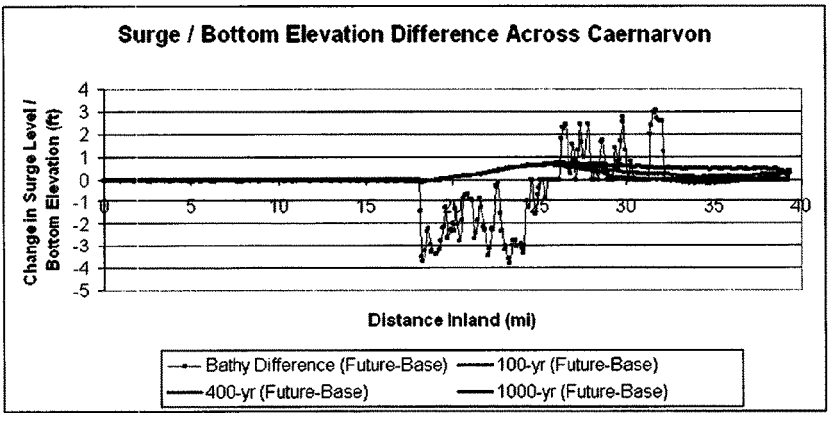

Figure 2.23 - Difference between future and base conditions for the 100-, 400-, and 1000-year water surface elevation across Caernarvon

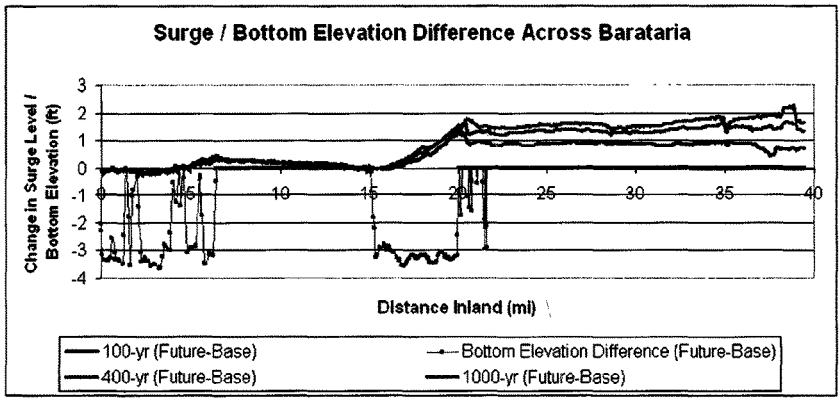

Figure 2.24 - Difference between future and base conditions for the 100-, 400-, and 1000-year water surface elevation across Barataria

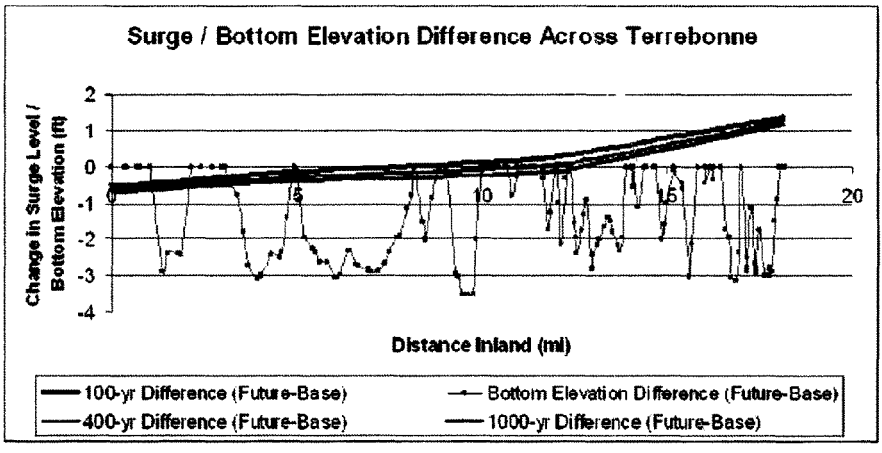

Figure 2.25 - Difference between future and base conditions for the 100-, 400-, and 1000-year water surface elevation across Terrebonne

Figure 2.26 plots the difference in peak surge elevations between the future and base conditions across all of coastal Louisiana. The plot is generally consistent with the analysis presented above with water surface elevation changes generally less than 2 ft, with the greatest change being increases landward of degraded marsh. There is essentially no change in Lake Pontchartrain and the greatest change is in the Barataria Basin. Note that in the vicinity of the Atchafalaya River, the future condition is predicted to have lower surge values than the base condition. The reason for

this is that this area is predicted to experience land building, not loss. Reductions in western Louisiana are similar to the east with reduction of less than 2 ft.

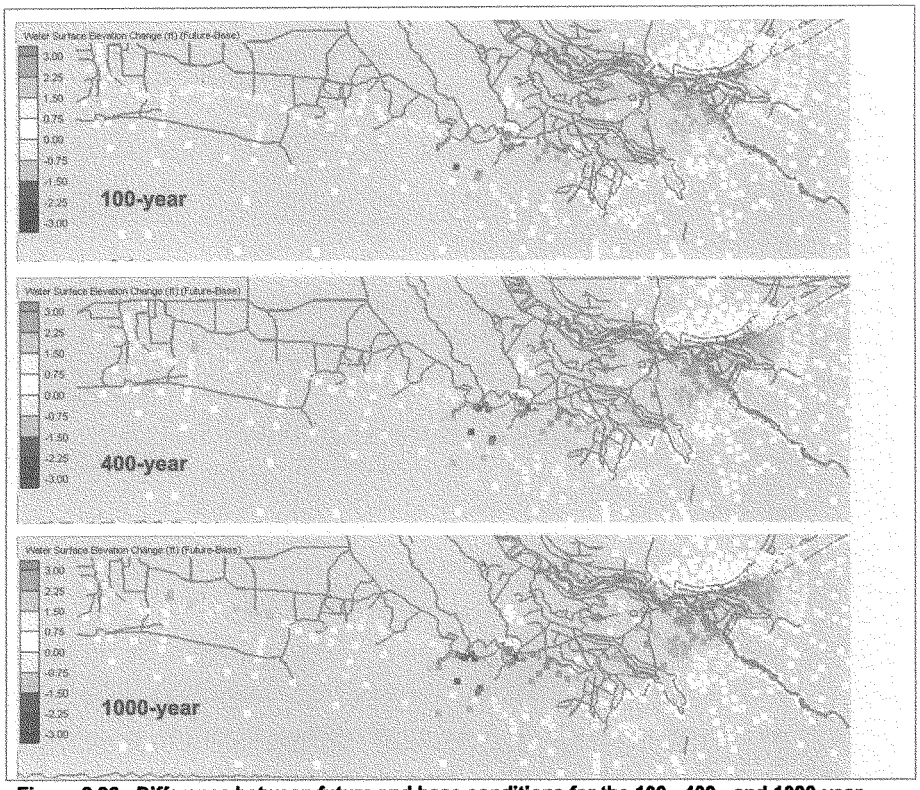

Figure 2.26 - Difference between future and base conditions for the 100-, 400-, and 1000-year water surface elevation across coastal Louisiana

The purpose of this analysis was to apply numerical models to assess the potential of coastal landscape features for reducing storm surge and waves for hurricanes with varying intensity. The impact of landscape features on surge propagation is a relatively new application for surge and wave models and an area of active research that suffers from a lack of quality data. The analysis provides valuable information on trends and relative performance but caution should be exercised

in making definitive quantitative assessments. Results indicate that coastal marsh does have surge reduction potential. The magnitude of the surge reduction potential is variable along the coast, with the maximum magnitude of the change generally being less than 2 ft for the 100-, 400- and 1000-year elevations.

2.4 West models

In addition to the 2007 baseline model, for the west three models have been developed to represent three potential levee alignments. These models are described briefly below.

Morganza and GIWW alignment (Grid WA)

This model includes a non-overtopping levee which followed the proposed Morganza alignment to the west of Bayou Lafourche, connecting with the existing levee system around Morgan City and Patterson before following a line to the seawards of the GIWW to Lake Charles, and then tying into high ground just before the Louisiana / Texas border.

Figure 2.27 shows the differences in maximum water level between the WA model grid and the 2007 base model.

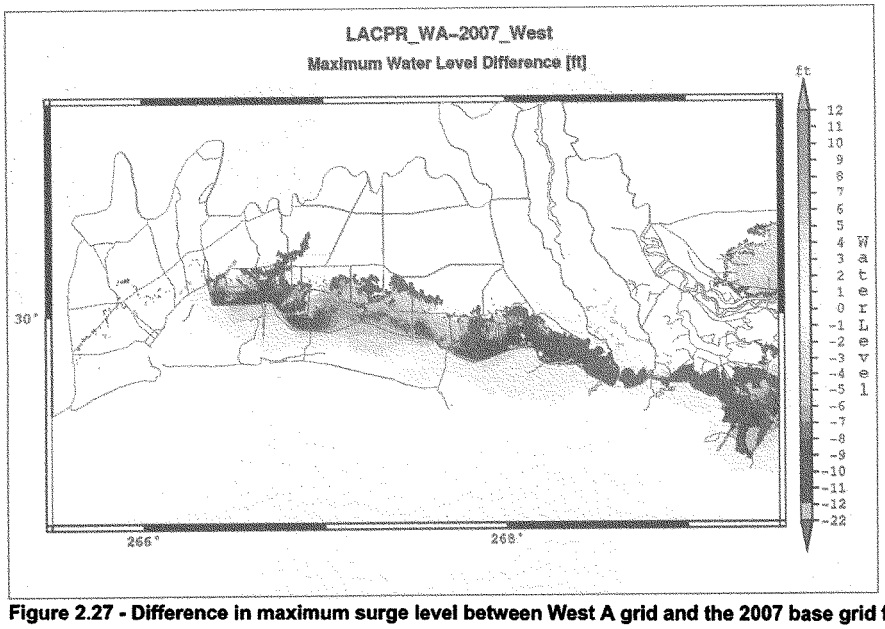

Figure 2.27 - Difference in maximum surge level between West A grid and the 2007 base grid for the West A storm suite

Morganza and Retired Levee Alignment, plus ring levees (Grid WB)

This model includes an incomplete levee system around Morganza, with two lines of defense, a non-overtopping levee from Morgan City to Abbeville, following a line between the GIWW and the higher ground, with the levee returning to higher ground around the Vermillion River. To the west of the Vermillion the model included small ring levees around a couple of areas of population and a larger ring around Lake Charles.

Figure 2.28 shows the differences in maximum water level between the WB model grid and the 2007 base model

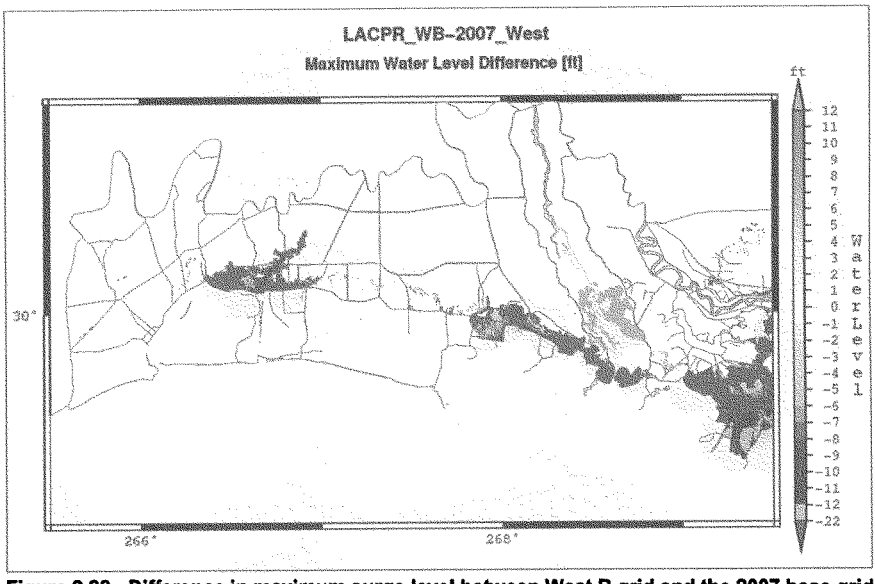

Figure 2.28 - Difference in maximum surge level between West B grid and the 2007 base grid for the West B storm suite

Combined Alignments (West C)

This model considers a twin line of defense within Morganza, with a 100-year defense round the outer line and a non-overtopping levee on the inside, a lower level of risk reduction between Houma and Morgan City. Between Patterson and Abbeville the same alignment as West B has been used. From Abbeville to Lake Charles a 100-year levee was modeled along the GIWW alignment, eventually tying into a ring levee around Lake Charles.

Figure 2.29 shows the differences in maximum water level between the WC model grid and the 2007 base model

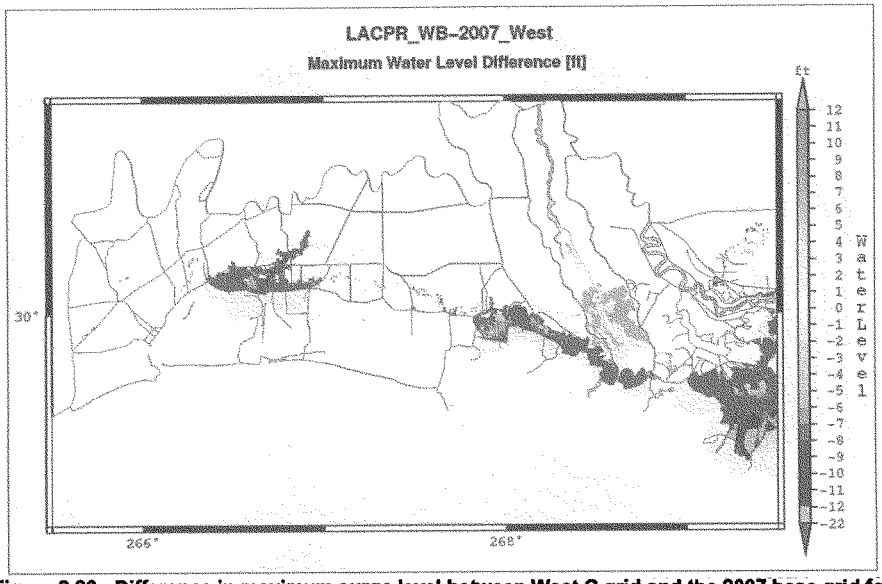

Figure 2.29 - Difference in maximum surge level between West C grid and the 2007 base grid for the West C storm suite

Future Degraded

In the same way as for the east, a model has been developed based on the projected changes in marsh etc over the next 50 years. This prediction has been made as part of the CLEAR program. The model uses the basic 2007 grid as a starting point.

Figure 2.30 shows the differences in maximum water level between the CLEAR/no action model grid and the 2007 base model. The degraded landscape generally results in around a 1 to 2 ft increase in peak surges across western Louisiana with the exception of the Atchafalaya area. This is suggested to equate to possible marsh growth as a result of discharges from the Atchafalaya River.

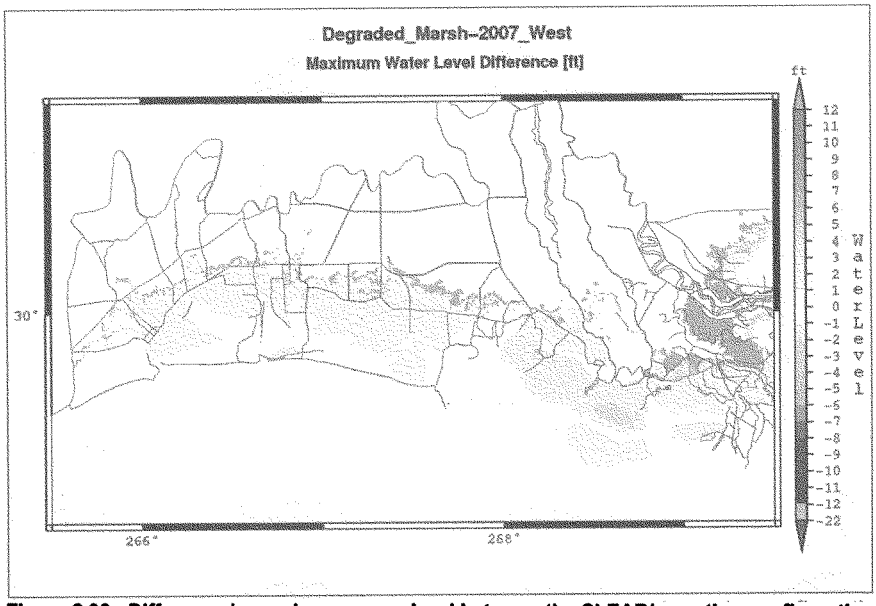

Figure 2.30 - Difference in maximum surge level between the CLEAR/no action configuration and the 2007 base case for the no action marsh storm suite

2.5 Statistics of storm surges and wave conditions

The previous sections of this chapter have described the storm surge and wave models developed for LACPR. The results from these models represent specific storms and only the peak values were extracted from the full suite of storms. For use within the derivation of levee heights and economic damage calculations based on flood depths the results need to be translated into stage frequency relationships. This is done using the JPM-OS methodology described in volume I.

The JPM-OS methodology uses the results from the models and statistically obtains the water level or wave conditions relating to a particular return period of event. This has been done for all models at selected points (discussed further below), and for some model (the base models) for all points. Undertaking the analysis at all points within the model allows for the creation of water level and wave height surfaces for display, which cannot be easily created from the reduced point sets used for the main calculations.

The created surge and wave values do not represent a single storm event with a particular frequency. Rather they represent the water level or wave condition that could occur with that frequency. For example, if the 100-year surge level occurs along the South Shore of Lake Pontchartrain then it is very unlikely that at the same time the 100-year surge level occurs in the Barataria Basin (as the values could be higher or lower depending on the track of the storm).

Surfaces of extreme water level have been prepared for some of the models for a range of event frequencies. The following figures show the 100-year and 400-year frequency water levels for the 2010 base model for Planning Units 1 and 2, and the 2007 base model for Planning Units 3a, 3b and 4 are shown here in **Figures 2.31** through **2.40**. A complete set of statistical surfaces for the various alternatives plans can be found in the Evaluation Results Appendix.

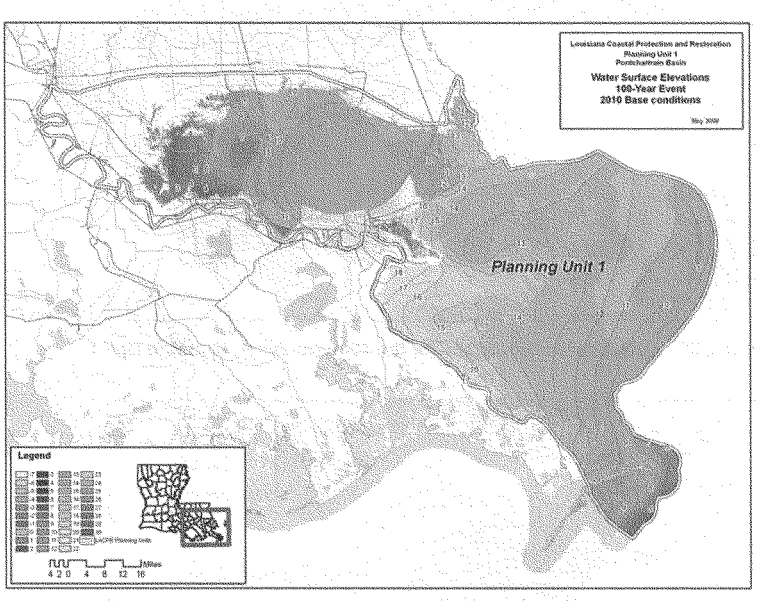

Figure 2.31 - Statistical water surface for the 100-year event, Planning Unit 1

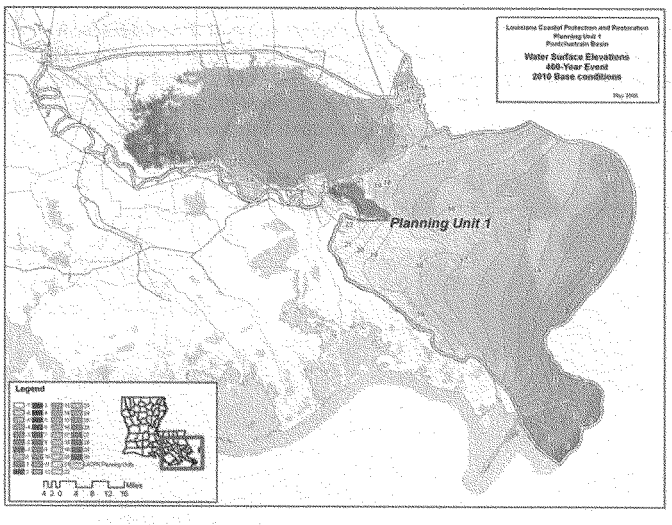

Figure 2.32 - Statistical water surface for the 400-year event, Planning Unit 1

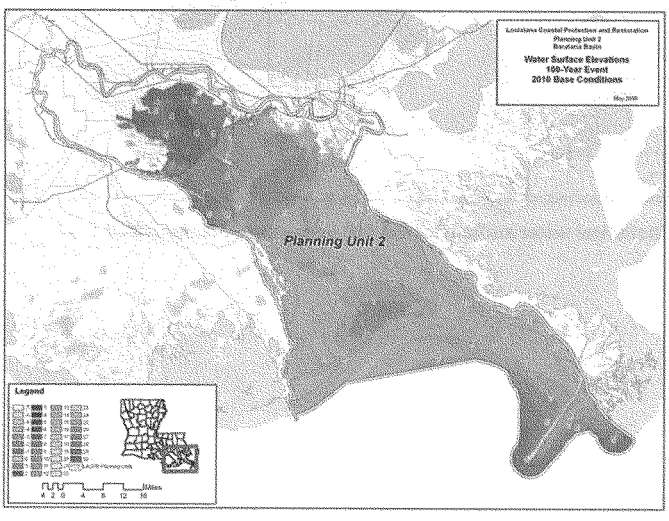

Figure 2.33 - Statistical water surface for the 100-year event, Planning Unit 2

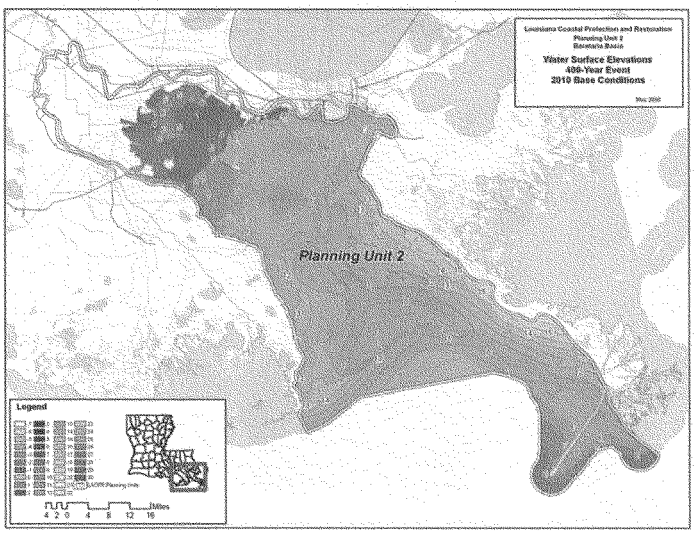

Figure 2.34 - Statistical water surface for the 400-year event, Planning Unit 2

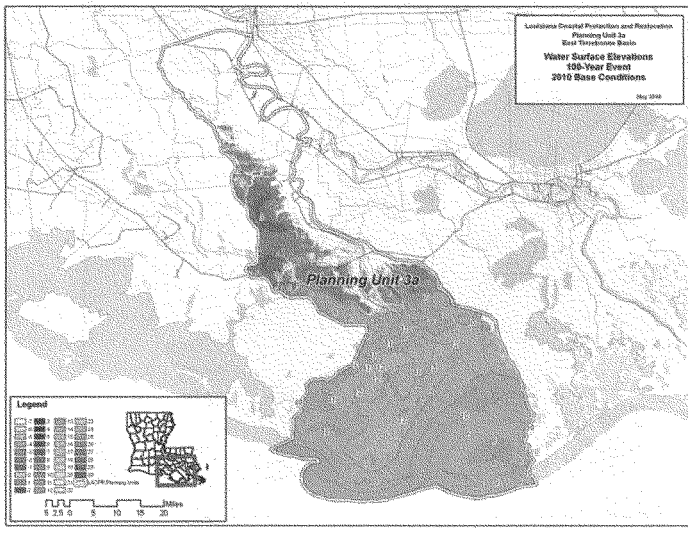

Figure 2.35 - Statistical water surface for the 100-year event, Planning Unit 3a

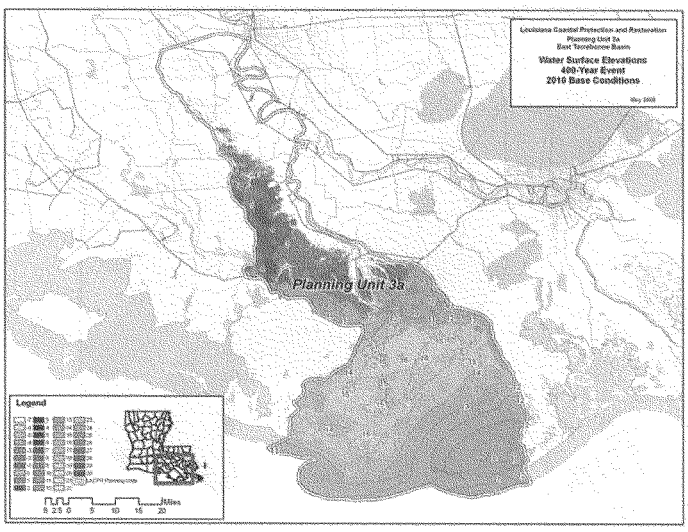

Figure 2.36 - Statistical water surface for the 400-year event, Planning Unit 3a

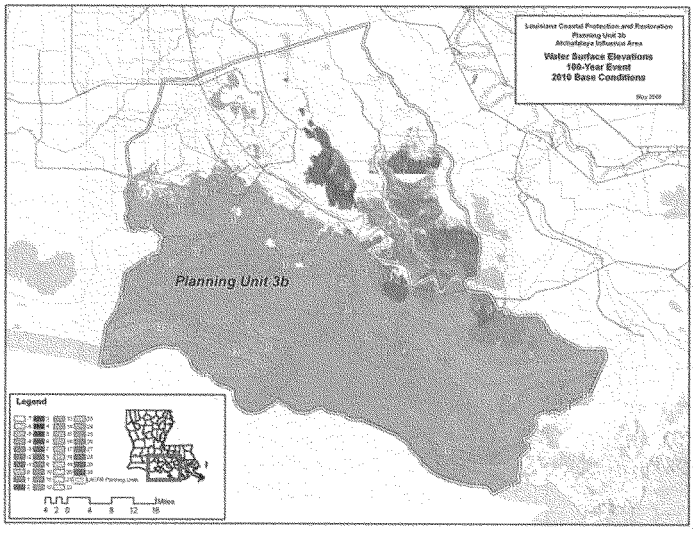

Figure 2.37 - Statistical water surface for the 100-year event, Planning Unit 3b

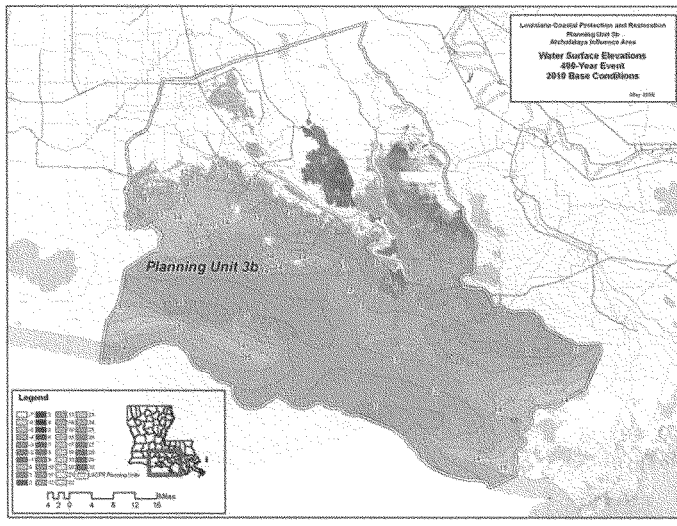

Figure 2.38 - Statistical water surface for the 400-year event, Planning Unit 3b

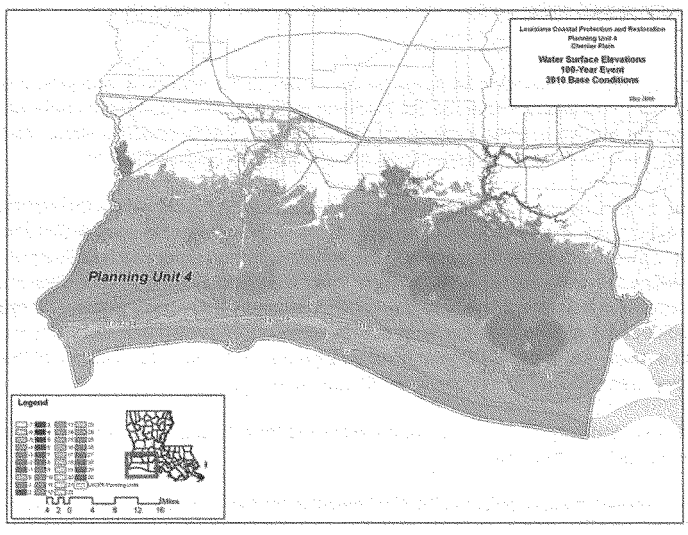

Figure 2.39 - Statistical water surface for the 100-year event, Planning Unit 4

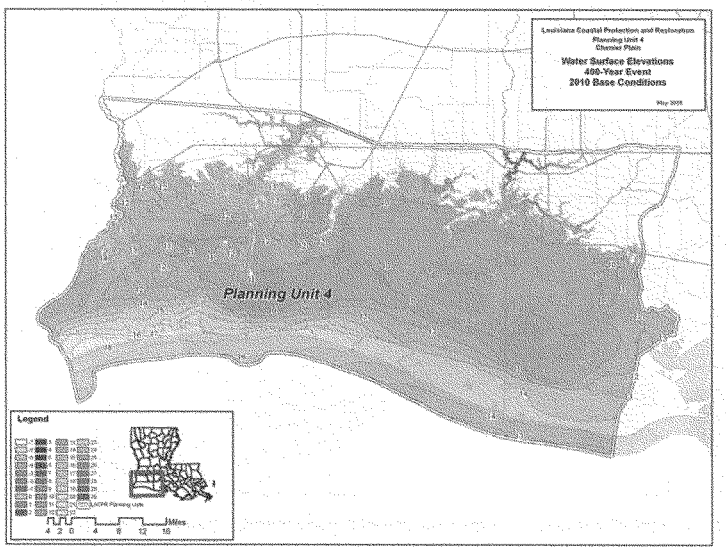

Figure 2.40 - Statistical water surface for the 400-year event, Planning Unit 4

To establish the stage frequencies for surge levels for use in the economic analysis, a series of points were developed that represented the planning subunits. For these points data was extracted from the models to obtain values for the 100-, 400-, 1000- and 2000-year events. The 10-year surge levels have been obtained from the 2006 FEMA Region 6 study.

In the west some problems occurred with the inland parts of the model where the statistical values of surge decayed faster than expected. The low values were adjusted to ensure that the 100-year or higher events were not lower than the 10-year value. This was achieved by raising the 100-year and higher values to be the same as the 10-year values. In addition, the values were checked to ensure a progression from the 10-year to the 2000-year, and values were adjusted if required. The process of adjustment always took the level at the higher return period and used this for lower return period events - i.e. if the 1000-year level was greater than the 2000-year level then the 1000-year level was reduced to match the 2000-year level.

The wave heights and surge levels have been extracted for a series of point files. The following three sets of points have been used:

L274 - representing a set of points in the East selected for the purposes of levee design in Planning Units 1 and 2;

W177 - representing a set of points in the west selected for the purposes of levee design in Planning Units 3a, 3b and 4;
Q835 - covering the whole model areas and used for quality control and comparison of results outside of the key design areas.

The spatial extent of these three data sets is given in **Figures 2.41** through **2.43**.

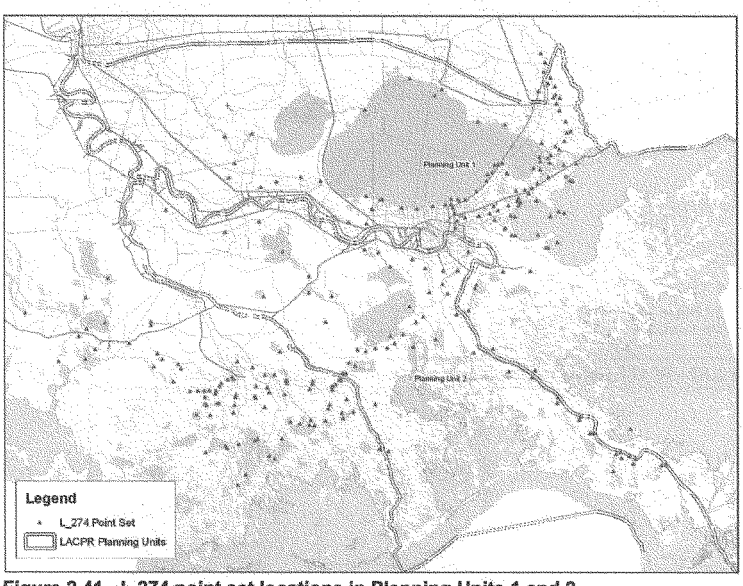

Figure 2.41 - L 274 point set locations in Planning Units 1 and 2

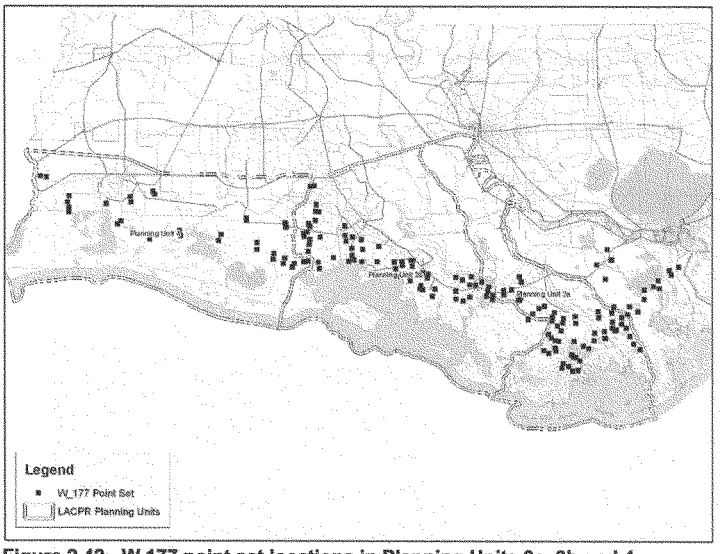

Figure 2.42 - W 177 point set locations in Planning Units 3a, 3b and 4

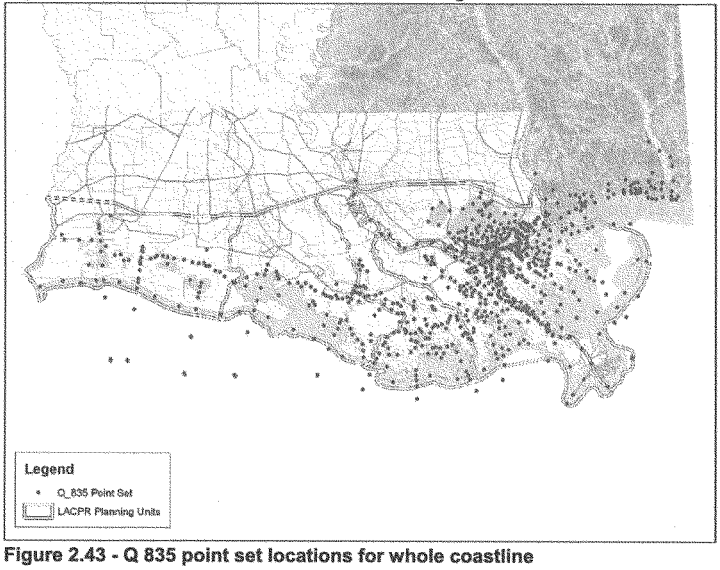

Figure 2.43 - Q 835 point set locations for whole coastline

2.6 Discussion on East models

There are four basic model grids for the Lake Pontchartrain area that model some form of closure of the Lake Pontchartrain. These four models represent the full (impenetrable) barrier (East A), the approximately 100-year surge level weir (East B), a full barrier with openings at Chef Menteur and the Rigoletes (East C) and a barrier across Lake Borgne (East D).

Below, the four options are compared in order to investigate their relative impacts on surge levels within Lake Pontchartrain, outside Lake Pontchartrain and along the Mississippi coastline. In addition, the East B grid has been compared to the 2010 base grid to investigate particularly the potential impacts on the Mississippi coast.

2.6.1 Comparison of weir with full barrier (East B with East A)

The setup and results of the full closure (East A) and weir plan (East B) have been described separately in Section 2.3.2.

Comparison of changes on Mississippi coast
The weir plan results in maximum changes on the Mississippi coast of up to 3.2 ft and beyond Bay St. Louis the maximum drops to 0.8ft and to 0.5ft by Gulfport. The average change is a maximum of 2.3 ft adjacent to the Louisiana border, dropping to 0.5 ft beyond Bay St. Louis and 0.2ft at Gulfport. For the full barrier the maximum values are 5.4ft, 1.8ft and 1.3ft at the same points and the averages are 3.6ft, 0.9ft and 0.4ft.

The differences have been made by comparing the results of 36 hurricanes with different tracks and intensities that had been run for the 2010 baseline, East A and East B model grids. The maximum difference values arose from hurricanes of frequency > 400 year (i.e. similar or greater than Hurricane Katrina) with a track passing from south to north and making landfall around the end of the Barataria Bay.

These results are shown in **Figure 2.44** and **Figure 2.45**.

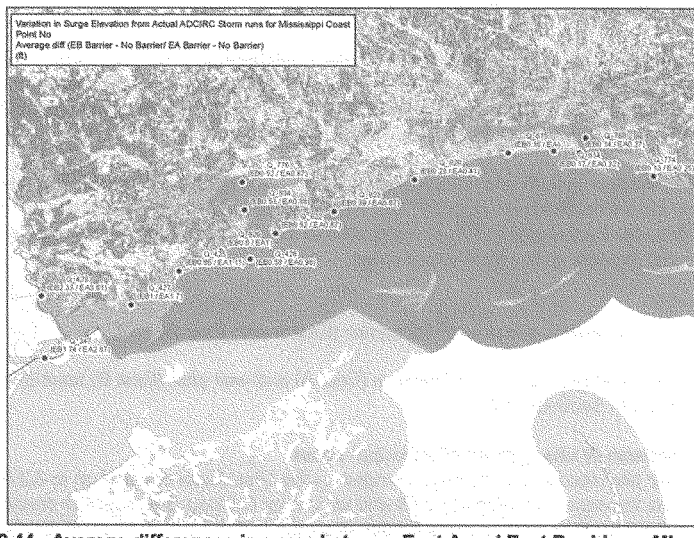

Figure 2.44 - Average differences in surge between East A and East B grids on Mississippi coast

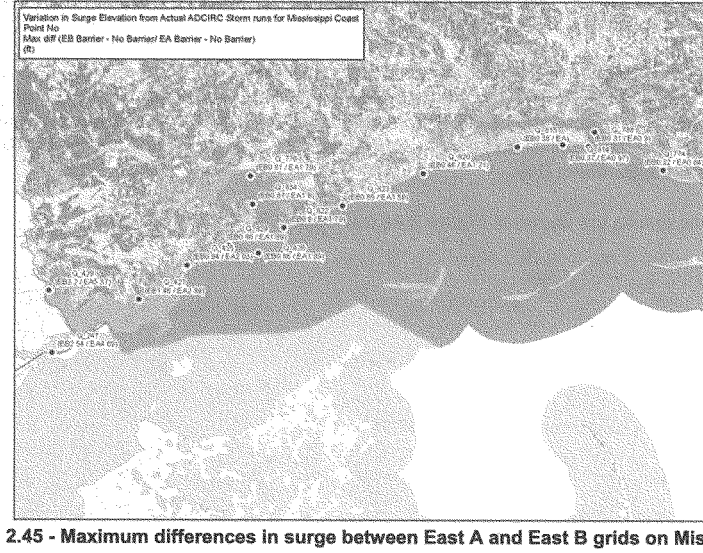

Figure 2.45 - Maximum differences in surge between East A and East B grids on Mississippi coast

Comparison of changes inside and outside Lake Pontchartrain

Full blockage (East A) when compared to a 100-year surge level weir (East B) shows additional reductions in 100-year surge level within the lake of between 3.1 ft close to the weir to 0.1-0.2 ft on the far side of the lake. The differences are shown on **Figure 2.46**. Outside of the barrier levels are raised by at least 1 ft for an area from St. Bernard Parish across to the Mississippi coast. Differences are greater for the 1000-year storm where the full closure makes a larger impact on levels in the lake, reducing maximum surge levels by up to 7 ft over the weir alternative, shown in **Figure 2.47**.

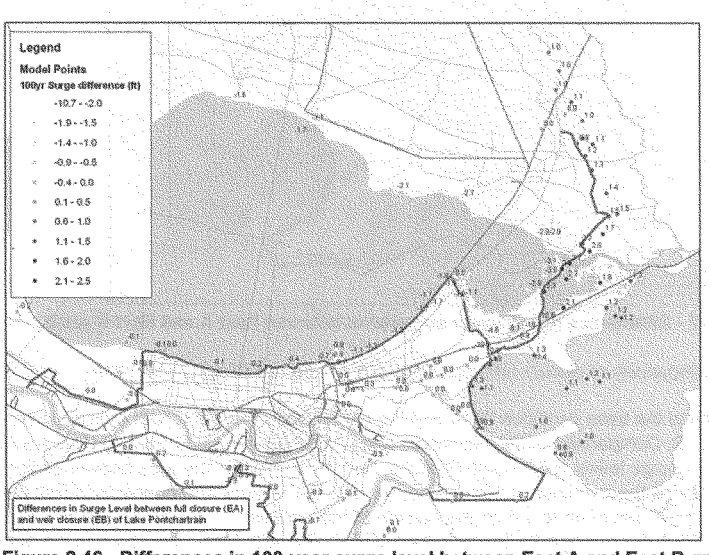

Figure 2.46 - Differences in 100-year surge level between East A and East B grids

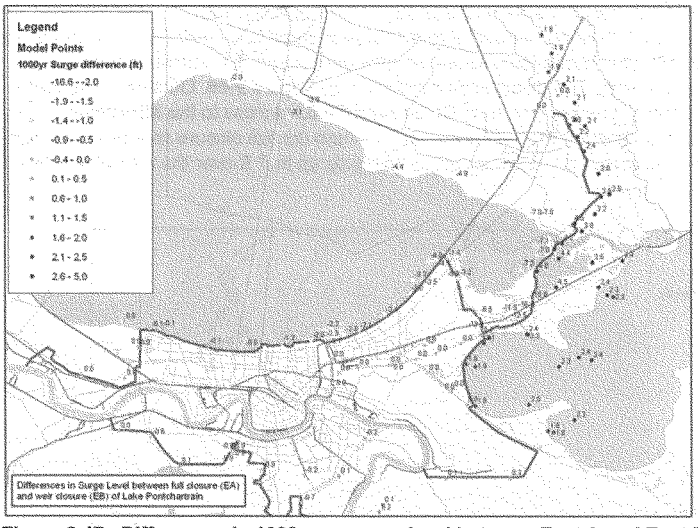

Figure 2.47 - Differences in 1000-year surge level between East A and East B grids

2.6.2 Comparison of closure with openings and no closure (East C with base)

The results of the base condition 2010 and the partial closure (East C) are described separately in Section 2.3.2. Comparison of the two model grids (2010 baseline and East C) shows a limited reduction in surge levels within Lake Pontchartrain. Apart from in the area between New Orleans East and Slidell the reduction in 100-year surge with the closure with openings is less than 1ft, and it induces increases of up to 1 ft on the outside of the closure. For the 1000-year the changes are around the 1 to 1.2 ft range for the main part of the Lake, with increases of 1.5 to 2 ft outside the closure. The differences between East C and baseline are shown in **Figure 2.48** and **Figure 2.49**.

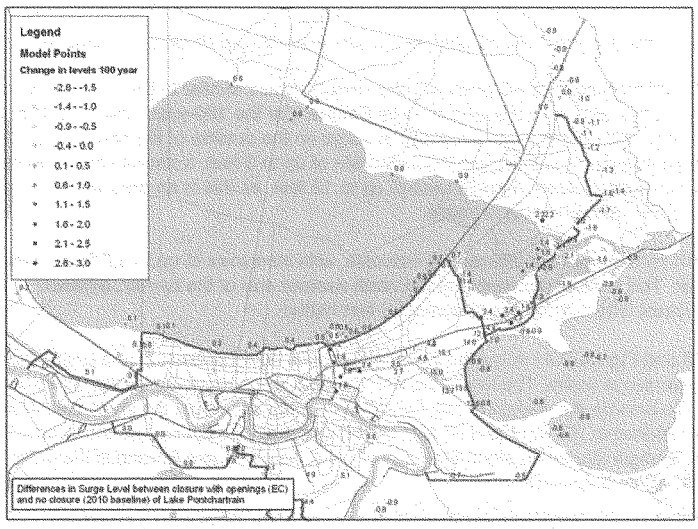

Figure 2.48 - Difference in 100-year surge level between the East C and 2010 base grids

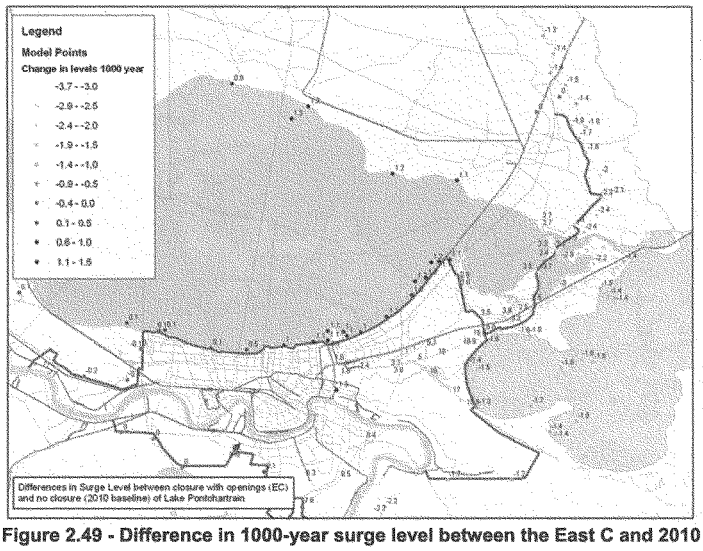

Figure 2.49 - Difference in 1000-year surge level between the East C and 2010 base grids

2.6.3 Comparison of full block more seaward with weir closure (East D with East B)

The East D grid represents a full block of Lake Pontchartrain, similar to East A but at a location further towards the Gulf, on a line across Lake Borgne. For the 100-year surge the differences between East D and East B show increases in surge on the outside of the barrier of around 2 feet, and increased reductions within Lake Pontchartrain of up to 3 feet. It shows very large surge reductions within the Golden Triangle area of up to 13 feet, as this is an area which before was on the outside of the weir, and now is inside.

For the 1000-year surges the differences increase, with increases of up to 3.5 feet on the outside of the barrier. This is most noticeable at the north eastern end of the barrier, adjacent to the Mississippi coast where the surge is some 3.3 feet higher.

Figure 2.50 and **Figure 2.51** show the surge differences between East D and East B for the 100- and 1000-year surge levels. The key issue with the East D model would be the constructability of the barrier. To provide the degree of risk reduction given in the model the levee would need to be in excess of 26 feet at 100-year and 37 feet at 1000-year. These figures are those developed for the Golden Triangle alignment along the edge of Lake Borgne. Lake Borgne is the order of 8-10 feet deep along the alignment.

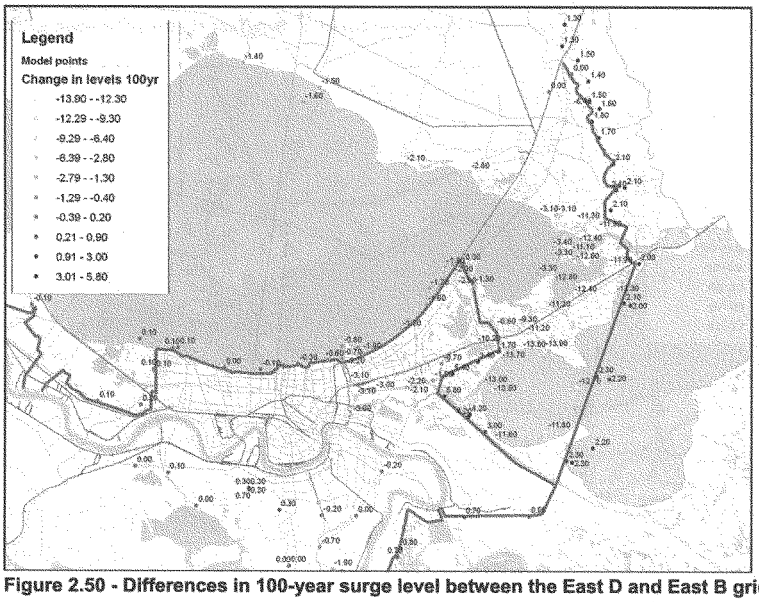

Figure 2.50 - Differences in 100-year surge level between the East D and East B grids

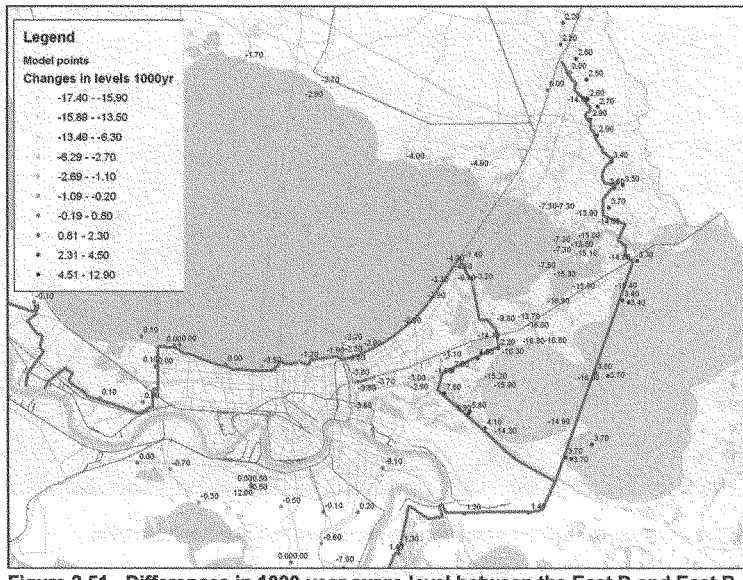

Figure 2.51 - Differences in 1000-year surge level between the East D and East B grids

2.6.4 Weir alternative (East B) to 2010 base - Risk reduction in and around Lake Pontchartrain and potential Impacts on Mississippi.

The Lake Pontchartrain surge reduction weir plan was modeled with the East B weir set at approximately the 100-year surge level thus preventing any surges generated in the Gulf and transmitting via Lake Borgne into Lake Pontchartrain until such time that levels exceed 12 feet at the weir face. Therefore for all events where Lake Borgne stages are blocked from entering Lake Pontchartrain, surges in Lake Pontchartrain are generated only by wind shear blowing across the lake causing the normal water level in the lake to tilt up on the windward side and to depress on the leeward side.

A comparison of water level frequencies for the Lake Pontchartrain 2010 base to water level frequencies for the Lake Pontchartrain surge reduction plan, East B, for the four locations shown on **Figure 2.52** follow on **Figures 2.53** through **2.56**.

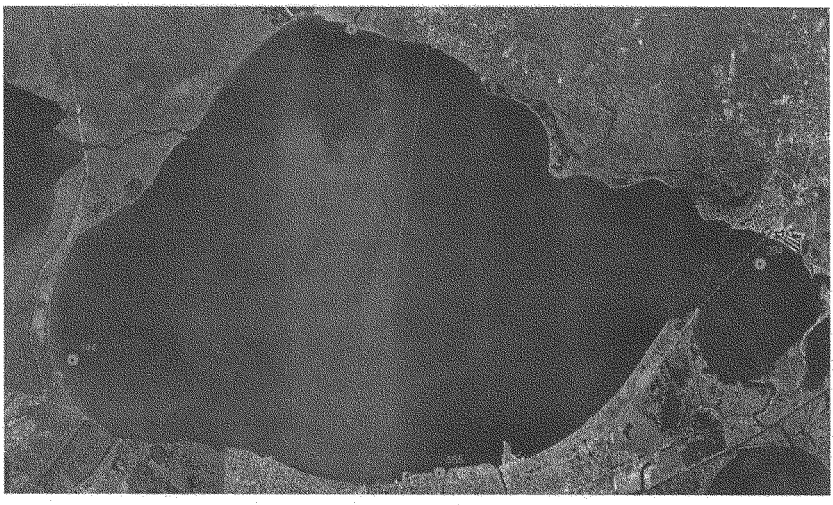

Figure 2.52 - Point locations for stage frequency curves in Lake Pontchartrain

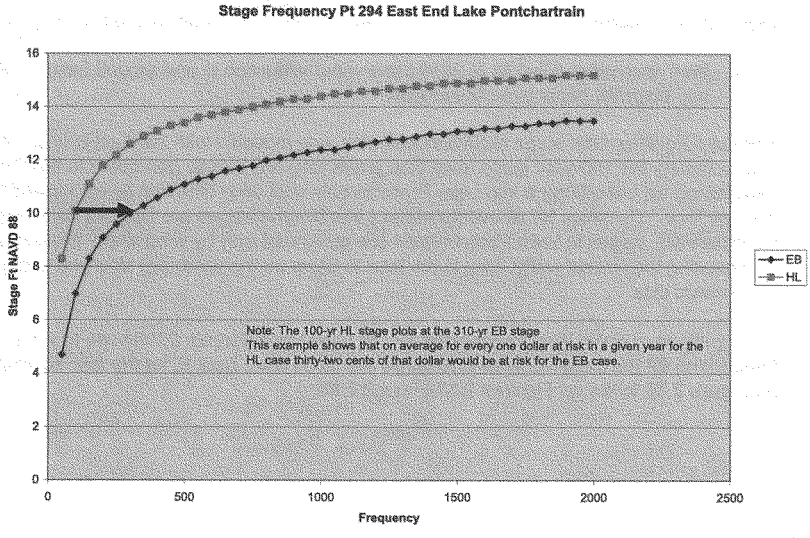

Figure 2.53 - Stage-frequency curves point 294 east end of Lake Pontchartrain

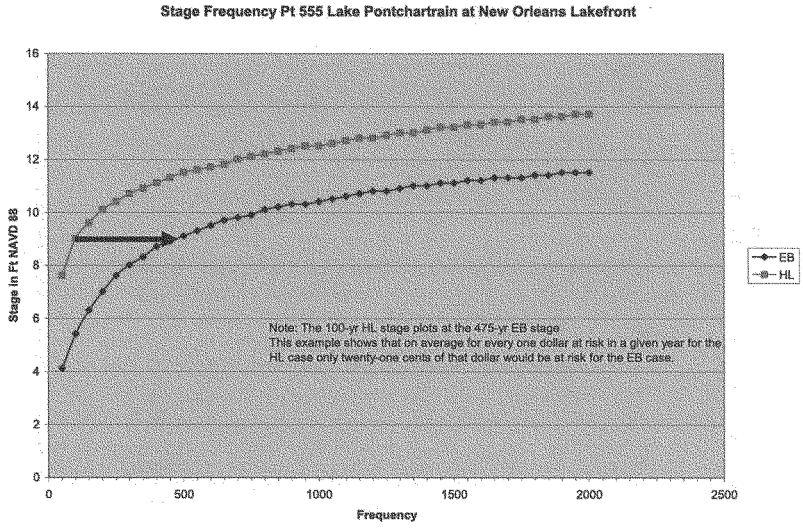

Figure 2.54 - Stage-frequency curves point 555 New Orleans lakefront Lake Pontchartrain

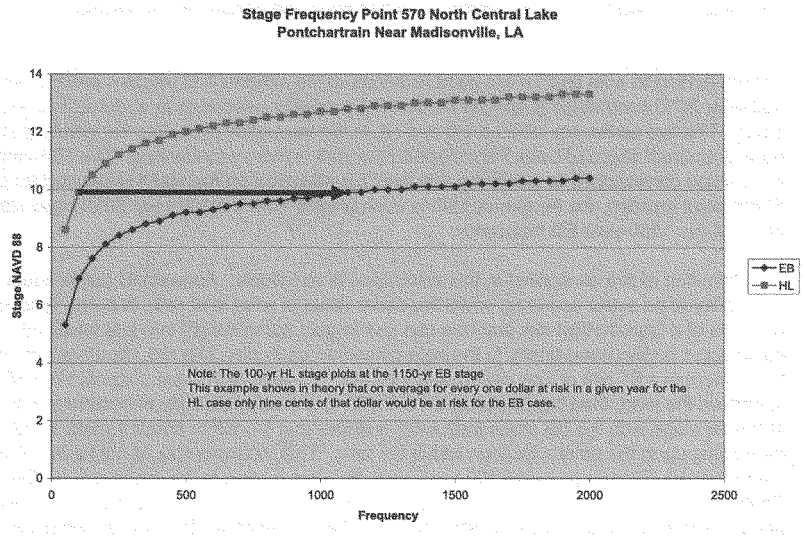

Figure 2.55 - Stage-frequency curves point 570 near Madisonville of Lake Pontchartrain

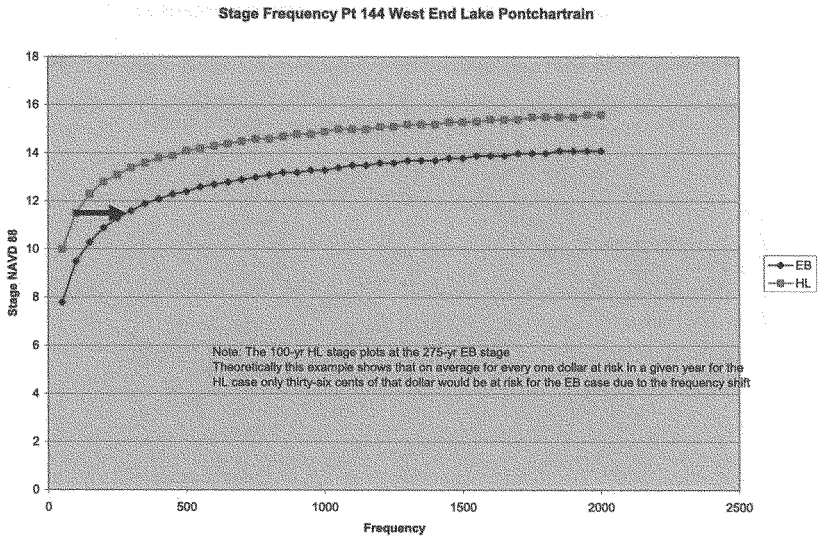

Figure 2.56 - Stage-frequency curves point 144 west end of Lake Pontchartrain

It is interesting to note that for the point locations at the eastern and western ends of the lake the 100-year 2010 base stage (Labelled HL for High Level) corresponds to about 275 to 310 year stage. The two series are labelled EB and HL in the frequency plots. For the points located on the north and south shore of Lake Pontchartrain the shift is more pronounced. The 100-year 2010 base stage corresponds to about the 475-year EB elevation on the south shore and for the North Shore point location, the frequency shift is even greater with the 100-year 2010 base stage plotting at about the 1150-year EB frequency.

The definition of risk as applied in this discussion is as follows: Annualized risk is equal to the probability of an event occurring times the dollar value of what is at risk. A purely theoretical example of a hypothetical risk that one can use to gain some insight into just what reduction in risk could occur is discussed briefly in the note placed on each plot. The risk hypothesized, assumes that if there is a $1 risk of loss associated with the 100-year 2010 base stage at each of the points shown on the map, then based purely on the shift in annual probability of occurrence, that same 100-year stage with the EB stage frequency reduces that $1 risk to about 9 cents to 36 cents depending upon the point's location within the lake. This example should not be interpreted to mean that the EB plan provides 100-year level of risk reduction to the areas around the Lake Pontchartrain since these points are actually in the lake and development around the lake perimeter varies in elevation and certainly some development would likely remain at risk with the 100-year water level associated with the EB plan.

The conclusion that one can draw from the above comparisons is that the Lake Pontchartrain surge reduction plan does afford considerable risk reduction to riparian interests around the lake's perimeter as well as to interests located within those leveed areas adjacent to the lake.

There is a potential that any changes to the hurricane risk reduction system may have an impact on the surrounding areas. Of particular interest for LACPR is the potential impact that partially closing Lake Pontchartrain may have on the adjacent Mississippi coastline. Specific work has been carried out to look at the differences between the surge frequency surfaces for the 2010 base conditions and the 12-foot weir across Lake Pontchartrain.

Figures 2.57 through **2.59** show the difference in the 100-, 400- and 1000-year still water levels outside of the barrier system.

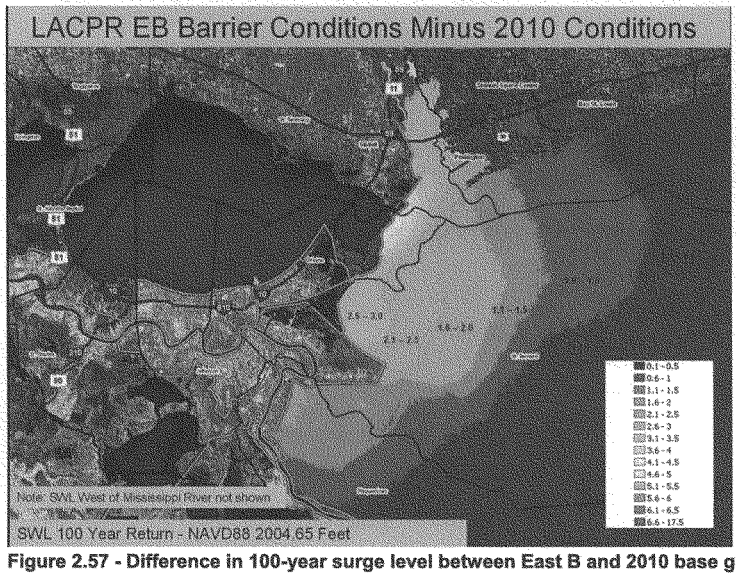

Figure 2.57 - Difference in 100-year surge level between East B and 2010 base grids

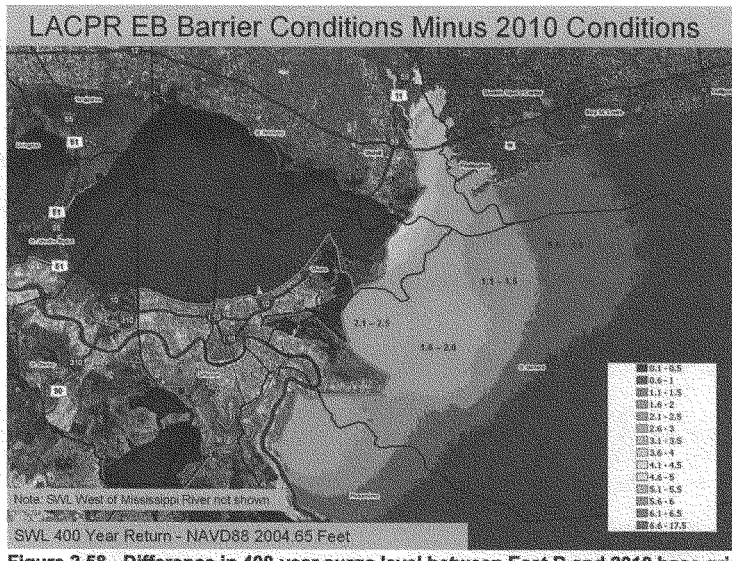

Figure 2.58 - Difference in 400-year surge level between East B and 2010 base grids

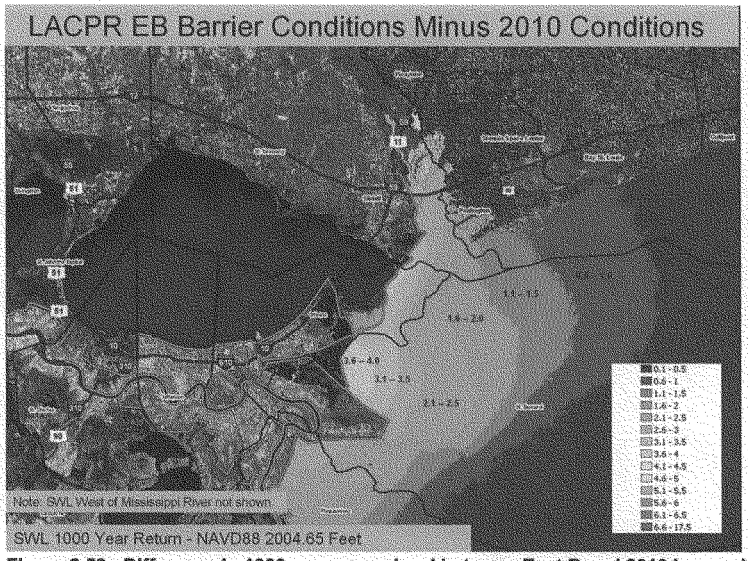

Figure 2.59 - Difference in 1000-year surge level between East B and 2010 base grids

2.6.5 Barataria Basin

A similar comparison showing stage-frequency shifts with an accompanying risk reduction can be made in Planning Unit 2 for the Weir Surge Reduction Plan. **Figure 2.60** shows point locations selected for the comparative stage frequency plots. For the Barataria Basin analysis, we again use the same nomenclature as used in the stage frequency plots for the Lake Pontchartrain analysis discussed above, i.e. HL stands for a high level levee plan (No weir, the levee alignment is shown as a red line in **Figure 2.60**) and EB stands for East Barrier (with-weir, the weir alignment is shown as a green line in **Figure 2.60**). Note that with the weir levee plan, it is assumed the "red line levee" will be in place and constructed to the 2010, 100-yr protection grade.

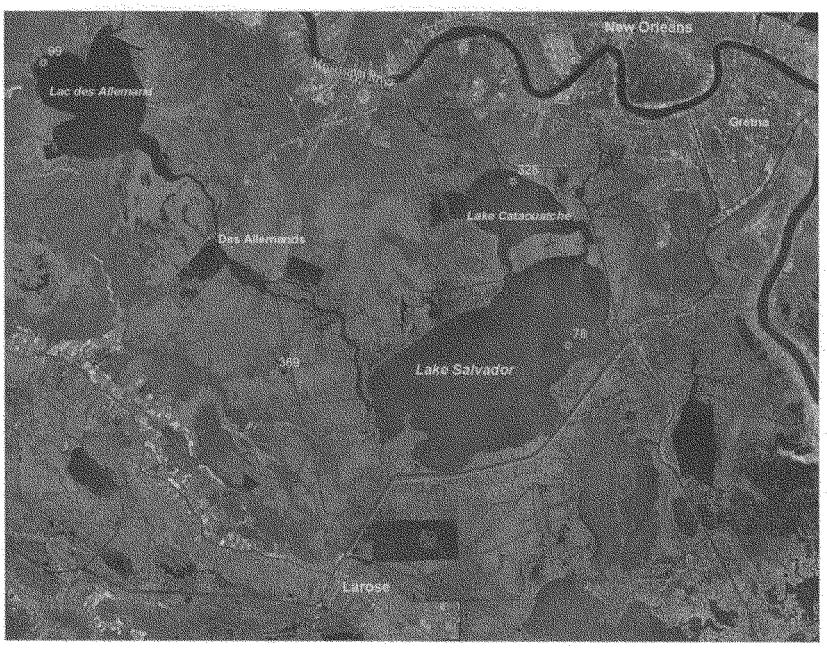

Figure 2.60 - Index map showing Locations of Point for stage-frequency curves in Barataria Basin

Comparative stage-frequency plots for the points shown in **Figure 2.60** follow in **Figures 2.61** through **2.64**. Note that in general the amount of risk reduction associated with the EB plan is greatest in the area south of Des Allemands to the protected side of weir levee. As can be seen for point 99 north of Des Allemands EB and HL curves are essentially the same curves with the EB curve being only slightly lower in elevation. Even for the HL plan, the area north of Des Allemands appears to have a much less exposure to risk from storm surges as compared to the area south of Des Allemands.

The EB plan for Planning Unit 2 provides substantial risk reduction to the areas located behind the weir. However, as with the Lake Pontchartrain Surge Reduction Plan, the blocking or confinement of the surge to the south of the EB weir in Planning Unit 2 does create higher water levels to those areas located outside the line of protection. Therefore, the application of a weir plan must also address the inducements in areas not protected by the plan. For the weir plan, the induced flooding in those areas has been taken into account by the application of both structural and non-structural plans in the areas where additional flooding is predicted to occur. Structural measures involve adding additional heights to existing levee systems impacted by the weir plan and the application of non-structural measures where no protection exists.

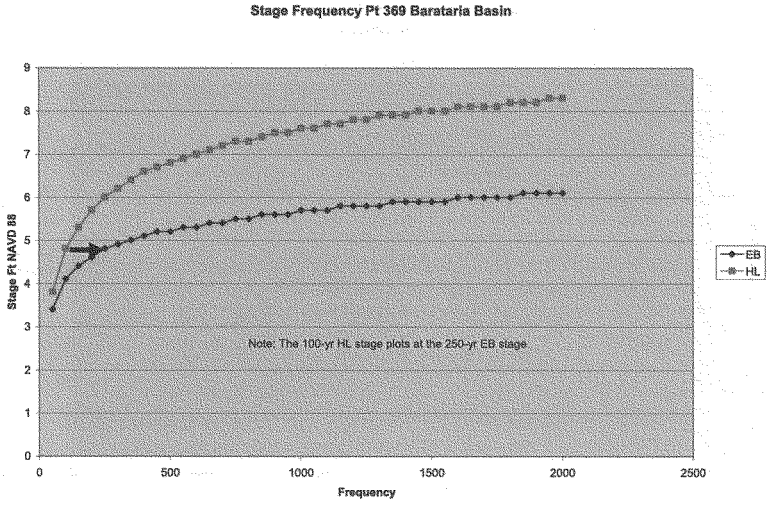

Figure 2.61 - Stage Frequency for Point 369 Barataria Basin

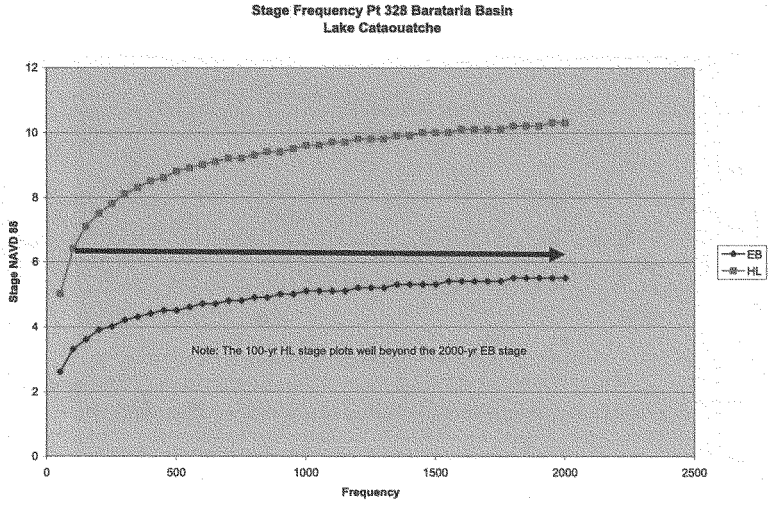

Figure 2.62 - Stage Frequency for Point 369 Lake Cataouatche

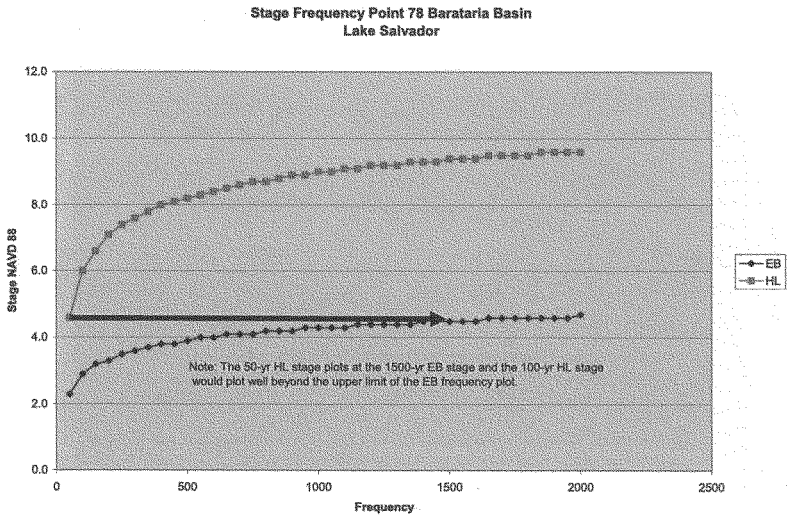

Figure 2.63 - Stage Frequency for Point 78 Lake Salvador

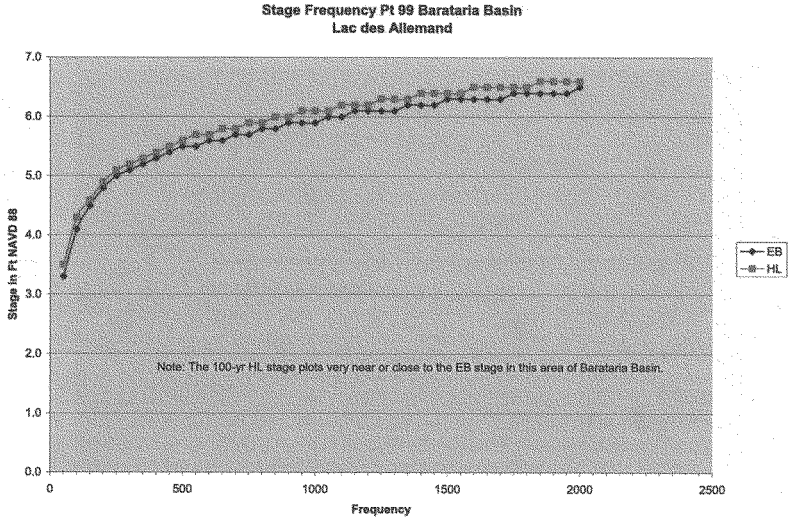

Figure 2.64 - Stage Frequency for Point 99 L

2.6.6 Conclusion

In conclusion, only the East B grid has been taken forward as an alternative to the baseline model in the analysis of differing structural alignments. This is because the impacts of the full closure on both the Mississippi coast and on constriction heights and the limited beneficial effects of the un-gated openings suggest that neither of these alternatives will provide a more sustainable and cost effective alternative in hydraulic terms.

Because of the potential adverse impacts associated with LACPR plans on adjacent areas along the Mississippi coast, a comprehensive review of the system wide affects of the Louisiana plans was undertaken to ensure that those plans such as the weir barrier plan were sufficiently analyzed to determine the potential impacts on water level frequencies elsewhere in the system. Details are presented in the System Analysis Section of the main report.

2.7 Discussion on West models

There are four primary model grids for the west:

- **Base condition**
- **West A - Continuous, non-overtopping levee** from the Larose to Golden Meadow Hurricane Protection Project to west of Vinton (near the Texas border); follows the authorized Morganza alignment with a ring levee around Berwick/Patterson, then follows the GIWW west to Vinton.
- **West B - Series of regional, non-overtopping levees**. Includes the authorized Morganza alignment with secondary alignment along the GIWW south of Houma; continuous levee from west of Patterson to Abbeville plus ring levees in Planning Unit 4
- **West C - Series of regional, non-overtopping levees plus 100-year levee along the GIWW from Abbeville to Vinton.** Includes the authorized Morganza alignment with secondary alignment along the GIWW south of Houma; continuous levee from west of Patterson to Abbeville; 100-year levee from Abbeville to south of Lake Charles with ring levee around Lake Charles and Vinton.

Whereas in the east the model grids were developed to test a number of possible alternatives so that the most appropriate one could be identified, in the west the grids were developed to specifically model significantly different levee alignments. As such the base condition, the West A and West B grids were required to represent most of the planning alternatives proposed.

The West C grid, was proposed to investigate the twin lines of defense, and also the potential increase in general risk reduction that could be achieved by introducing a 100-year levee along the GIWW. The levee was taken at a nominal 12 feet to approximate a 100-year levee. As the modeling did not account for wave overtopping, and as the number of storms that created flooding within the levees was small, the internal stages derived from the statistics were not taken as being appropriate to use in the alternative with the 12-foot levee along the GIWW and the normal interior stage frequency calculations were carried out instead.

3 COMPUTATION PROCESS FOR STAGE FREQUENCIES

3.1 Introduction

This chapter describes the general process applied to the hydrodynamic model results in order for them to be translated into the data required for hydraulic evaluation in the framework of LACPR. Section 3.2 describes the general process for the entire planning area. On some aspects the approach as applied to the planning units in the West (3a, 3b, and 4) differs from that applied in the East (Planning Units 1 and 2). Therefore, first the common approach is described, with specific attention for the deviations in East and West where applicable. In Section 3.3 the approach and values used to establish hydraulic values for the future scenarios (2060) are outlined.

3.2 Overview of approach

3.2.1 Typology of planning subunits based on their relative location

The planning subunits that make up the planning units are the level on which the evaluation of the various planning alternatives has taken place. Based on their relative location with respect to the levee system, planning subunits can be identified in three categories (**Figure 3.1**):

A external - the area is always outside of the levee system;
B internal - the area is within a current levee system and will remain inside a levee system;
C semi-internal - the area is currently outside of the levee system, but may become inside the levee system in the future.

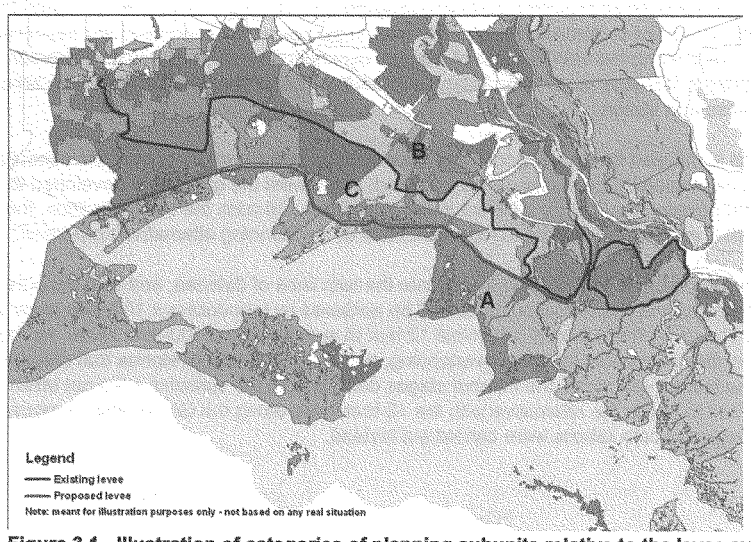

Figure 3.1 - Illustration of categories of planning subunits relative to the levee system

The determination of the stage frequency curves for the planning subunits in these categories is discussed below:

A A stage frequency relationship has been established representing the effects outside the levee systems for differing hydraulic models, as the levee alignments within the model can have an impact on conditions outside of the levees. The outside surge levels for different frequencies originate from the statistical analysis of the hydrodynamic model results. For more information about these models and the statistical method, the reader is referred to Chapter 3 and 4 in Volume I.

B The stage frequency curve has been determined representing the effects of wave/surge overtopping, rainfall and pumping. In these calculations the wave/surge overtopping volumes have been determined using the exterior waves and surge levels, and the levee characteristics. Rainfall and pumping have been included to compute the total water volume in either one or a connected series of planning subunits. The stage has been established using this volume and a single stage-storage relationship. For more information about this procedure, the reader is referred to Chapter 6 in Volume I.

C Per planning subunit, both a stage frequency for the exterior and stage frequencies from interior drainage have been developed. This category combines Category A and B.

The stage frequency curves for the planning subunits have been developed based on the evaluation of the 10-year, 100-year, 400-year, 1000-year and 2000-year hydraulic event. They have been computed for the hydraulic model alternatives (e.g. Base, EB etc), and for areas with new levees this procedure is repeated for the different levels of risk reduction (100-, 400-, and 1000-year).

East
Since in the East a levee system already exists, all three categories as described above are present in Planning Units 1 and 2. In addition, a number of special cases result in another set of internal stage frequencies. Examples of these are the introduction of the Golden Triangle alignment along the edge of Lake Borgne, or the closure of the Algiers and Harvey Canals.

West
In the West there is no current levee system. Thus, no internal areas exist in these Planning Units, and all planning subunits (and drainage areas) are external (Category A) or semi-internal (Category B).

3.2.2 Hydraulic coding system

Various planning alternatives have been assessed using the results from the modeling efforts (see Chapter 2). These planning alternatives consider new levees, different levee alignments, and a range of flood risk reduction standards. The modeling grids that were evaluated with the surge and wave models do not always match exactly with the planning alternatives in terms of levee

alignments. Hence, combinations of various modeling grids have often been applied to specify the conditions for a specific planning subunit in a specific planning alternative.

To ensure consistency in the approach, a coding system has been developed to specify which modeling results apply to the planning subunit for each planning alternative. As the models that have been used for the planning units in the East and the West differ, the hydraulic coding systems differ accordingly. These coding systems are described below:

East
The model grids that were used in the East were:

- 2010 base model grid (BS), used for:
  Planning Unit 1: the "High Level" alternatives (HL);
  Planning Unit 2: the West Bank Improvements and Ridge alignments;
- East B model grid (EB), used for:
  PU1: the Lake Pontchartrain Barrier alternatives (LP);
  PU2: the GIWW weir alternatives.

Results from the models were used to develop results for stage frequencies as indicated in **Table 3.1**.

Table 3.1 - Hydraulic coding system for the east

Hydraulic Code	Model Grid	Description
BS-ext	Baseline	Stage frequency developed from ADCIRC for areas outside of levees
EB-ext	East B	Stage frequency developed from ADCIRC for areas outside of levees
BS-xxxxa	Baseline	Stage frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'a'
BS-xxxxb	Baseline	Stage frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'b'
EB-xxxxa	East B	Stage frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'a'
EB-xxxxb	East B	Stage frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'b'

Table 3.2 explains what the various conditions represent in Planning Units 1 and 2.

Table 3.2 - Explanation of conditions 'a' and 'b' used in hydraulic codes (east)

Condition	Planning Unit 1	Planning Unit 2
a	The inner alignment around Lake Borgne (closure from B. Biennenue to Michoud Canal)	Represents the closure of the Harvey and Algiers Canals
b	The outer alignment around Lake Borgne (closure along Lake Borgne Shoreline)	Represents the Harvey and Algiers Canals being open

The planning alternatives have been coded for straightforward identification. This coding is made up of five elements:

PU1-HL-a-100-1
1 2 3 4 5

Each element for Planning Units 1 and 2 is described below.

Element 1	PU1	Planning Unit 1
	PU2	Planning Unit 2
Element 2	HL	High Level Plan Alternative (PU1)
	LP	Lake Pontchartrain Surge reduction barrier Alternative (PU1)
	WBI	West Bank Levee Alternative (PU2)
	G	GIWW Weir alternatives (PU2)
	R	Ridge alternatives (PU2)
Element 3	a	the inner alignment around Lake Borgne (closure from B. Bienvenue to Michoud Canal)
	b	The outer alignment around Lake Borgne (closure along Lake Borgne Shoreline)
	<blank>	All PU2 alternatives
Element 4	100	Alternative provides a 100-year level of risk reduction
	400	Alternative provides a 400-year level of risk reduction
	1000	Alternative provides a 1000-year level of risk reduction
Element 5	Planning Unit 1	
	1	Existing defenses including Oakville extension
	2	As 1 plus Laplace and all of the North Shore
	3	As 1 plus Laplace and Slidell only
	Planning Unit 2	
	1	Existing defenses - plus closure of Harvey and Algiers canals and Larose to Golden Meadow
	2	As 1 plus levees to Luling and Highway 90
	3	As 2 plus levees to Sunset Drainage District
	4	As 3 plus levees along Bayou Lafourche (Lockport)

Descriptions and maps of each of the LACPR alternatives are located in the Evaluation Results Appendix.

An example of the system for linking the hydraulic codes to the alternatives for the East is shown in **Table 3.3** below for some of the planning subunits in Planning Unit 1:

Table 3.3 - Example of the coding of the stage frequency results per planning subunit per alternative

Planning subunit	Planning Alternative				
	PU1-HL-a-400-1	PU1-HL-b-400-1	PU1-LP-a-400-1	PU1-LP-b-400-1	...
St Bernard Wet	BS-0400a	BS-0400b	EB-0400a	EB-0400b	...
St Bernard Dev	BS-0400a	BS-0400b	EB-0400a	EB-0400b	...
STBE_17a	BS-ext	BS-0400b	EB-ext	EB-0400b	...
ORLE_16b	BS-ext	BS-0400b	EB-ext	EB-0400b	...
STBE_14a	BS-ext	BS-ext	EB-ext	EB-ext	...
...	...	...	...	...	...

West

The model grids that were used in the West were:

- Baseline model grid, referred to as WT;
- West Model Grid A (WA), used for:
  Planning Units 3a, 3b, and 4;
- West Model Grid B (WB), used for:
  Planning Units 3a and 3b.

At the onset of the modeling the planning alternatives in the west were less well defined. This resulted in the levee alignments within the model grids not matching single planning alternatives. Therefore some combining of results from different models has been required in order to represent the planning alternatives.

Results from the models were used to develop results for stage frequencies as indicated in **Table 3.4**.

Table 3.4 - Hydraulic coding system for the west

Hydraulic Code	Model Grid	Description
WT-ext	Baseline	Stage frequency developed from ADCIRC for all areas
WA-ext	West A	Stage frequency developed from ADCIRC for areas outside of levees included in model
WB-ext	West B	Stage frequency developed from ADCIRC for areas outside of levees included in model
WT-xxxx	Baseline	Stage Frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction
WA-xxxxa	West A	Stage Frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction – condition 'a'
WA-xxxxb	West A	Stage Frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'b'
WA-xxxxc	West A	Stage Frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction - condition 'c'
WB-xxxx	West B	Stage Frequency developed from internal drainage for areas inside levees protected to the xxxx level of risk reduction

Table 3.5 explains what the various conditions represent in Planning Units 3a, 3b, and 4.

Table 3.5 - Explanation of conditions 'a,' 'b' and 'c' used in hydraulic codes (west)

Condition	Planning Unit 3a	Planning Unit 3b	Planning Unit 4
a	Continuous levee from Larose to Morgan City	Not Used	Full levee along GIWW alignment - connecting into Planning Unit 3b
b	Levee from Larose, returning to high ground to the west of Houma, plus Morgan City ring	Not Used	12-foot levee along GIWW together with localized ring levees
c	As 'a' but with a 100-year defense around Morganza and a secondary defense line	Not Used	Levee along GIWW, returning to high ground to the west of the Vermilion River

The planning alternatives have been coded for straightforward identification. This coding is made up of four elements:

PU3a-M-0100-1
1 2 3 4

Each element for Planning Units 3a, 3b and 4 is described below.

Element 1
- PU3a Planning Unit 3a
- PU3b Planning Unit 3b
- PU4 Planning Unit 4

Element 2
- M Morganza Levee - from Larose to Morgan City (PU3a)
- RL Ring Levees - used in PU3b and 4
- G GIWW alignment - used in 3a for the secondary alignment alternatives and for the alignment along the GIWW in PU3b and 4
- F Franklin alignment - used in PU3b for alignment which extends between the GIWW and the areas of risk.

Element 3
- 100 Alternative designed to provide a 100-year level of risk reduction
- 400 Alternative designed to provide a 400-year level of risk reduction
- 1000 Alternative designed to provide a 1000-year level of risk reduction

Element 4
- 1
  - PU3a Continuous levee from Larose to Morgan City
  - PU3b All alternatives
  - PU4 Continuous GIWW levee from PU3b to the west of Lake Charles (G) and ring levees (RL)
- 2
  - PU3a Levee from Larose which returns to the west of Houma to higher ground
  - PU3b Not used
  - PU4 GIWW levee which returns to the west of the Vermillion River

3 PU3a Not Used
PU3b Not Used
PU4 GIWW levee at 12' with a return and isolated ring levees

Descriptions and maps of each of the LACPR alternatives are located in the Evaluation Results Appendix.

An example of the system for linking the hydraulic codes to the planning alternatives for the West is shown in **Table 3.6** below for some of the planning subunits in Planning Unit 4:

Table 3.6 - Example of the coding of the stage frequency results per planning subunit per alternative

Planning Subunit	Planning Alternative		
	Base2007	PU4-G-0100-1	...
CALC_ 11i	WT-ext	WT-0100	...
CALC_ 5e	WT-ext	WT-0100	...
VMLN_ 10f	WT-ext	WA-0100a	...
VMLN_ 12d	WT-ext	WA-0100a	...
CALC_ 10g	WT-ext	WT-0100	...
CALC_ 10h	WT-ext	WT-0100	...
...	...	...	...

3.3 Future conditions

3.3.1 Factors affecting the future conditions

Part of the hydraulic evaluation of the planning alternatives is to consider the performance in the future (2060). A number of factors affect the hydraulic performance in the future. The following factors have been taken into account:

1. Sea level rise;
2. Subsidence;
3. Changes to the foreshore (marshes).

These factors are visualized schematically in **Figure 3.2**.

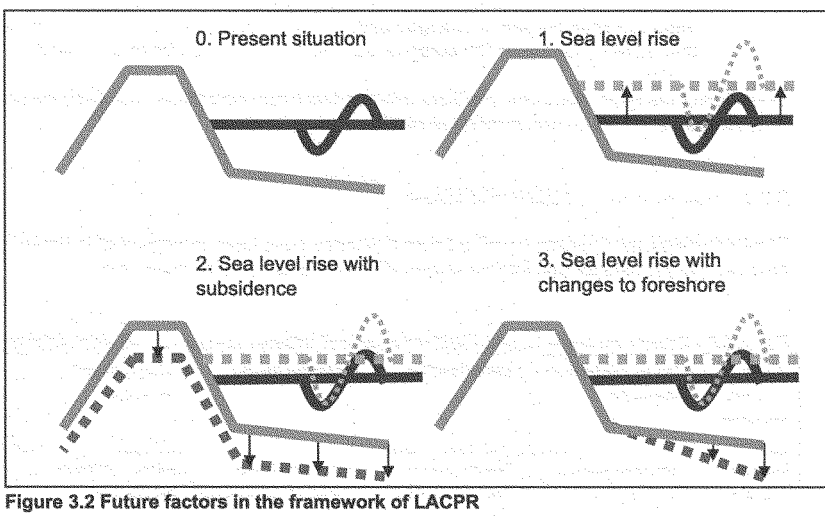

Figure 3.2 Future factors in the framework of LACPR

These factors potentially result in higher surges and wave heights, and also affect the levee heights that are required to provide a specific level of risk reduction. In the framework of LACPR the following approach has been chosen to account for these future factors:

Ad. 1,2) Sea level rise and subsidence has been allowed for in a combined value added to the surge levels. Three scenarios have been evaluated for the relative sea level rise (sea level rise + subsidence): no sea level rise, a mid range sea level rise value and a high sea level rise value. The values used for these scenarios are given in **Table 3.7**.

Table 3.7 – Relative sea level rise

Planning Unit	No sea level rise (for sensitivity analysis only)	Mid Range	High Range
1 - Pontchartrain Basin	+ 0 ft	+ 1.3 ft	+ 2.6 ft
2 - Barataria Basin	+ 0 ft	+ 1.9 ft	+ 3.2 ft
3a – Terrebonne	+ 0 ft	+ 1.9 ft	+ 3.2 ft
3b - Teche/Vermilion	+ 0 ft	+ 1.9 ft	+ 3.2 ft
4 - Mermentau	+ 0 ft	+ 1.3 ft	+ 2.6 ft

Ad. 3) Two future developments of the foreshore conditions have been evaluated. The first is the "maintain coast" condition which has been represented by the existing (2010) bathymetry assuming that the coastline will be maintained. In fact, the surge levels and the waves do not change in this foreshore condition. The second is the "degraded coastal features" condition which has been represented by the bathymetry computed by the CLEAR model. The degraded coastal features model has only been run for the 2010

base condition but with a changed bathymetry. Based on these runs, the effect of a degraded foreshore on the surge levels and the waves has been quantified.

Considering the three values for sea level rise and two future coastlines, this gives six possible alternatives for the future with a range of results.

3.3.2 Effects on levee heights and stages

The table below summarizes how the future scenarios have been incorporated in the levee heights, and the exterior and interior stages. These effects are discussed below:

Table 3.8 - Effects on levee heights and exterior and interior stages for different future scenarios

Future coastline scenarios	Future sea level rise scenarios	Levee heights	Exterior Stages	Interior stages	
				No action	Maintain levee heights
Maintain coastline	Sea Level rise 0 Sea Level rise 1 Sea Level rise 2	Present situation + effect of sea level rise scenario	Present situation + effect of sea level rise scenario	Based on present heights with increased overtopping due to sea level rise scenario	No change with present situation
Degraded coastline	Sea Level rise 0 Sea Level rise 1 Sea Level rise 2	Present situation + effect sea level rise scenario + effect degraded coastline	Present situation + effect sea level rise scenario + effect degraded coastline	Based on present heights with increased overtopping due to sea level rise scenario + effect degraded coastline	No change with present situation

The levee heights for 2060 were established as described below (see also **Table 3.8**):

Maintain coastal features.
For the maintained coast alternatives, the design heights were taken from the Base case which represents the current situation. These heights were increased based on the sea level rise values given in Table 3.7. for all Planning Units to determine the 2060 levee heights.

East - degraded coastal features
For the degraded coastal features the future degraded model results were used to compute design heights for the 'without barrier' alignments. For the 'with barrier' option the difference in levee height between the without and with barrier alternatives was computed. This value was then added to the levee heights calculated for the future degraded model to give a 'with barrier' levee height for the 2060 situation. In all cases sea level rise was applied by adding the values from **Table 3.7**.

West - degraded coastal features
For the degraded coastal features the model results were used to compute design heights for the same levee alignments as the baseline model. For the other modeled alternatives the change in levee height from the baseline model results to alternative model results was applied to the levee heights calculated for the degraded coastal features model. In all cases sea level rise was applied by adding the values from **Table 3.7** to the levee heights.

The stages for 2060 have been computed as follows (see also **Table 3.8**):

Exterior – Maintained coast
Exterior stage frequency values for the "maintained coast" were obtained by adding the sea level rise values to the external stage frequencies established for the current conditions.

Exterior – Degraded coast
For the degraded coastal features, the exterior stage frequencies have been developed from the degraded coastal model results and then increased by sea level rise. This could only be done directly for the baseline situations, because the degraded coastal features only have been evaluated for this situation. Therefore, the various alternative geometries (e.g. barrier alignment in the East or different levee alignments in the West) are treated differently. For these alternatives, the difference between the current geometry and the alternative geometry results have been applied to the degraded coastal features exterior stage frequencies of the baseline situation, and these are then adjusted for sea level rise.

Interior (East only)
For interior stage frequencies, two options have been considered: "No Action" (this is used as a baseline for the economic evaluation) and with any alternative in place. In the "No Action" plan, the levee heights are kept at a constant elevation and new overtopping rates are calculated using the degraded coastal features model, and calculating the effects on overtopping of the three sea level rise alternatives. These overtopping rates are then used to develop new stage frequency tables for those planning subunits. For the alternatives, the overtopping rate is assumed to be constant as the degree of risk reduction by the levees is maintained over time, and therefore the stage frequencies did not change.

4 PLANNING UNIT 1

4.1 Introduction

This chapter presents the results of the calculations of the levee heights and the stage frequency curves for the planning subunits of Planning Unit 1 (see **Figure 4.1**).

Section 4.2 describes the results of the levee heights being computed for the levee system in 2010 for the two different situations. The first is a levee system with a barrier at MRGO (called the High Level Plan, modeled in the 2010 base model grid); the second a levee system with closure of Lake Pontchartrain along US90, full closure of IHNC/GIWW along west shore of Lake Borgne, and a weir closure West Bank from Belle Chasse to Larose along GIWW (called the Barrier Plan, modeled in the East Model Grid B).

In section 4.3 the stage frequency curves for Planning Unit 1 are determined. For the economic evaluation both the High Level plan and the Barrier plan have been evaluated in more detail. For each, four return periods have been considered for the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). In addition, there are a number of special cases that result in additional internal stage frequencies. These are dealt with by determining a second set of internal stage frequency curves.

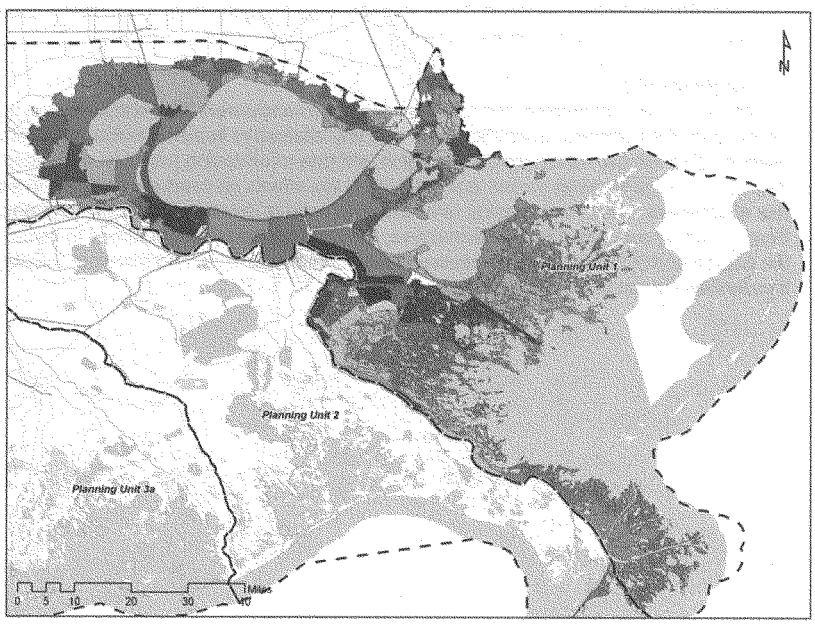

Figure 4.1 Planning subunits in Planning Unit 1

4.2 Levee height

The levee height design followed the methodologies described in Volume I of this appendix. Levee heights were computed for both the High Level Plan and the Barrier Plan for three design standards (100-year, 400-year, and 1000-year) for all levees included in the modeling. Using both the waves developed from the no bed friction model and waves from the with bed friction model has resulted in two sets of levee heights. Below, only the results of the no friction wave model are presented, thus giving the more conservative design heights.

Figure 4.2 to **Figure 4.7** show the 100-year, 400-year, and 1000-year design heights in feet for the levees in Planning Unit 1 for the High Level Plan and the Barrier Plan. The design height values have been adjusted to include any adjustments made to increase the levee heights to match the authorized levee heights. In some instances the heights were also adjusted because the results from the hydrodynamic models were not considered to be representative of the locations. A detailed overview of the design height is given in the fact sheets in Annex B.

The resulting height values have been used as input for costing design options and for establishing overtopping rates.

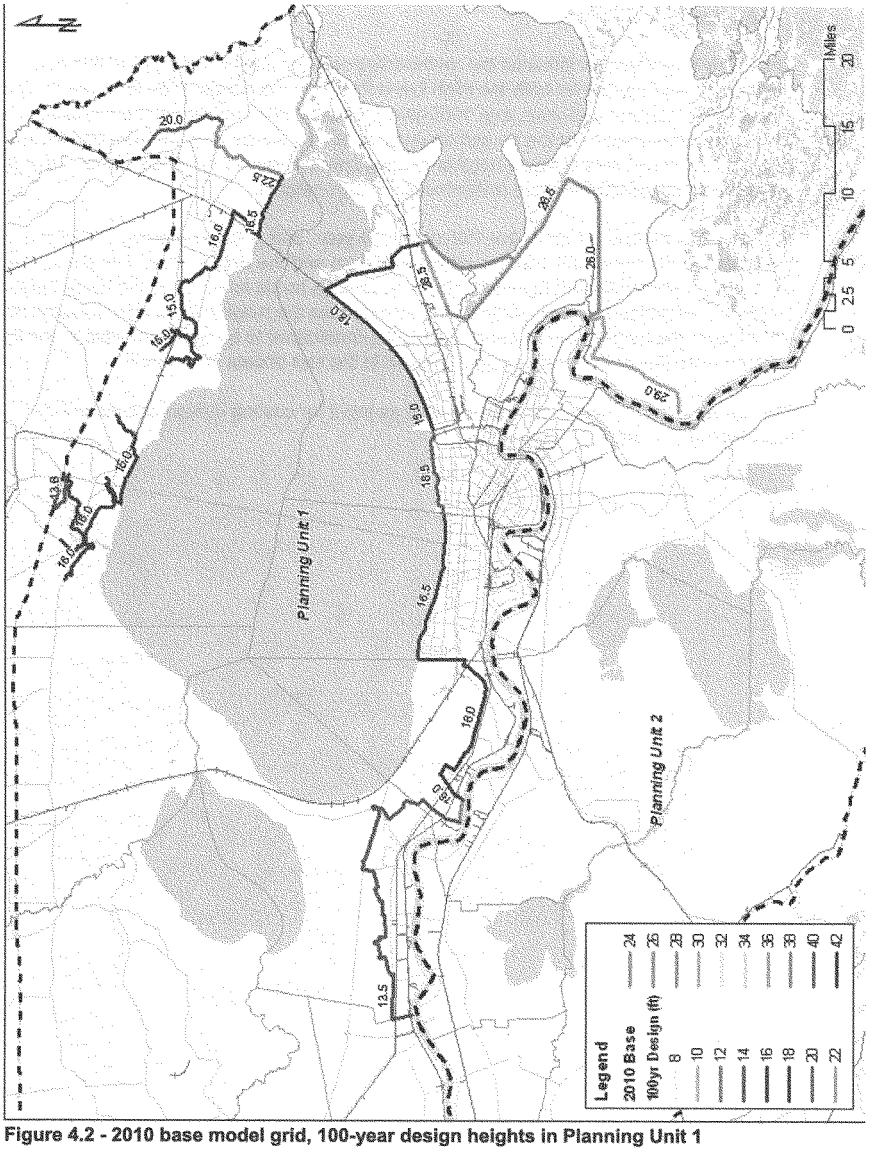

Figure 4.2 - 2010 base model grid, 100-year design heights in Planning Unit 1

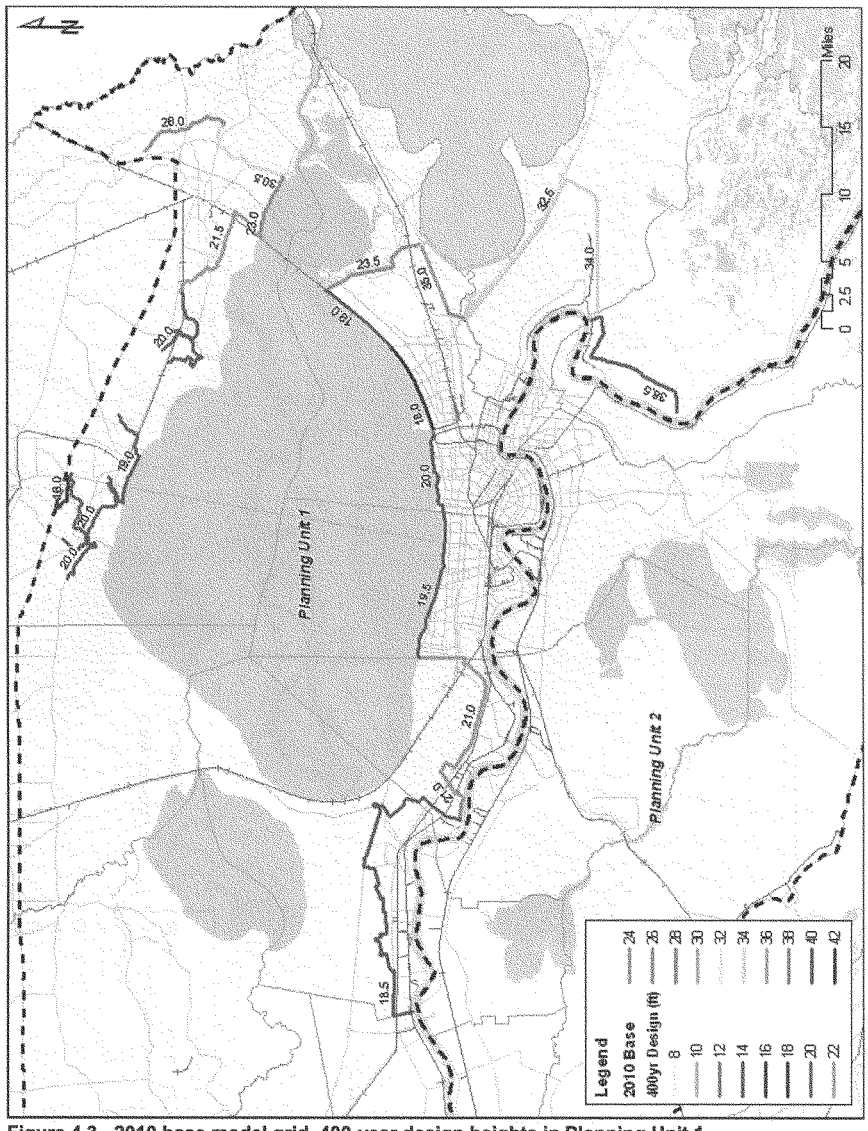

Figure 4.3 - 2010 base model grid, 400-year design heights in Planning Unit 1

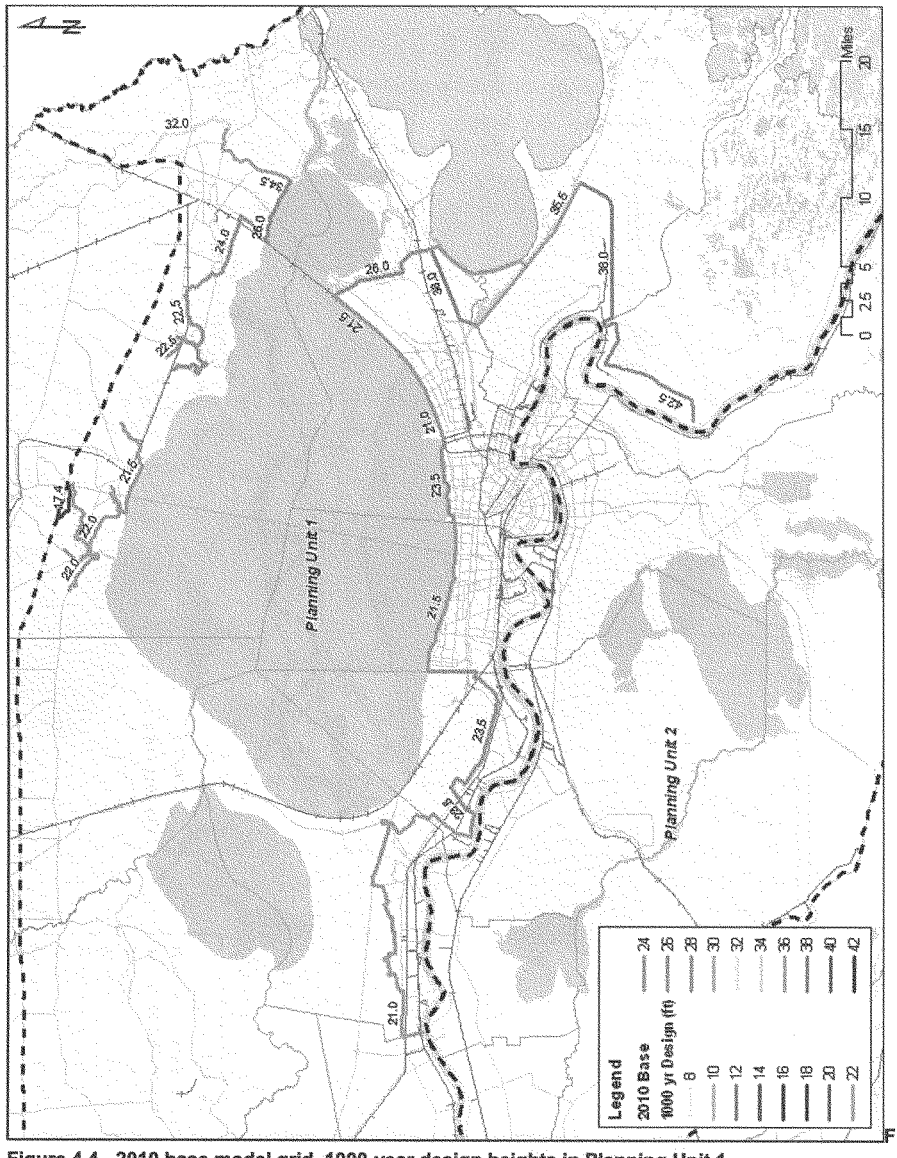

Figure 4.4 - 2010 base model grid, 1000-year design heights in Planning Unit 1

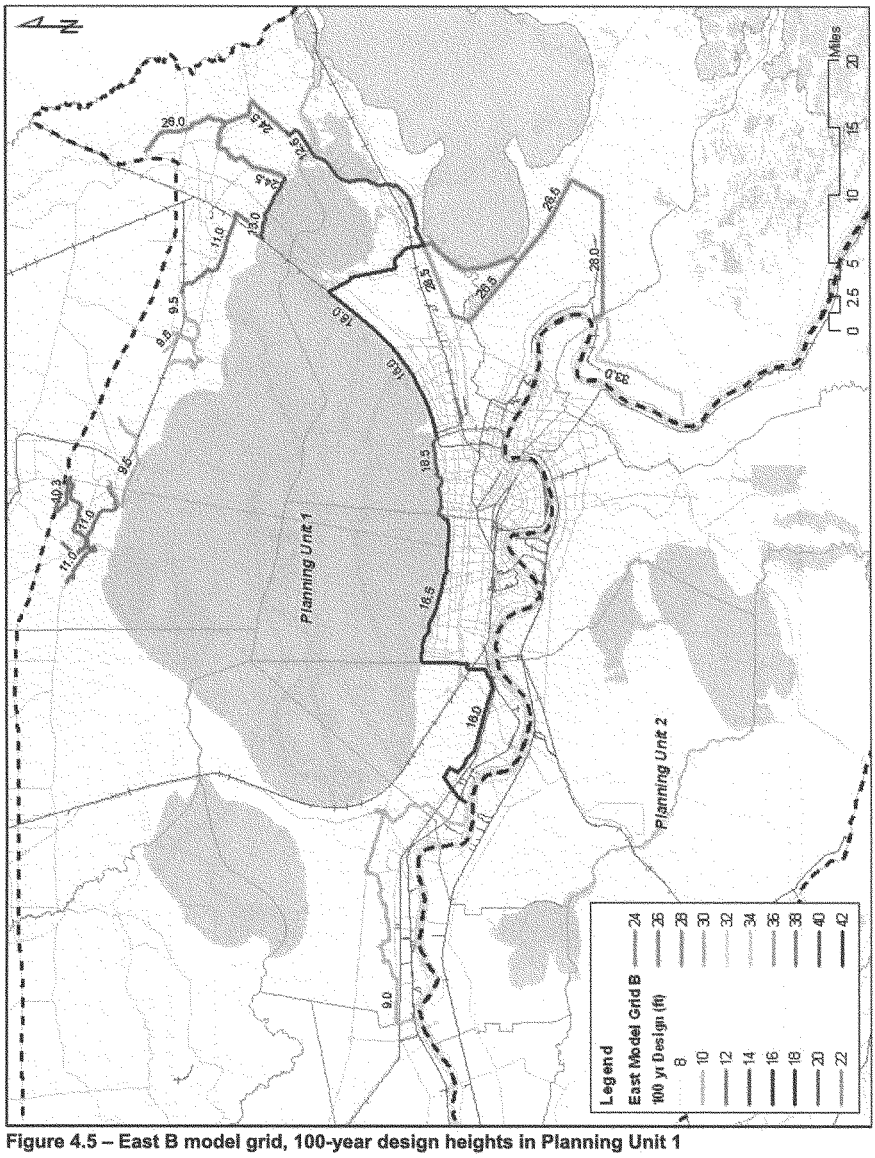

Figure 4.5 – East B model grid, 100-year design heights in Planning Unit 1

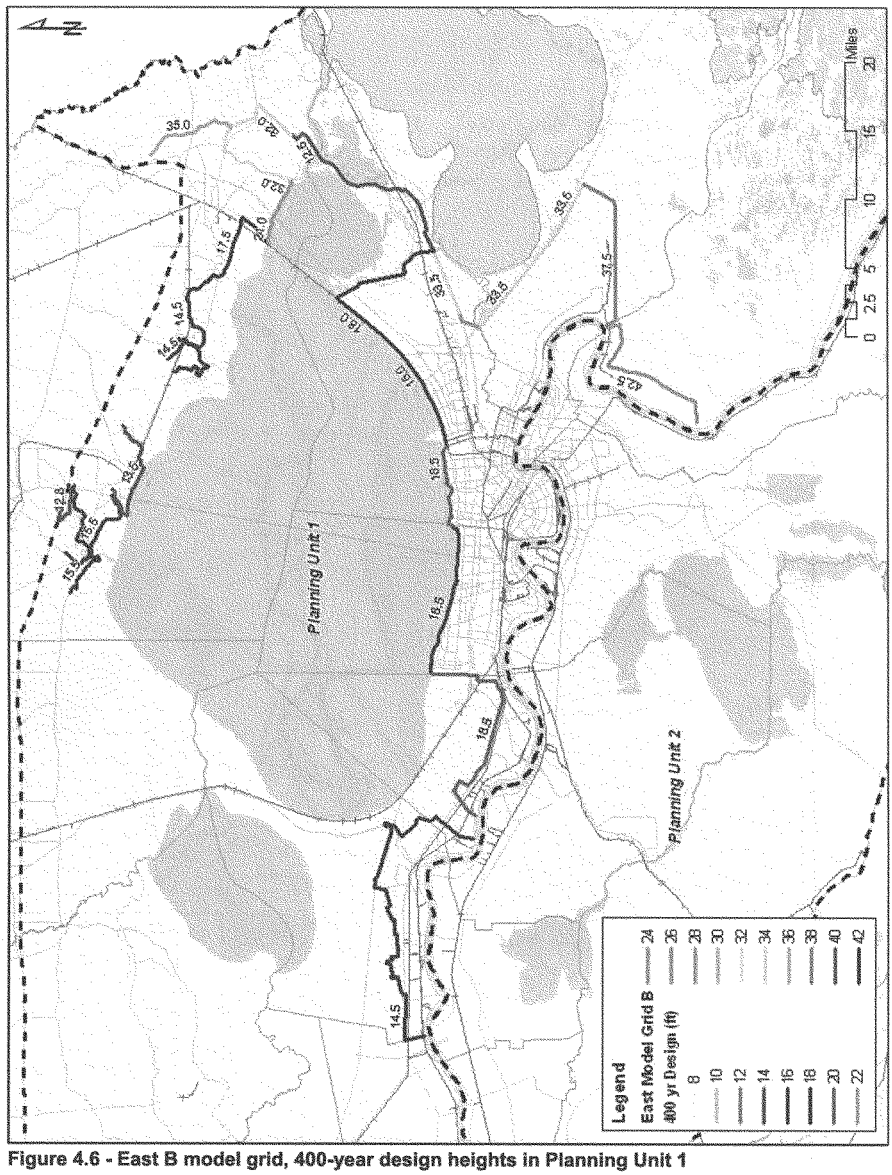

Figure 4.6 - East B model grid, 400-year design heights in Planning Unit 1

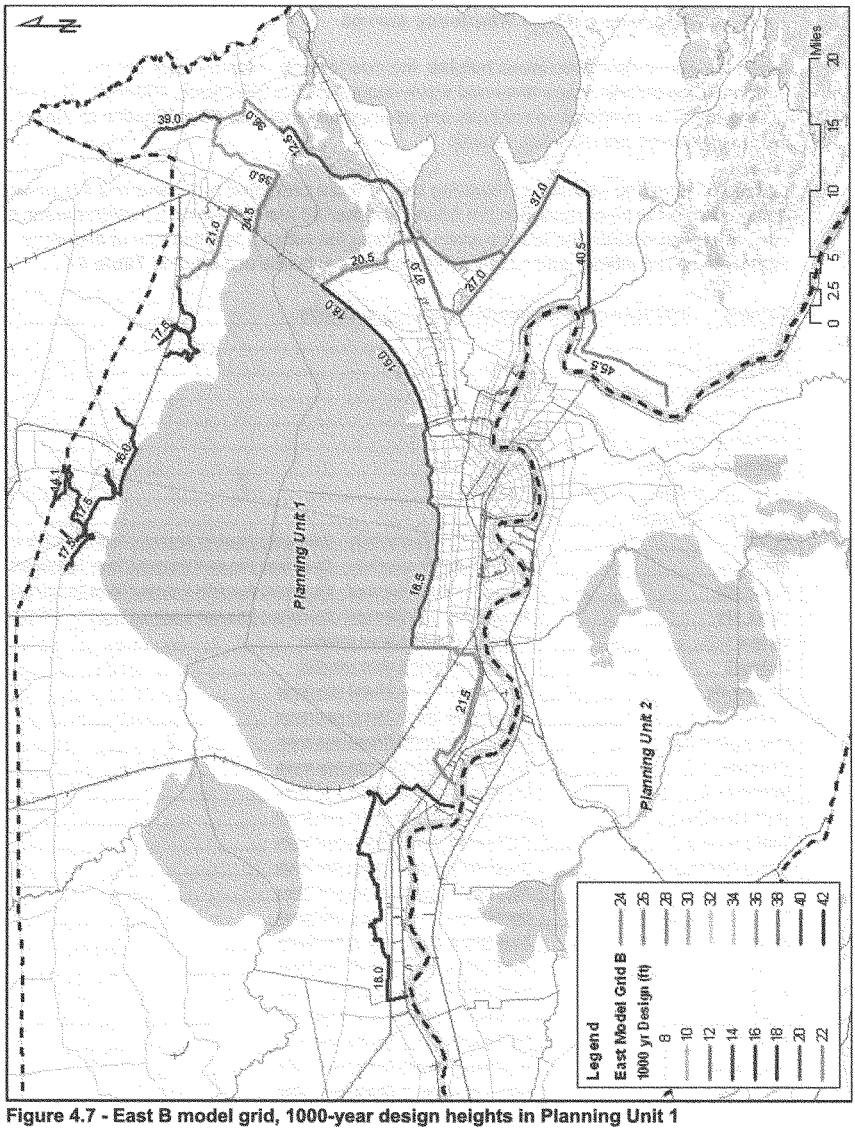

Figure 4.7 - East B model grid, 1000-year design heights in Planning Unit 1

4.3 Interior and exterior frequency curves

Given the above described levee heights, the overtopping volumes were computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). The overtopping volumes are an important input for determination of the interior stage frequency curves per planning subunit.

Of the 100 planning subunits in Planning Unit 1, 9 are protected by levees and it is proposed that these continue to be protected in the future. Another 36 are semi-interior, which means they are currently being outside the levee system, but may fall within a levee in one of the future alternatives. The interior and semi-interior planning subunits are listed in **Table 4.1**.

Table 4.1 - Interior and semi-interior planning subunits

Name	Type
New Orleans East	Interior
New Orleans - Metropolitan	Interior
East Jefferson	Interior
St Charles - Norco	Interior
St Charles - remainder	Interior
St Bernard - wetland	Interior
St Bernard - Developed	Interior
PLAQ_14s	Semi-interior (Plaquemines - Scarsdale) (requires improved levee for interior)
PLAQ_15s	Semi-interior (Plaquemines - Scarsdale) (requires improved levee for interior)
PLAQ_16s	Semi-interior (Plaquemines - Scarsdale) (requires improved levee for interior)
PLAQ_17s	Semi-interior (Plaquemines - Scarsdale) (requires improved levee for interior)
Laplace 1	Interior (no flooding when exterior)
Laplace 2	Interior (no flooding when exterior)
Madisonville_1	Semi-interior (North Shore) new levee
Madisonville_2	Semi-interior (North Shore) new levee
Madi_to_Mande	Semi-interior (North Shore) new levee
Mandeville_1	Semi-interior (North Shore) new levee
Mandeville_2	Semi-interior (North Shore) new levee
West_Lacombe_1	Semi-interior (North Shore) new levee
West_Lacombe_2	Semi-interior (North Shore) new levee
West_Lacombe_3	Semi-interior (North Shore) new levee
East_Lacombe_1	Semi-interior (North Shore) new levee
East_Lacombe_2	Semi-interior (North Shore) new levee
Slidell_1	Semi-interior (North Shore) new levee
Slidell_2	Semi-interior (North Shore) new levee
Slidell_3	Semi-interior (North Shore) new levee
Slidell_4	Semi-interior (North Shore) new levee
Slidell_5	Semi-interior (North Shore) new levee
Oak_Harbour	Semi-interior (North Shore) new levee
STTA_10e	Semi-interior (North Shore) new levee
STTA_11d	Semi-interior (North Shore) new levee
STTA_10i	Semi-interior (North Shore) new levee

Name	Type
STTA_12b	Semi-interior (North Shore) new levee
STTA_12c	Semi-interior (North Shore) new levee
ORLE_15a	Semi-interior (Golden Triangle) only interior with new levee alignment
ORLE_16b	Semi-interior (Golden Triangle) only interior with new levee alignment
ORLE_17b	Semi-interior (Golden Triangle) only interior with new levee alignment
STBE_16a	Semi-interior (Golden Triangle) only interior with new levee alignment
STBE_17a	Semi-interior (Golden Triangle) only interior with new levee alignment

For each planning subunit, **Table 4.2** lists the model grids that have been used, as well as the application of the special cases. The special cases are described following the table. For those cases where several planning subunits are located within a single drainage area the name of the drainage area is listed.

Table 4.2 - Planning subunit to hydraulic model grid matrix

	Hydraulic Code													
Planning Subunit	BS-ext	BS-0100a	BS-0100b	BS-0400a	BS-0400b	BS-1000a	BS-1000b	EB-ext	EB-0100a	EB-0100b	EB-0400a	EB-0400b	EB-1000a	EB-1000b
Laplace	x	x		x		x		x	x		x		x	
St Charles Norco		x		x		x			x		x		x	
St Charles - rest		x		x		x			x		x		x	
East Jefferson		x	x	x	x	x	x		x	x	x	x	x	x
New Orleans Metro		x	x	x	x	x	x		x	x	x	x	x	x
New Orleans East		x	x	x	x	x	x		x	x	x	x	x	x
St Bernard Wetland		x	x	x	x	x	x		x	x	x	x	x	x
St Bernard developed		x	x	x	x	x	x		x	x	x	x	x	x
Plaquemines - Scarsdale	x	x		x		x		x	x		x		x	
Golden Triangle	x		x		x		x	x		x		x		x
Madisonville	x	x		x		x		x	x		x		x	
Madisonville to Mandeville	x	x		x		x		x	x		x		x	
South Covington	x	x		x		x		x	x		x		x	
Mandeville	x	x		x		x		x	x		x		x	
West Lacombe	x	x		x		x		x	x		x		x	
East Lacombe	x	x		x		x		x	x		x		x	
Slidell	x	x		x		x		x	x		x		x	
Other External areas	x							x						
	x = stage frequency calculated													

The hydraulic model grid codes at the top of the table columns relate to the various layout alternatives and corresponding sources of information; below these are listed for the 100-year design options:

BS-ext - exterior stage frequency results from the ADCIRC modeling for grid BS;
EB-ext - exterior stage frequency results from the ADCIRC modeling for grid EB;
BS-0100a - interior stage frequency results developed for a 100-year design option - alternative a (see **Table 4.3** below) based on the BS model grid;
BS-0100b - interior stage frequency results developed for a 100-year design option - alternative b (see **Table 4.3** below) based on the BS model grid;
EB-0100a - interior stage frequency results developed for a 100-year design option - alternative a (see **Table 4.3** below) based on the BS model grid;
EB-0100b - interior stage frequency results developed for a 100-year design option - alternative b (see **Table 4.3** below) based on the BS model grid.

The 0400 and 1000 codes refer to the 400-year and 1000-year design options.

Table 4.3 - Planning subunit interior stage frequency alternatives

Planning Subunit	Condition 'a'	Condition 'b'
Laplace	with levee	N/A
St Charles Norco	baseline	N/A
St Charles – rest	baseline	N/A
East Jefferson	baseline	effects of Golden Triangle alignment
New Orleans Metro	baseline	effects of Golden Triangle alignment
New Orleans East	baseline	effects of Golden Triangle alignment
St Bernard Wetland	baseline	effects of Golden Triangle alignment
St Bernard developed	baseline	effects of Golden Triangle alignment
Plaquemines - Scarsdale	with levee	N/A
Golden Triangle	N/A	with levee along Lake Borgne
Madisonville	with levee	N/A
Madisonville to Mandeville	with levee	N/A
South Covington	With levee	N/A
Mandeville	with levee	N/A
West Lacombe	with levee	N/A
East Lacombe	with levee	N/A
Slidell	with levee	N/A
Other External areas	N/A	N/A

Comments on the specific planning subunits are given in **Table 4.4**. Additional area specific information is given in Annex B.

Table 4.4 - Specific planning subunit comments

Planning Subunit	Comment
Laplace	This is considered as initially without a levee system and therefore exterior. Several of the options (both without and with the Lake Pontchartrain barrier) consider the construction of a levee and so the effect on stage frequency has been calculated with a range of different design standards of a levee protecting Laplace.
St Charles - east bank	The two planning subunits within St Charles Parish on the east bank are within an existing levee system and have been considered as interconnected above a flood level of 4.5'.
East Jefferson New Orleans Metro New Orleans East St Bernard Wetland St Bernard Developed	The central area of New Orleans has been considered as an interconnected system. The following assumptions have been made in developing the stage frequency results in this area. St Bernard wetland and St Bernard Developed are inter-linked at 10.5'. For flood levels greater than 12.5' water flows from St Bernard into New Orleans Metro. Similarly if flooding in New Orleans East exceeds 12.5' then water flows into New Orleans Metro. Water flows between New Orleans Metro and East Jefferson at flood levels greater than 5'. A maximum level of 16' has generally been used within the central New Orleans area. In addition, the effects of creating a levee along the edge of Lake Borgne (the Golden Triangle alignment) were considered by increasing the storage in St Bernard wetland and reducing the overtopping lengths to New Orleans East. Also the design height of this levee, together with overtopping rates, was evaluated. The evaluation of this option resulted in the 'b' alternatives in the matrix above.
Golden triangle	There are a number of areas that have an interior stage frequency when the Golden Triangle alignment is implemented. These areas have the same stage frequency as St Bernard Wetland and are presented in the 'b' alternatives.
Plaquemines - Scarsdale	There are a number of areas within the existing low levee system south of St Bernard Parish that have been included using exterior stage frequency values for the baseline case, and have then been analyzed with improved levee systems at a range of design standards to provide interior stage frequency results.
Slidell	This area covers the Slidell part of the North Shore. Some parts of the existing levee system have been modeled and the results from the hydrodynamic modeling have been used and the impacts have been considered for low frequency events. The 'a' alternatives consider the creation of a new levee around the areas which then links up with a further levee to the west (see below).
Madisonville, Madisonville to Mandeville, Mandeville, South Covington, West and East Lacombe	These areas cover the North Shore to the west of Slidell and use the exterior stage frequency results from the hydrodynamic modeling for the current no levee situation. Interior stage frequencies have been developed as 'a' alternatives using both the BS and EB grids and with a range of design standards.

4.4 Interconnected drainage areas

Two groups of planning subunits have been considered as interconnected during severe flood events. Details on how these areas have been linked are given below.

4.4.1 St Charles

The planning subunits St Charles Norco and St Charles-rest are considered as connected, with the link being at 4.5 feet. If both areas have a stage of less than 4.5 feet, then the stages are used from the system. If one is greater than 4.5 feet, then the excess flood volume is added to the other and a new stage is computed. If both stages then reach a level higher than 4.5 feet, the total volume is used with the total stage storage to give a single value of stage. This value is then limited to the higher of the authorized height (13 feet), the design height (32) or the surge (32).

4.4.2 New Orleans

There are five areas that are considered to be linked within the New Orleans area. These are two in St Bernard, New Orleans East, New Orleans Metro, and East Jefferson. A flow chart of this linkage is shown in **Figure 4.8**.

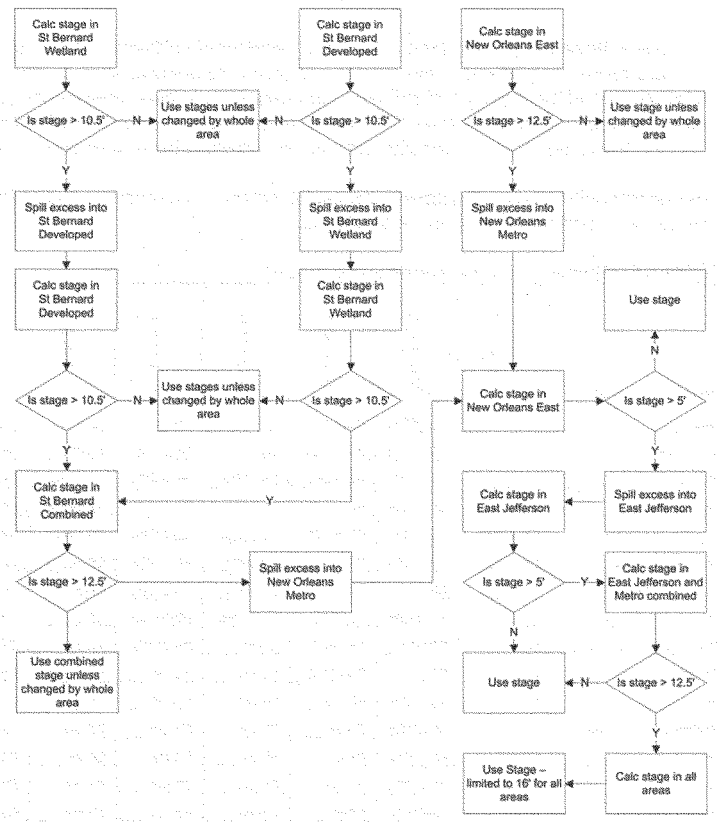

Figure 4.8 – Flow chart showing linkages between New Orleans drainage areas

The key points for this system are that there is no connection between New Orleans East and St Bernard until the New Orleans Metro/East Jefferson areas have filled to 12.5 feet as well.

4.5 Hydraulic results to planning alternatives

At the initial planning stage, a large number of planning alternatives have been considered. These cover the two basic alternatives:

- High Level plan: raising and extending the existing levee system;
- Barrier plan: construction of a barrier of some sort across the entrance to Lake Pontchartrain together with levee improvements.

In addition, three design standards have been considered (100-year, 400-year, and 1000-year) as well as a range of variations in alignments and areas. All of these alternatives are used for the determination of the interior and exterior stage frequencies, thus providing results that are the input for the economic evaluation. Following the screening process carried out by the LACPR planning team, eight alternatives and the baseline situation have been considered in more detail.

The relationship between the planning subunits, selected alternatives, and hydraulic codes (as discussed in section 4.2) is given in **Table 4.5**.

Table 4.5 - Planning subunit to alternative matrix

Planning Subunit	2010_base	HL-a-100-3	HL-b-400-3	LP-a-100-1	LP-a-100-3	LP-b-400-1	LP-b-400-3	LP-b-1000-1	LP-b-1000-2
Laplace	BS-ext	BS-0100a	BS-0400a	EB-ext	EB-0100a	EB-ext	EB-0400a	EB-ext	EB-1000a
St Charles Norco	BS-0100a	BS-0100a	BS-0400a	EB-0100a	EB-0100a	EB-0400a	EB-0400a	EB-1000a	EB-1000a
St Charles – rest	BS-0100a	BS-0100a	BS-0400a	EB-0100a	EB-0100a	EB-0400a	EB-0400a	EB-1000a	EB-1000a
East Jefferson	BS-0100a	BS-0100a	BS-0400b	EB-0100a	EB-0100a	EB-0400b	EB-0400b	EB-1000b	EB-1000b
New Orleans Metro	BS-0100a	BS-0100a	BS-0400b	EB-0100a	EB-0100a	EB-0400b	EB-0400b	EB-1000b	EB-100[illegible]
New Orleans East	BS-0100a	BS-0100a	BS-0400b	EB-0100a	EB-0100a	EB-0400b	EB-0400b	EB-1000b	EB-100[illegible]
St Bernard Wetland	BS-0100a	BS-0100a	BS-0400b	EB-0100a	EB-0100a	EB-0400b	EB-0400b	EB-1000b	EB-1000b
St Bernard developed	BS-0100a	BS-0100a	BS-0400b	EB-0100a	EB-0100a	EB-0400b	EB-0400b	EB-1000b	EB-1000b
Plaquemines - Scarsdale	BS-ext	BS-0100a	BS-0400a	EB-ext	EB-0100a	EB-ext	EB-0400a	EB-ext	EB-1000a
Golden Triangle	BS-ext	BS-ext	BS-0400b	EB-ext	EB-ext	EB-0400b	EB-0400b	EB-1000b	EB-1000b
Madisonville	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
Madisonville to Mandeville	BS-ext	BS-0100b	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
South Covington	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
Mandeville	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
Madisonville	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
West Lacombe	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
East Lacombe	BS-ext	BS-0100a	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-1000a
Slidell	BS-ext	BS-0100a	BS-400a	EB-ext	EB-0100a	EB-ext	EB-0400a	EB-ext	EB-1000a
Other External areas	BS-ext	BS-ext	BS-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext	EB-ext

The alignments and levee heights for each of the alternatives are shown on the summary maps in the Evaluation Results Appendix. These maps all go along with an accompanying table showing stage frequency comparisons (baseline to alternative) for a selection of planning subunits.

4.6 Plaquemines

This section of the report is primarily focused on the determination of interior stages for the existing/authorized Plaquemines levees. The existing levees from Oakville to Venice (the Plaquemines area) were originally not included in the levee improvements. However since the potential alternatives may affect the standard of risk reduction afforded to this area, the heights of the levees and stage frequencies needed to be established.

An overview of the levees and storage areas is shown in **Figure 4.9** below. The existing levee heights (authorized as well as non-federal) as shown in the map and in **Table 4.6** were obtained from the USACE design memorandum for each of the reach of the Mississippi River and Tributaries Project and/or the New Orleans to Venice Hurricane Protection Project.

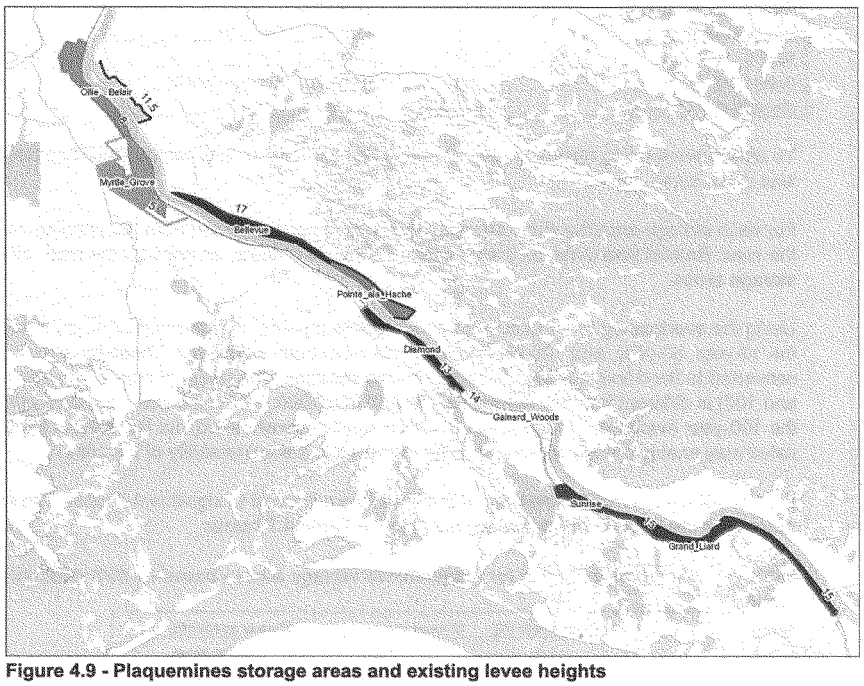

Figure 4.9 - Plaquemines storage areas and existing levee heights

Table 4.6 - Authorized levee heights

Location	Height in feet	Notes
Belair	11.5	Non-federal
Ollie	8	Non-federal
Myrtle Grove	5	Non-federal
Bellevue	17	Authorized - Reach C
Point ala Hache	17	Authorized - Reach C
Diamond - upper	12.8	Authorized - Reach A
Diamond - lower	13	Authorized - Reach A
Diamond - Mississippi	17	Authorized
Gainard Woods - west	13.5 – 14.5	Authorized - Reach A
Gainard Woods - Mississippi	17	Authorized
Sunrise - west	15	Authorized - Reach B1
Sunrise - Mississippi	17	Authorized
Grand Liard - west	15	Authorized - Reach B1 and B2
Grand Liard - Mississippi	17 – 16	Authorized

To obtain overtopping rates the levees were linked to model output points. The 2010 base model was used, here.

For each storage area the stage storage values have been obtained from the previous modeling of the area. Rainfall was taken as the standard 6.5 inch in 6 hours, as used for the main internal storage areas.

Using the overtopping rates calculated for the points together with the rainfall, flood volumes for the 10-year, 50-year, and 100-year flood events have been calculated. These have then been converted to flood levels using the stage storage volumes. The range of events considered (10, 50 and 100) is different from the normal set (10,100,400,1000,2000) as it was considered that over the 100-year event the storage areas would be totally overwhelmed, but that having the 50-year value may enable some conclusions to be drawn as to the vulnerability of the area.

For the 100-year event the majority of the areas were flooded to significant depths. The stage values (for the 50% confidence level) are listed in **Table 4.7** below:

Table 4.7 - Stage values in feet for Plaquemines storage areas - based on 50% confidence level results

Area	10-year event (ft)	50-year event (ft)	100-year event (ft)
Bellevue	1.0	1.9	Full to levee
Pointe ala Hache	1.6	2.6	Full to levee
Diamond	0.1	0.1	Full to levee
Gainard Woods	-1.1	-1.1	9.4
Sunrise	-2.3	-1.9	9.7
Grand Liard	-4.7	-4.6	1.9
Ollie	1.4	1.4	Full to levee

The values for 400-, 1000-, and 2000-year events have been taken from the adjacent exterior stage frequency curves as it was assumed that the levees have only a minor effect on flood levels at these frequencies. For the 'with barrier' option changes in exterior stage frequency were applied for the higher return period events using the east model B results.

To take into account the impacts of implementing the barrier across Lake Pontchartrain and the Barataria Basin, the increase in surge levels along the Plaquemines was investigated.

The differences in surge heights along the levees was obtained for the without and with barrier options across Lake Pontchartrain and along the GIWW in the Barataria Basin (in Planning Unit 2) by comparing the surge levels from the High Level Plan and east model B results. The differences in surge level were taken as the changes required to increase the levee height to maintain the same standard of risk reduction.

The analysis suggested the following changes to levee heights:

- Ollie - increase levee heights by 2.6 feet to accommodate the maximum change in surge seen at 100-year;
- Bellevue - increase levee heights by 0.6 feet to accommodate the increase in surge level;
- Pointe a la Hache - increase levee heights by 0.3 feet to accommodate the increase in surge level;
- Diamond - Mississippi Levee - increase levee heights by 0.3 feet to accommodate the increase in surge level;
- Gainard Woods - Mississippi Levee - increase levee heights by 0.2 feet to accommodate the increase in surge level;
- Belair - at Belair the increase is around 2 feet, but the non-federal levee is overtopped at less than once in 50 years (1.8 feet still water over levee at 1 in 50 year event). Because of this low defense standard, no allowance was made for increases in this levee.
- Myrtle Grove - no allowances have been investigated for Myrtle Grove as the levee height has been taken as around 5 feet and assumed to be overtopped regularly.

No changes were made to the interior stage frequency values for storage areas in Plaquemines for future conditions as the changes were deemed to affect only a relatively small area and would only have a minor impact on the final results of the overall evaluation of alternatives developed for the planning unit. Since the storage areas behind the existing levees are already mostly totally overwhelmed for the 2010 Base condition, it was viewed that calculating new stage frequency values for future H&H conditions would result in little change under future conditions since the only change would be to the interior stages in the storage areas that were not overwhelmed under the 2010 Base conditions (a relatively small area). Developing new interior stage frequency values for these remaining areas would have been a very tedious effort for only minor impacts on the final

results. Therefore no changes were made. For exterior stage frequencies, which were also used in damage calculations for the areas in the Plaquemines where the interior storage areas were previously determined to be overwhelmed for the 2010 base conditions, the relative sea level rise values were added to the 2010 base stages for future conditions.

5 PLANNING UNIT 2

5.1 Introduction

Following the same structure as the previous chapter on Planning Unit 1, this chapter presents the levee heights and the stage frequency curves resulting from the calculations for the planning subunits of Planning Unit 2 (see **Figure 5.1**).

Section 5.2 describes the results of the levee heights being computed for both the 2010 base model grid and the east hydro model grid B, for three design standards (100-year, 400-year, and 1000-year), over all levees included in the modeling.

In section 5.3 the stage frequency curves for Planning Unit 2 are determined.

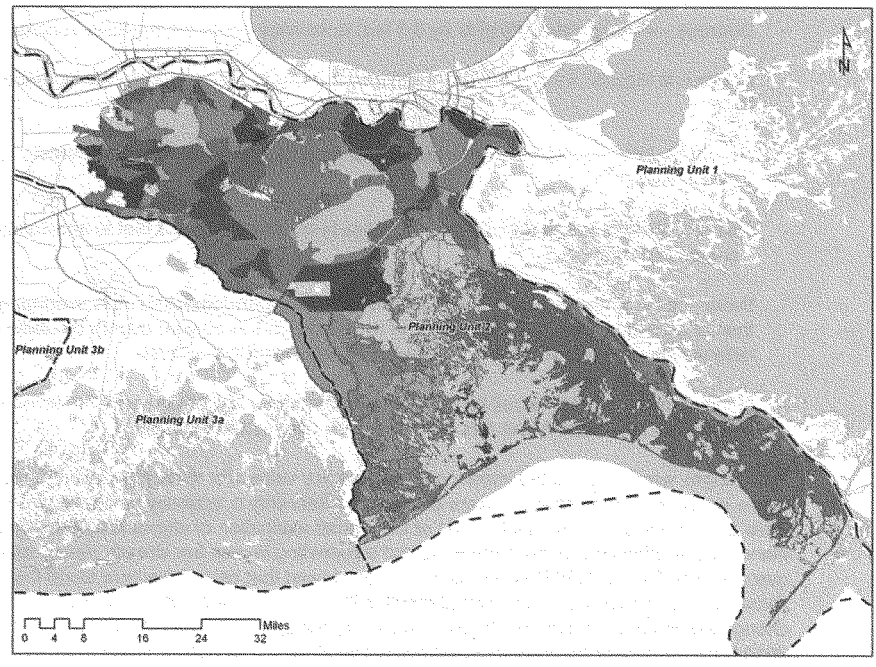

Figure 5.1 – Planning subunits in Planning Unit 2

5.2 Levee heights

The levee height design was carried out using the methodologies described in Volume I of this report. Levee heights were computed for both the 2010 base model grid and the east model B, for three separate design standards (100-year, 400-year, and 1000-year) for all levees included within the modeling.

Figures 5.2 through **5.7** show the 100-year, 400-year, and 1000-year design heights in feet for the levees within Planning Unit 2 for the 2010 base grid and the east hydro model B grid. These values have been adjusted to include any changes made for authorized heights or because of issues with specific modeling results. A detailed overview of the design heights is given in the fact sheets in Annex B.

The heights are again used for the purposes of costing design options and for establishing overtopping rates.

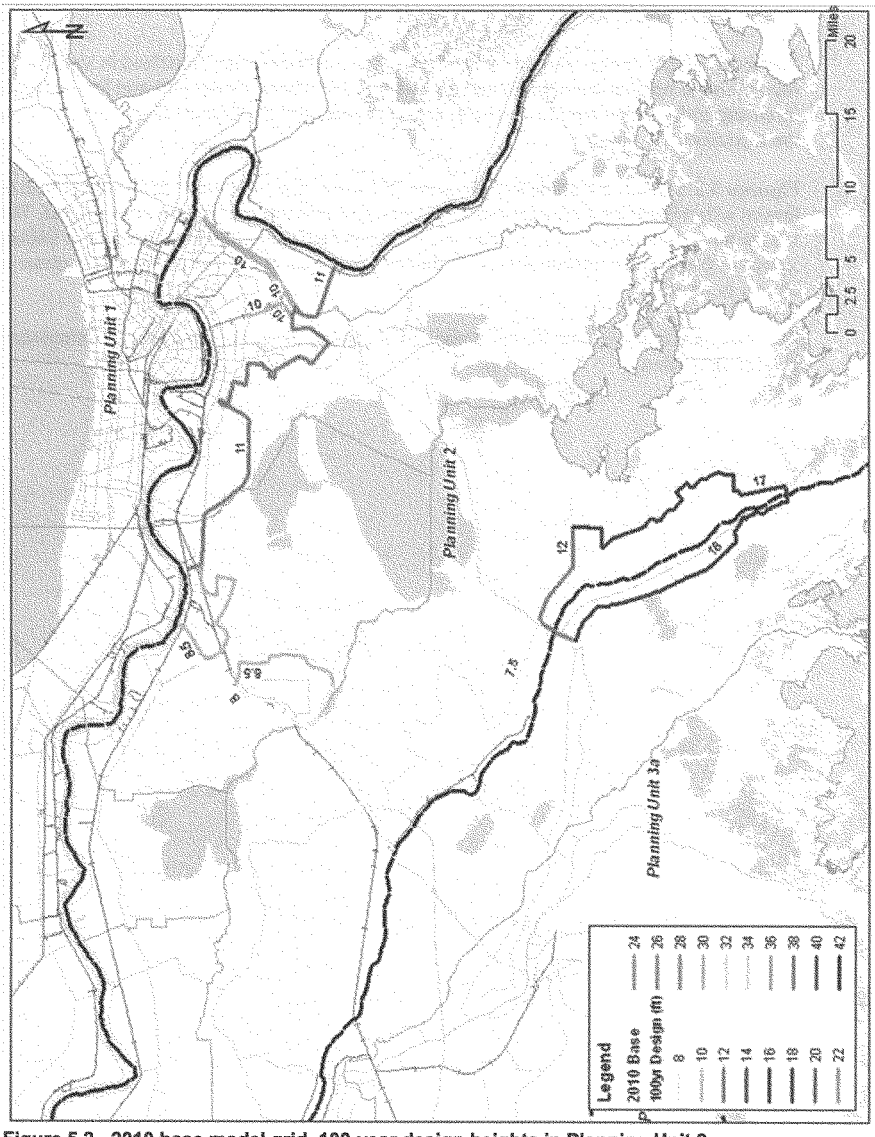

Figure 5.2 - 2010 base model grid, 100-year design heights in Planning Unit 2

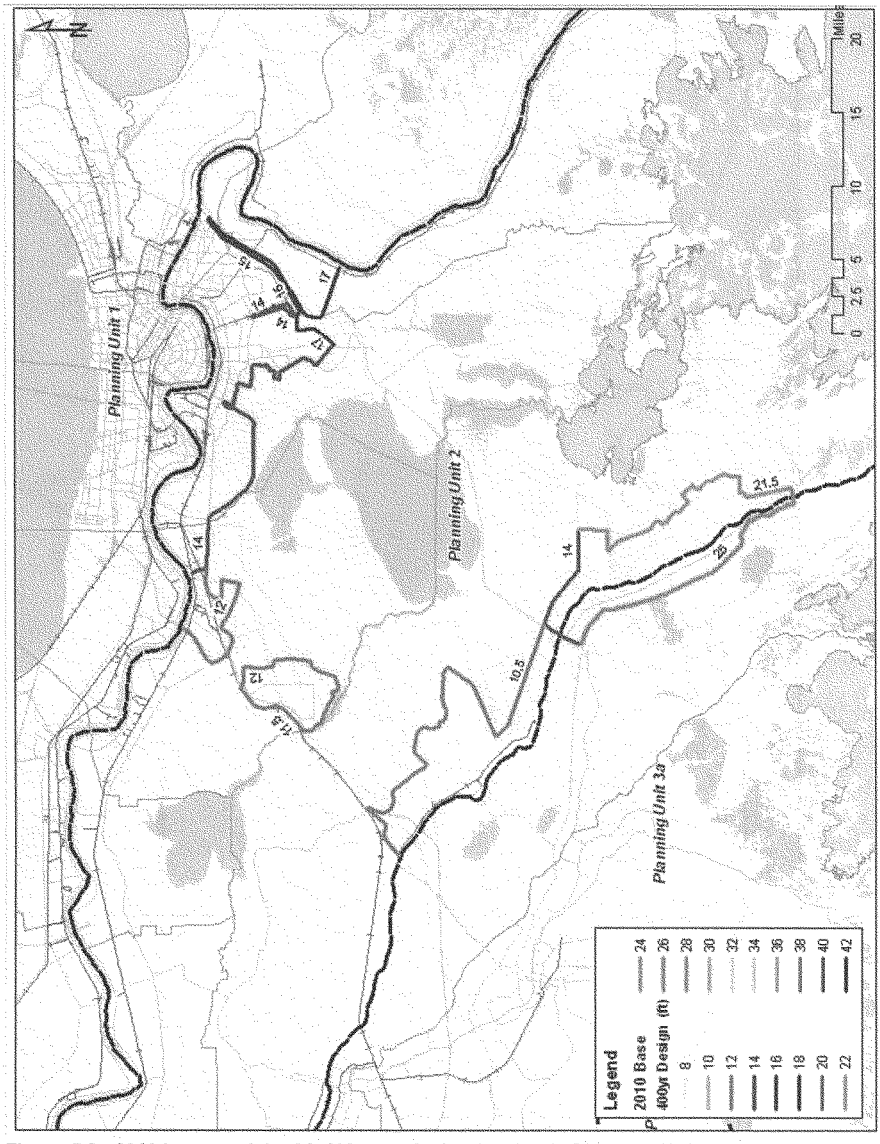

Figure 5.3 - 2010 base model grid, 400-year design heights in Planning Unit 2

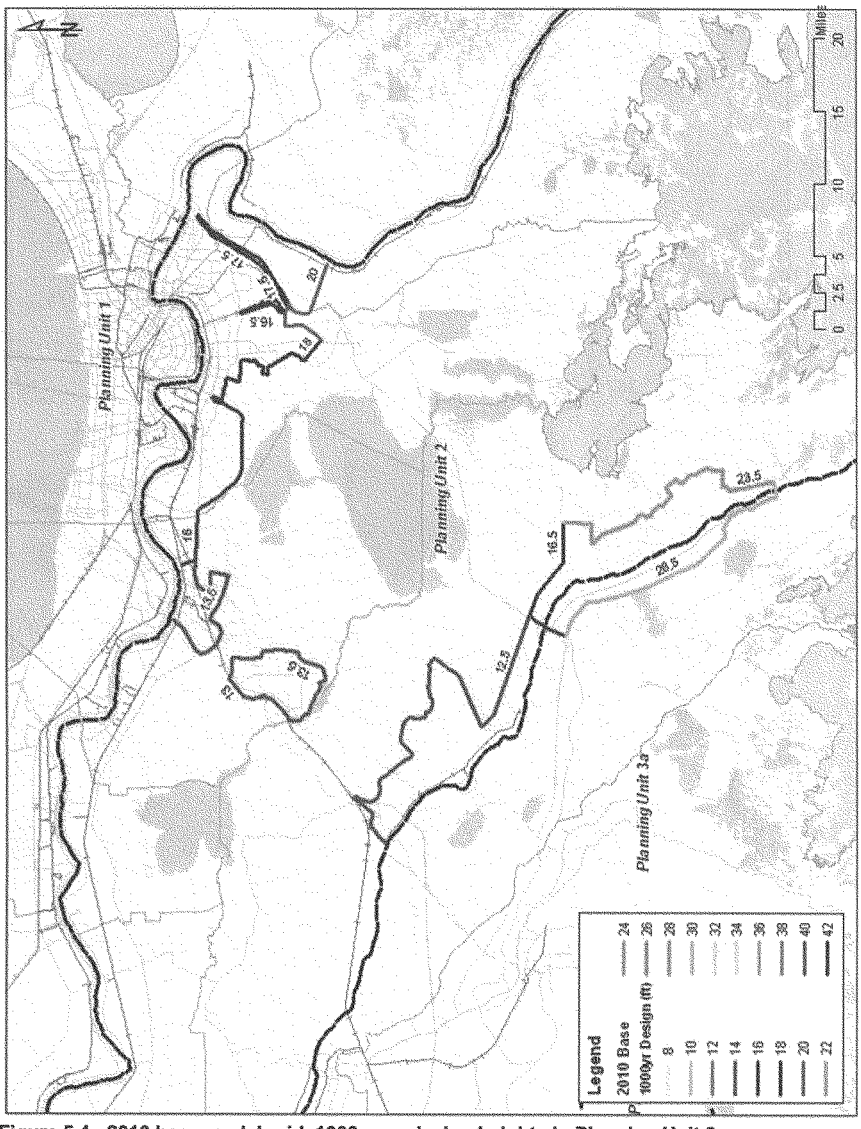

Figure 5.4 - 2010 base model grid, 1000-year design heights in Planning Unit 2

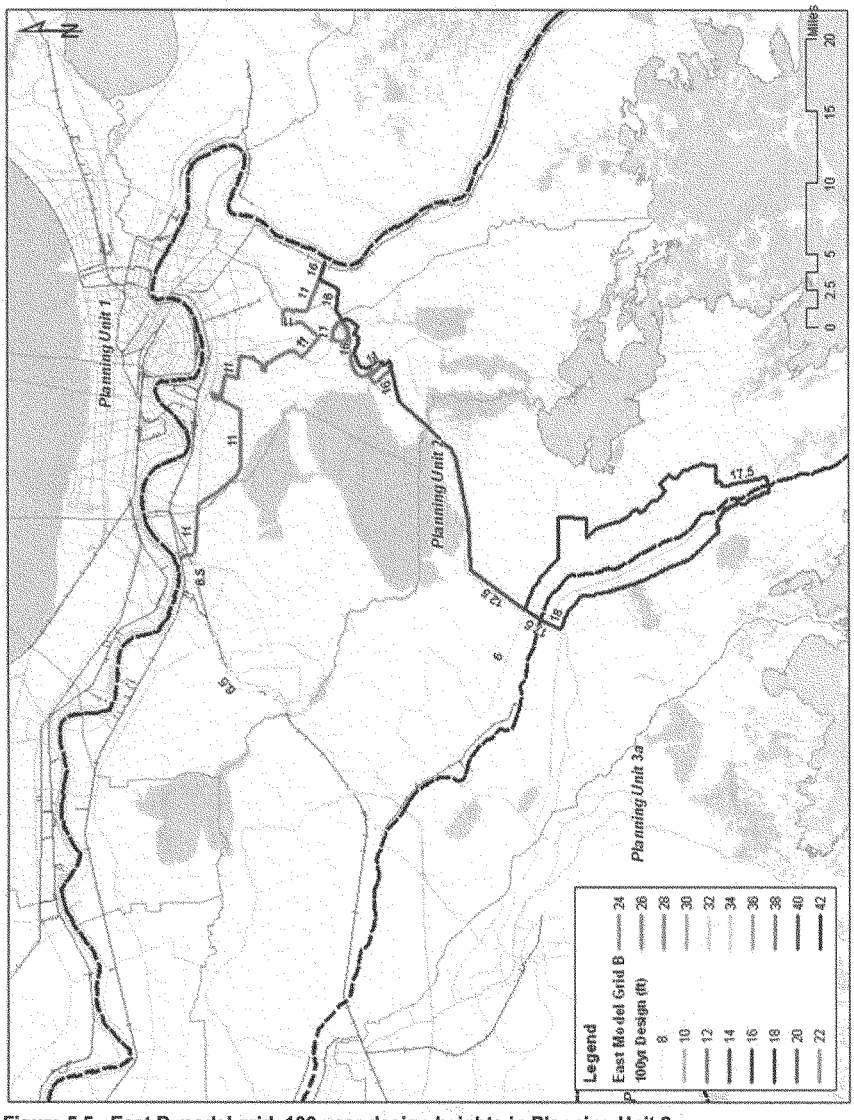

Figure 5.5 - East B model grid, 100-year design heights in Planning Unit 2

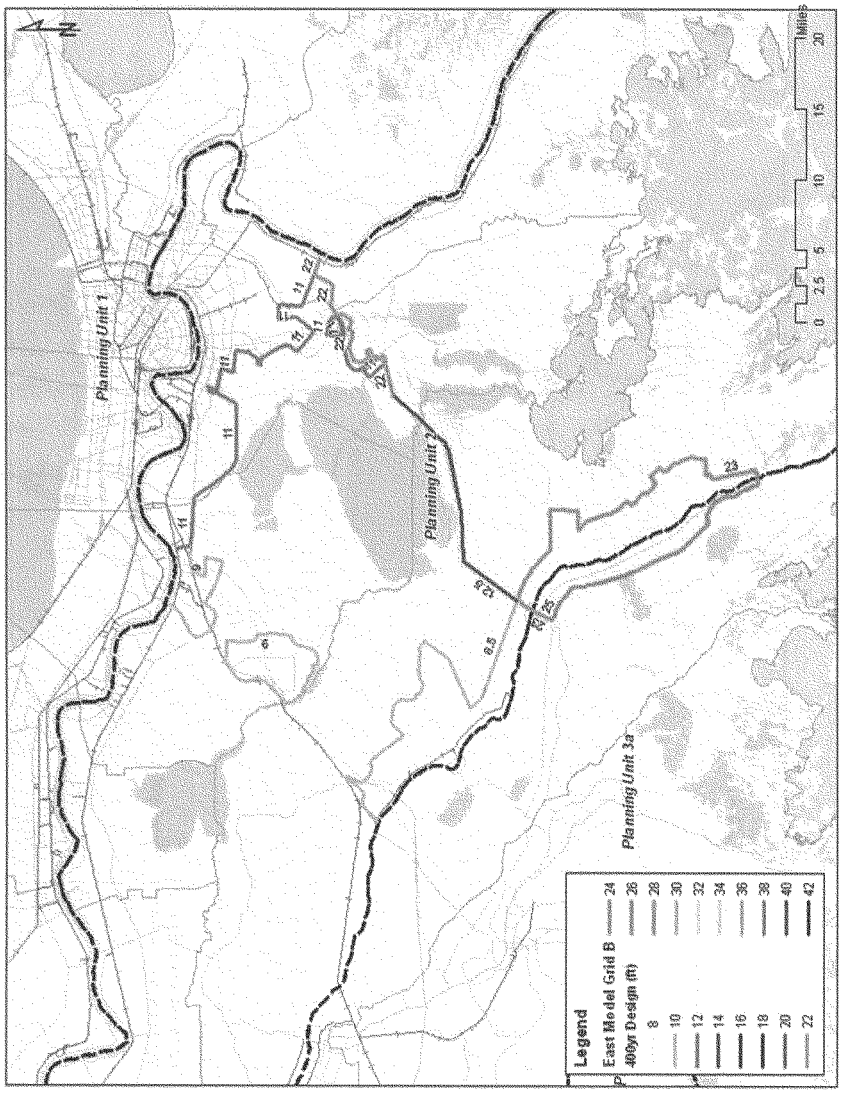

Figure 5.6 - East B model grid, 400-year design heights in Planning Unit 2

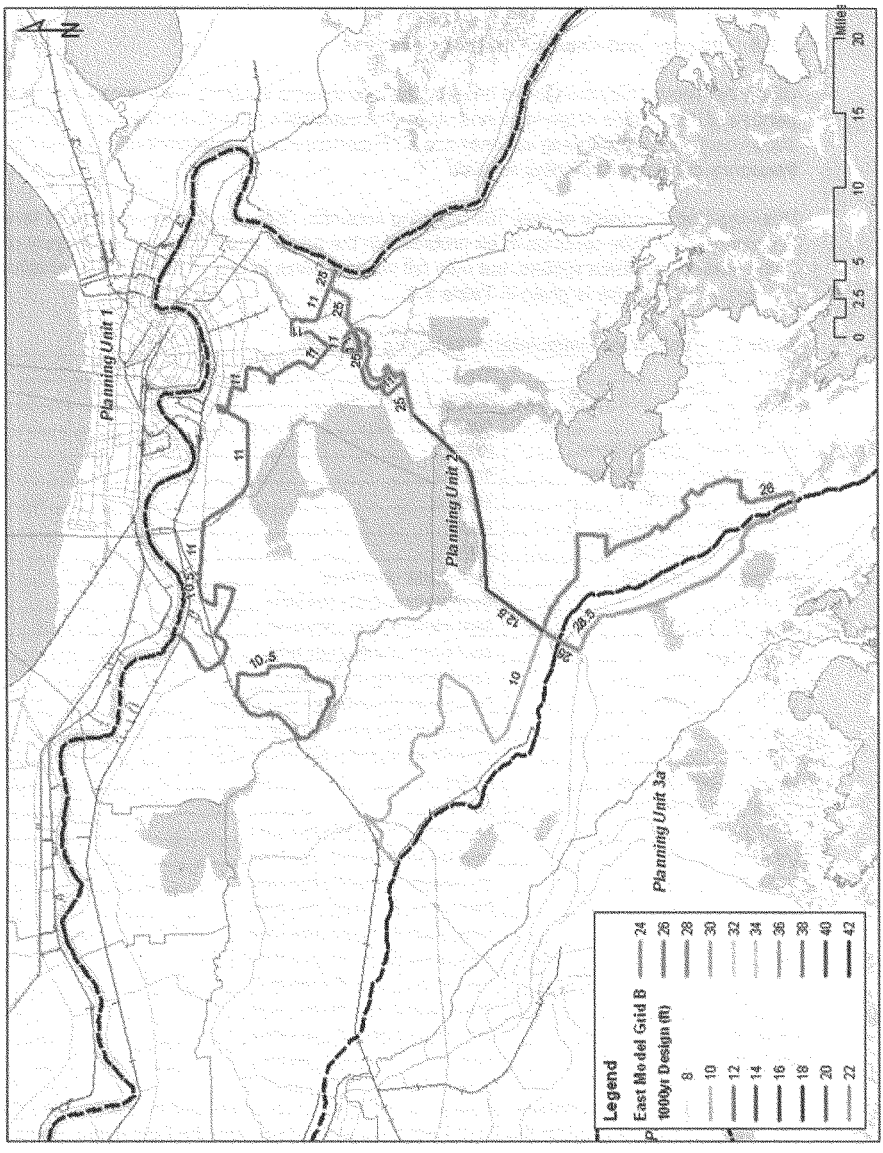

Figure 5.7 - East B model grid, 1000-year design heights in Planning Unit 2

5.3 Interior and exterior frequency curves

Given the above described levee heights, the overtopping volumes were computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). The overtopping volumes are an important input for determination of the interior stage frequency curves per planning subunit.

Planning Unit 2 consists of over 100 planning subunits. Of these, 9 are protected by levees and it is proposed that they continue to be protected in the future, and 21 fall in the category of currently being outside the levee system, but may fall within a levee in one of the future alternatives (semi-interior). A list of these is given in **Table 5.1**.

Table 5.1 - Interior and semi-interior planning subunits

Name	Type
Algiers	Interior
English Turn	Interior
West Jefferson - East of Harvey	Interior
West Jefferson - Harvey Estelle	Interior
West Jefferson - Ames	Interior
West Jefferson - Segnette	Interior
St Charles - Davis Pond	Interior
St Charles - Luling Boutte	Semi-interior - new levee
STCH_3c	Semi-interior (Luling) - new levee
STCH_5a	Semi-interior (Luling) - new levee
STCH_6a	Semi-interior (Luling) - new levee
St Charles - Sunset	Semi-interior - non federal levee
STCH_1c	Semi-interior (Sunset) - non federal levee
STCH_1d	Semi-interior (Sunset) - non federal levee
STCH_2c	Semi-interior (Sunset) - non federal levee
Larose to Golden Meadow	Interior
Plaquemines - Belle Chase	Interior
LAFO_2c	Semi-interior (Lockport) - new levee
LAFO_3b	Semi-interior (Lockport) - new levee
LAFO_3c	Semi-interior (Lockport) - new levee
LAFO_4c	Semi-interior (Lockport) - new levee
LAFO_5d	Semi-interior (Lockport) - new levee
LAFO_6b	Semi-interior (Lockport) - new levee
LAFO_6c	Semi-interior (Lockport) - new levee
LAFO_7e	Semi-interior (Lockport) - new levee
LAFO_8e	Semi-interior (Lockport) - new levee
LAFO_9a	Semi-interior (Lockport) - new levee

The stage storage relationships for these planning subunits are based on the results from two of the hydraulic modeling grids: the 2010 East base model (BS) and the East hydro model grid B (EB). Refer to Annex B for more details on individual sub-basins.

6 PLANNING UNIT 3A

6.1 Introduction

This chapter presents the results of the calculations of the levee heights and stage frequency curves for Planning Unit 3a. It addresses the development of the 100-year, 400-year, and 1000-year design heights in feet for the potential levee alignments within Planning Unit 3a for the 2007 base West, West A and the West B model grids.

In section 6.2 computation of the levee heights is described for the different alternatives in Planning Unit 3a in 2007.

As for the other planning units, the resulting heights are used for the purposes of costing design options and for establishing overtopping rates. The combination of the heights to given planning alternatives is coupled with the stage frequency analysis as is described in section 6.3.

Figure 6.1 shows the planning subunits in Planning Unit 3a.

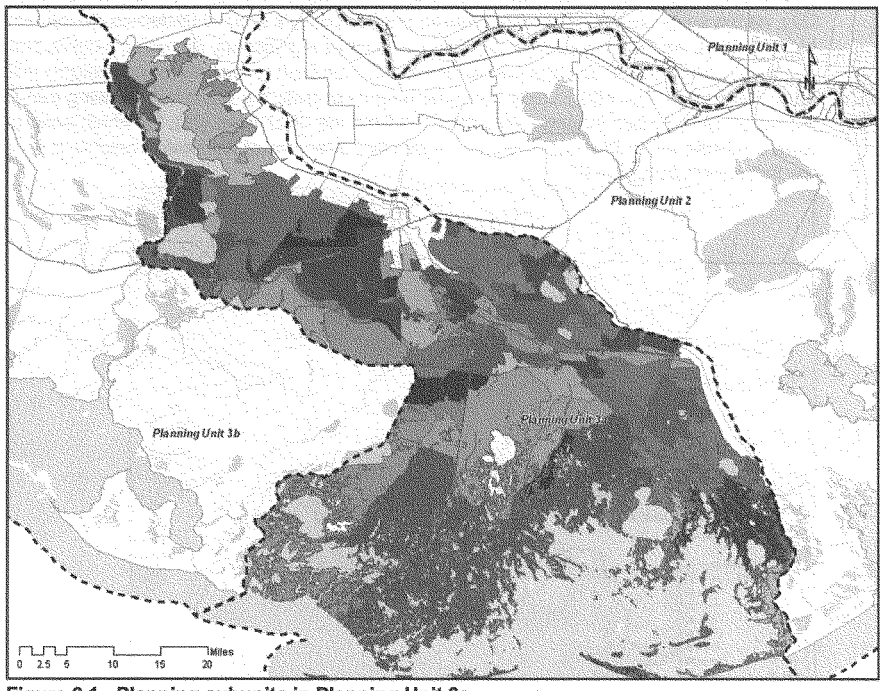

Figure 6.1 - Planning subunits in Planning Unit 3a

6.2 Levee height

6.2.1 General

The levee height design has been carried out using the methodologies described in Volume I of this appendix. Levee heights have been computed for the West A (WA) and the West B (WB) model grids, for three separate design standards (100-year, 400-year, and 1000-year) for all levees included in the alternatives. The design elevations are based on the wave heights and periods obtained from the no bed friction STWAVE models for the west.

Figures 6.2 through **6.9** show the levees designed for Planning Unit 3a using the WA and WB model grids for the 100-year, 400-year, and 1000-year levels of risk reduction. As the WB grid was only used only for the area of Morgan City, only the results for that area are shown with the WB grid.

The levee heights north of Morgan City run along the edge of the Atchafalaya Basin. This is an area where the effects of the extreme conditions are lesser than on the southern side of Morgan City, as the surge has to propogate through the narrowing's around Patterson (to the west of Morgan City). As the levels within the basin, and more significantly the wave heights, are not well modeled, particularly in the hydrodynamic models with high levees, the levee heights within the basin have been derived by using the levee heights outside the basin but reducing their height by the apparent reduction in surge level obtained from the 2007 base grid, for which base values were available within the model results. A detailed overview of the design heights is also given in the fact sheets in Annex B.

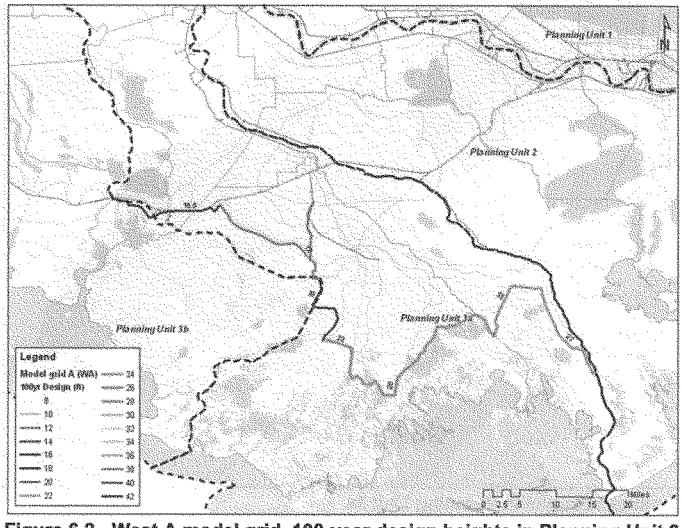

Figure 6.2 - West A model grid, 100-year design heights in Planning Unit 3a

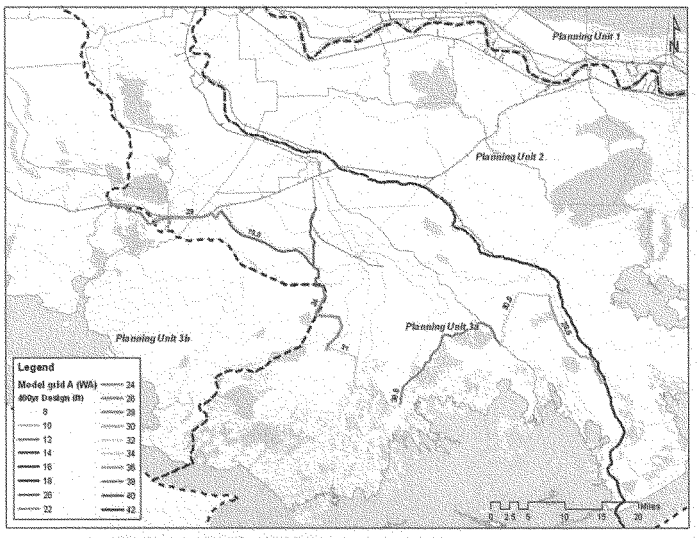

Figure 6.3 - West A model grid, 400-year design heights in Planning Unit 3a

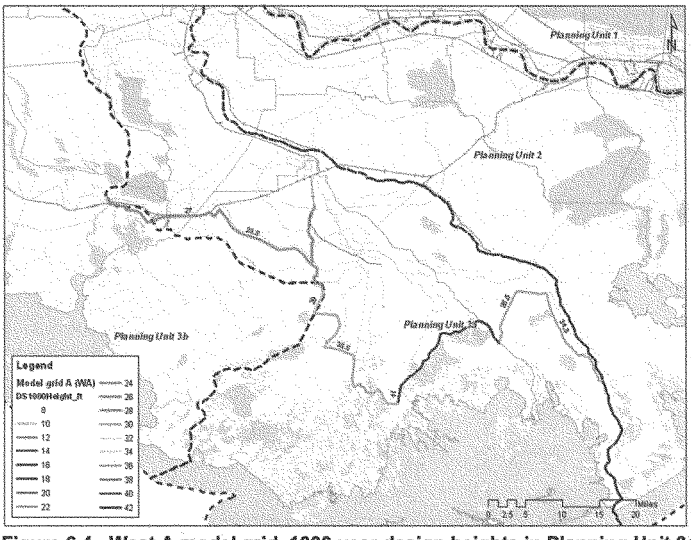

Figure 6.4 - West A model grid, 1000-year design heights in Planning Unit 3a

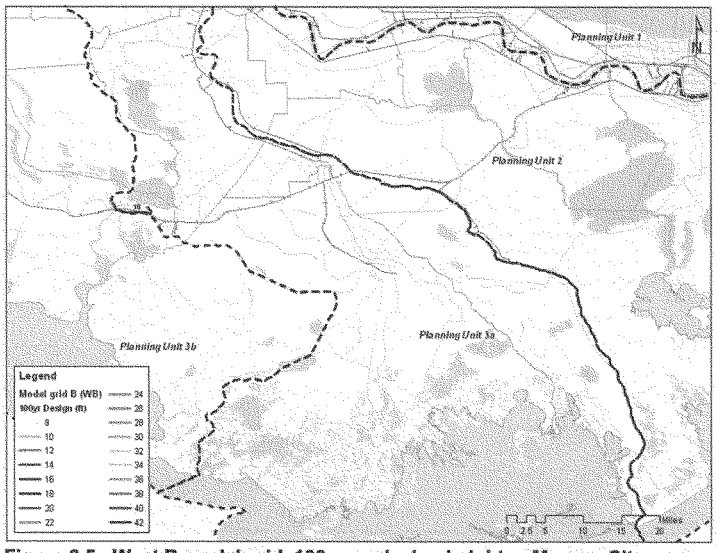

Figure 6.5 - West B model grid, 100-year design heights - Morgan City

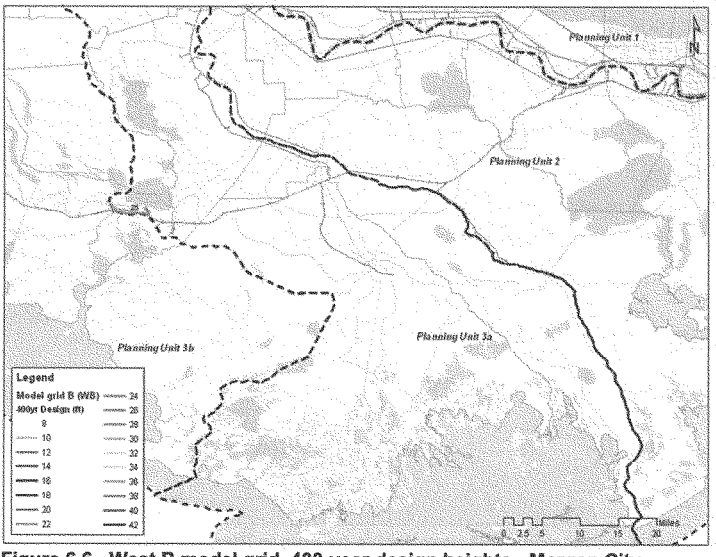

Figure 6.6 - West B model grid, 400-year design heights - Morgan City

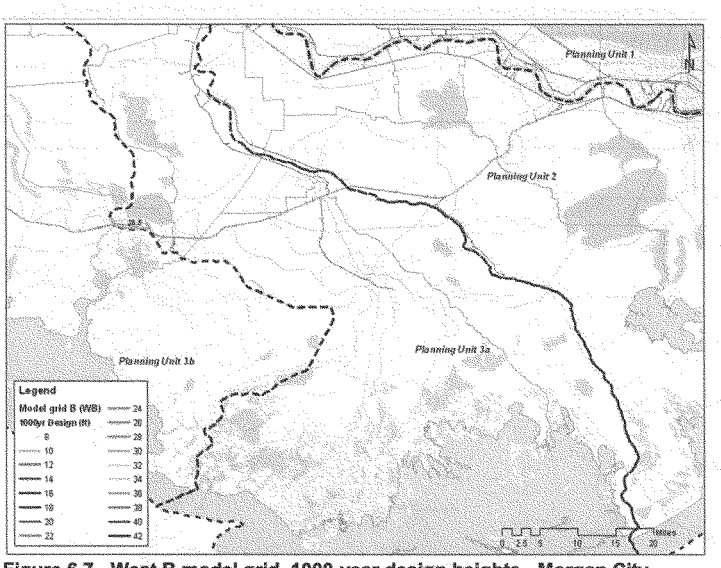

Figure 6.7 - West B model grid, 1000-year design heights - Morgan City

6.2.2 Ring levee alignment with secondary defense

This alternative in Planning Unit 3a protects the main Morganza area with a levee based on the 100-year design, while the higher grounds more inland of Morganza are protected by a secondary levee (only 400- and 1000-year design), thus providing it a 400- or 1000-year level of risk reduction. The 100-year design height of the levee at the perimeter of Morganza uses the same design heights derived from the WA grid for the 100-year design. The levee design height of the back levee is based on the interior stages in the Morganza area instead of the surge elevation of the 400- and 100-year conditions in front of this levee. The stage values were obtained by considering the overtopping of the outer levee line for the 400- and 1000-year events to provide a starting water level within the inner area.

Given the 100-year design height at the outer rim of Morganza in combination with a storm event with a return period of 400 years, an interior stage of 6.6 feet results for the Morganza polder (90% confidence level). To determine the wave height and period, first the return period of a 6.6 feet stage is derived for a point in the data set that is close to the levee alignment. The wave height and period have then been set, matching with the found figures. This results in a new set of input parameters based on which the levee height is calculated. For the 1000-year design height a similar procedure has been followed (see **Table 6.1**).

Table 6.1 - Input parameters used for design height of the Morganza back levee (stages taken from interior frequency curves)

Design standard	Stage (ft) (90%)	Significant wave height (ft)	Peak wave period (s)	Levee Height (ft)
400-year	6.6	2	4.51	9.5
1000-year	15	4.1	7.48	24

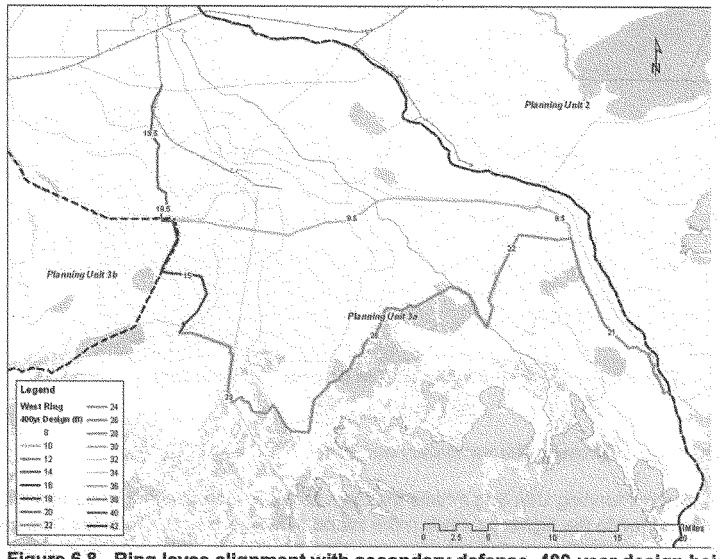

Figure 6.8 - Ring levee alignment with secondary defense, 400-year design heights in Planning Unit 3a

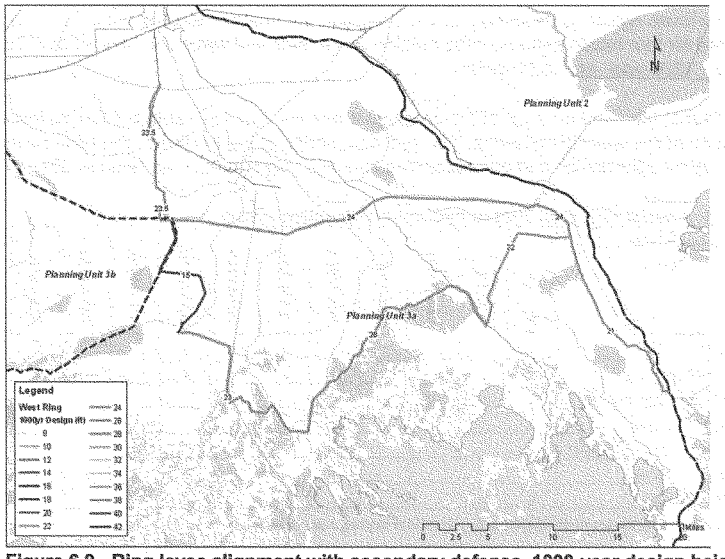

Figure 6.9 - Ring levee alignment with secondary defense, 1000-year design heights in Planning Unit 3a

6.3 Interior and exterior frequency curves

Over 200 planning subunits are defined in Planning Unit 3a. These fall into 2 groups, those which are always outside of the levees and those which fall inside the levees in one of the proposed alignments.

The planning subunits are grouped together to form areas considered for interior drainage. This grouping is based on the level of risk reduction being provided to the area by a particular levee alignment. Thus, the planning subunits are grouped into six internal drainage areas. These are listed in **Table 6.2** below.

Table 6.2 - Internal drainage areas

Name	Used in	Model
Morgan_City	Ring alternatives	WB
Morganza_with_ret_ring	Ring alternatives	WA
East_of_Morgan_City_ring	Morganza alternative	WA
Morganza_no_ret_ring	Morganza alternative	WA
Morganza_with_ret_ring_m_only	Ring Alternative (100-year levee)	WA
Morganza_back_levee	Ring Alternative (100-year levee)	Hand calculation

Given the levee height as described above in section 6.2, the overtopping volumes were calculated for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year).

The grids that are used for each of the drainage areas are listed in **Table 6.3** below, followed by a description of the specific variations. The hydraulic codes at the top line of the table relate to the various layout alternatives and corresponding sources of information; as listed in Chapter 3.

Table 6.3 - Interior drainage area to hydraulic alternative matrix

Interior Drainage Area	WT_ext	WA-ext	WA-0100a	WA-0400a	WA-1000a	WA-0100b	WA-0400b	WA-1000b	WA-0100c	WA-0400c	WA-1000c
Morgan_City	x					x	x	x			
Morganza_with_ret_ring	x					x	x	x			
East_of_Morgan_City_ring	x	x	x	x	x						
Morganza_no_ret_ring	x		x	x	x						
Morganza_with_ret_ring_m_only	x								x	x	x
Morganza_back_levee	x								x	x	x
Other External Areas	x	x									

Note - as there are no levees in the Base model, all interior drainage areas also have a WT-ext value.

Whereas in Planning Units 1 and 2 the interior drainage areas are separate (i.e. they don't overlap) in Planning Unit 3a the interior drainage areas may nest or intersect, depending on the respective planning alternatives.

Specific comments with respect to the internal drainage areas in Planning Unit 3a are given in **Table 6.4**. Additional area specific information is given in Annex B.

Table 6.4 - Specific planning unit comments

Polder Ring	Polder(s)	Comment
Morganza_with_ret_ring_m_only	Morganza	For the alternative "Ring levee alignment with secondary defense," stages in Morganza are only analyzed given the 100-year design and the 100-, 400-, 1000- and 2000-year return periods.
Morganza_back_levee	North_of_Houma Houma West_of_Houma	For the alternative "Ring levee alignment with secondary defense," stages behind the secondary defense are calculated based upon overtopping from the flooded polder Morganza. An exception is made for the 2000-year event where stages are equal to the base surge conditions.

6.4 Hydraulic results of planning alternatives

A number of planning alternatives have been considered. They cover a range of design standards (100-year, 400-year, and 1000-year) together with a range of variations in alignment. All of these alternatives are used for the determination of interior and exterior stage frequencies, thus providing results that are used as the input for the economic evaluation. For the West, all the alternatives and the baseline have been considered in more detail.

The key alternatives are as follows:

- Morganza plan
- Morganza/ring levee plan
- GIWW/Morganza/Ring levee plan

These three basic alternatives use levee height designs and interior drainage results from different model runs, and different internal drainage areas (because of the nesting/intersecting of drainage areas). **Table 6.5** shows the linkage between the six interior drainage areas and the planning alternatives. The codes are explained in Chapter 3.

Table 6.5 - Internal drainage area to alternative sets

		Planning Alternative Set			
		Baseline	**Morganza plan**	**Morganza/ring levee plan**	**GIWW/Morganza/Ring levee plan**
Internal Drainage Area	Morgan_City			WA-xxxxb	WA-xxxxb
	Morganza_with_ret_ring			WA-xxxxb	
	East_of_Morgan_City_ring		WA-xxxxa		
	Morganza_no_ret_ring		WA-xxxxa		
	Morganza_with_ret_ring_m_only				WA-0100c
	Morganza_back_levee				WA-xxxxc
	Other External Areas	WT-ext	WA-ext	WA-ext	WA-ext

The alignments and levee heights for each of the alternatives are shown on the summary maps in the Evaluation Results Appendix. These maps all go along with an accompanying table showing stage frequency comparisons (baseline to alternative) for a selection of planning subunits.

7 PLANNING UNIT 3B

7.1 Introduction

This chapter presents the details of the calculations of the levee heights and stage frequency curves for Planning Unit 3b.

Section 7.2 addresses the development of the 100-year, 400-year, and 1000-year design heights in feet for the potential levee alignments in Planning Unit 3b for the 2007 base West, West A, and the West B model grids.

The heights computed during the process are used for costing design options and for establishing overtopping rates. The combination of the heights to given planning alternatives is coupled with the stage frequency analysis as described in section 7.3. **Figure 7.1** shows the planning subunits in Planning Unit 3b.

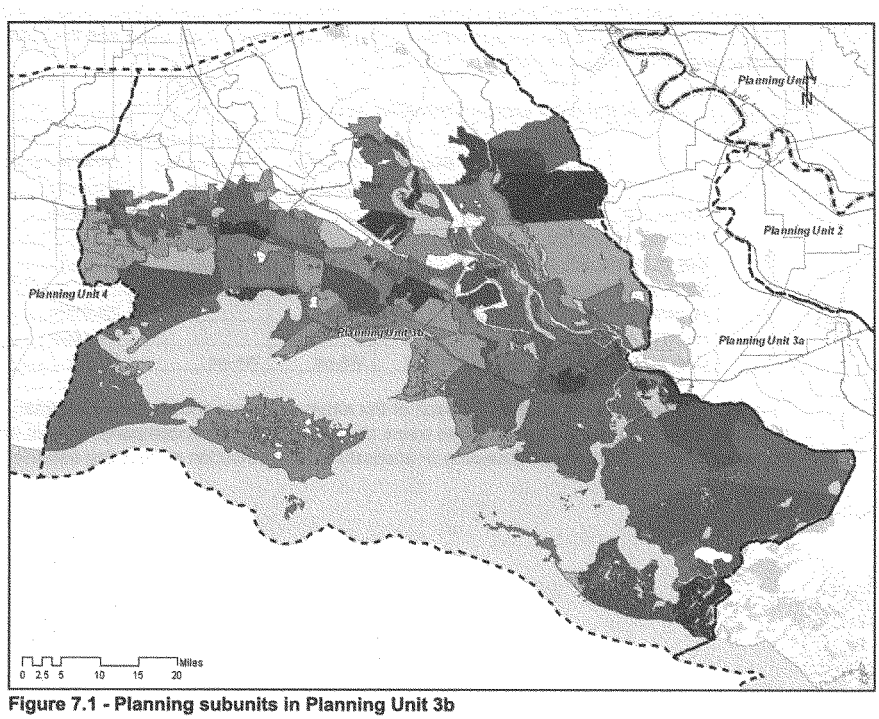

Figure 7.1 - Planning subunits in Planning Unit 3b

7.2 Levee height

The levee height design has been carried out using the methodologies described in Volume I of this report.

Levee heights have been computed using three models, for three separate design standards (100-year, 400-year, and 1000-year) for all levees included within the alternatives. The design elevations are based on the wave heights and periods obtained from the no bed friction STWAVE models for the west.

The 2007 base West (WT) model grid (containing no levees) has been used for designing a series of localized ring levees. The West A (WA) model grid has been used to design levee heights along the GIWW alignment, while the West B (WB) model grid has been used to design levee heights along a defense line between the GIWW and the higher ground, extending from Franklin to Abbeville.

The results of levee height designs from these three models for the 100-, 400-, and 1000-year levels of risk reduction are given in the figures below (**Figure 7.2 – 7.10**). A detailed overview of the design heights is given in the fact sheets in Annex B.

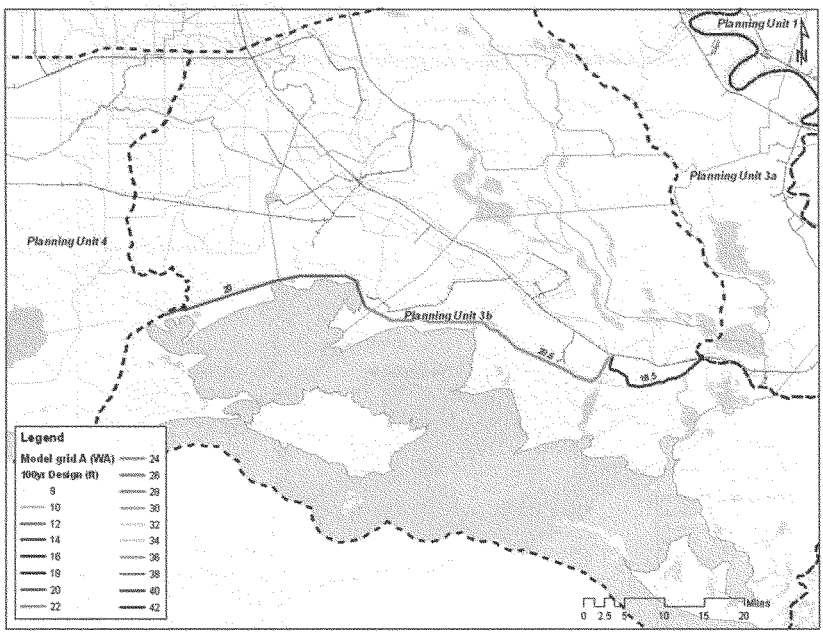

Figure 7.2 - West A model grid, 100-year design heights in Planning Unit 3b

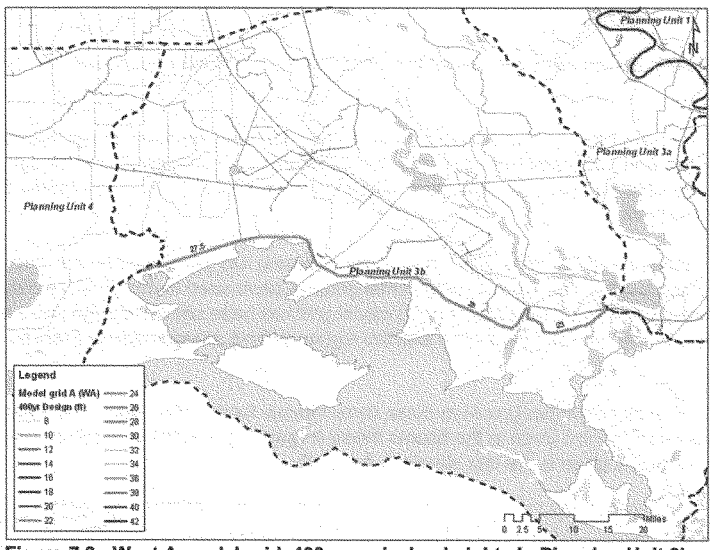

Figure 7.3 - West A model grid, 400-year design heights in Planning Unit 3b

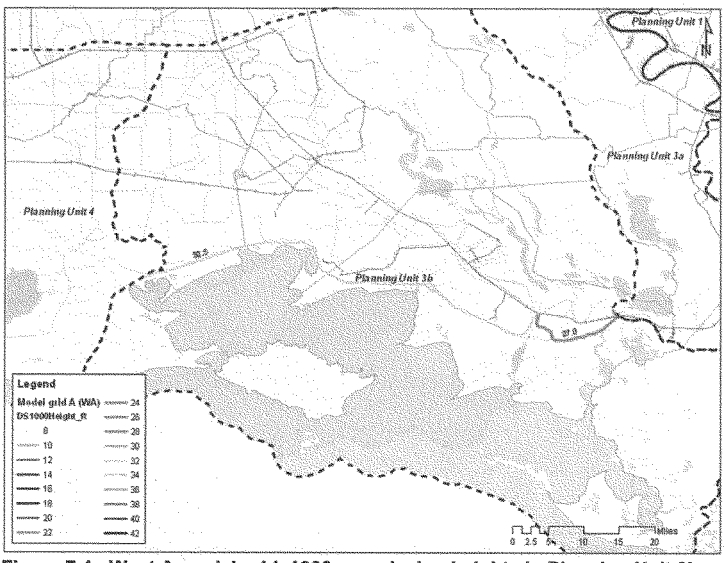

Figure 7.4 - West A model grid, 1000-year design heights in Planning Unit 3b

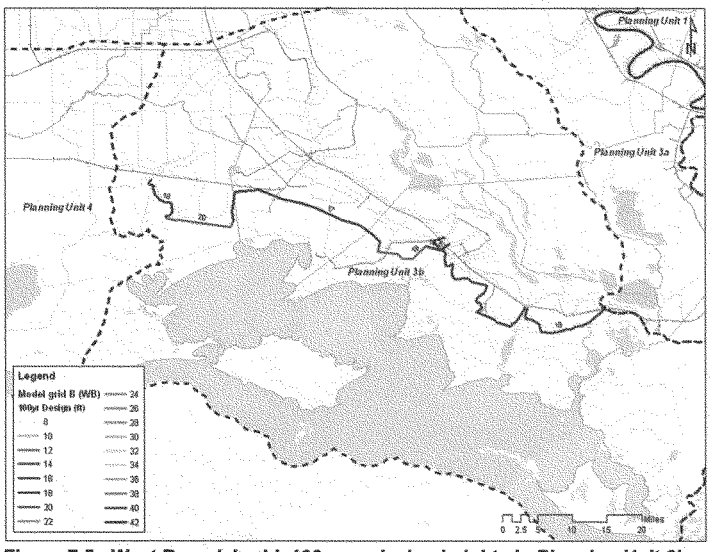

Figure 7.5 - West B model grid, 100-year design heights in Planning Unit 3b

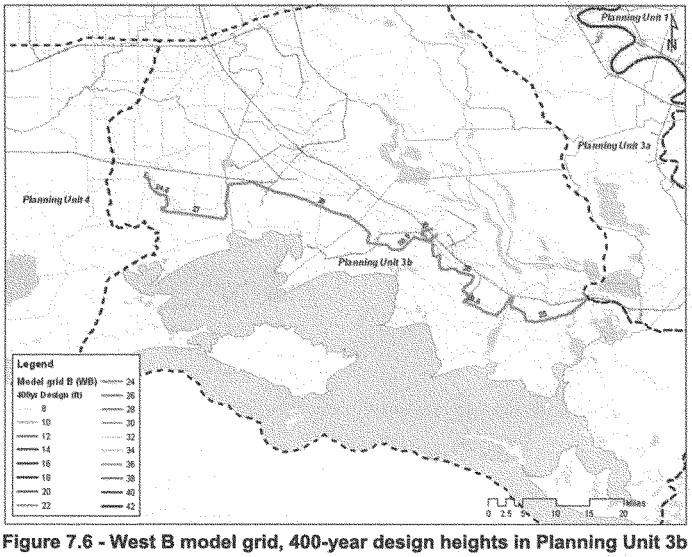

Figure 7.6 - West B model grid, 400-year design heights in Planning Unit 3b

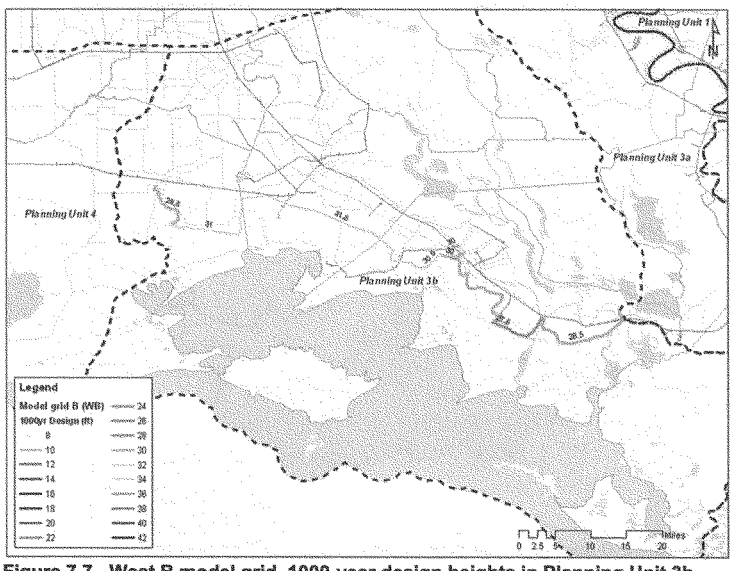

Figure 7.7 - West B model grid, 1000-year design heights in Planning Unit 3b

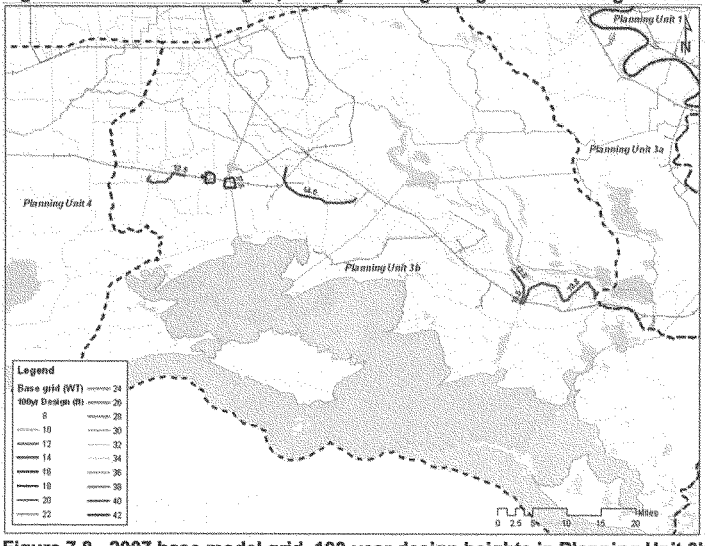

Figure 7.8 - 2007 base model grid, 100-year design heights in Planning Unit 3b

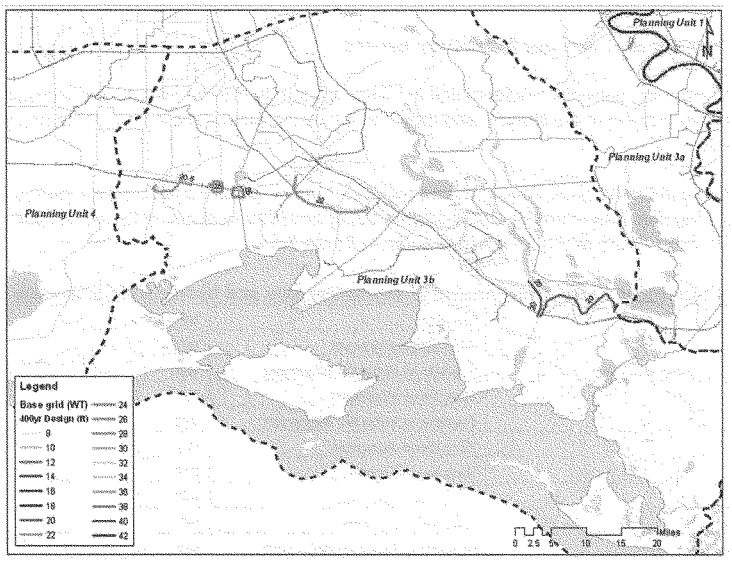

Figure 7.9 - 2007 base model grid, 400-year design heights in Planning Unit 3b

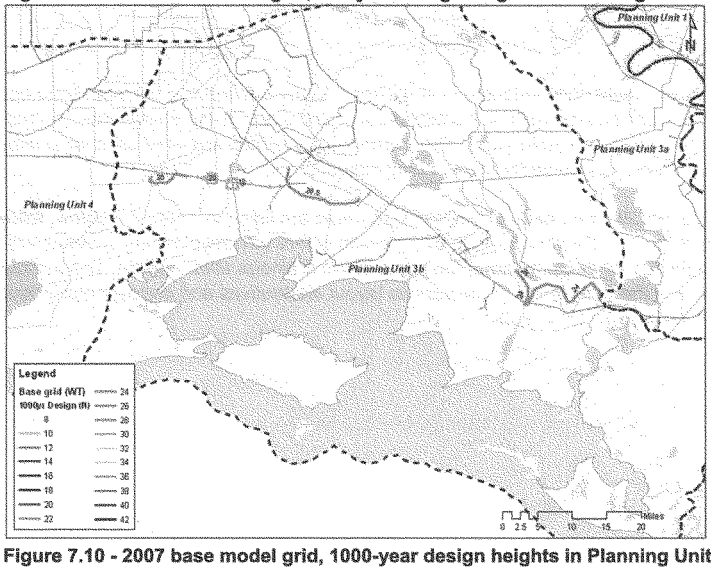

Figure 7.10 - 2007 base model grid, 1000-year design heights in Planning Unit 3b

7.3 Interior and exterior frequency curves

Over 200 planning subunits are identified in Planning Unit 3b. These fall into two categories: those being always outside of the levees, and those falling inside the levees in one of the proposed alignments.

The planning subunits have been grouped together to form areas considered for interior drainage. This grouping was based on the risk reduction being provided to the area by a particular levee alignment. A list of the drainage areas is given in **Table 7.1**.

Table 7.1 - Drainage areas - relationship to alternative sets and hydrodynamic models

Name	Used in	Model
Abbeville	Ring levee alignment	WT
Abbeville_to_Delcambre_ring	Franklin to Abbeville Alignment	WB
Baldwin	Ring levee alignment	WB
Charenton_ring	Franklin to Abbeville Alignment	WB
Delcambre	Ring levee alignment	WT
Erath	Ring levee alignment	WT
Franklin	Ring levee alignment	WB
New_Iberia	Ring levee alignment	WT
New_Iberia_ring	Franklin to Abbeville Alignment	WT
Patterson	GIWW alternative	WA
South_of_Franklin_ring	GIWW alternative	WA
GIWW_PU3b_ring	GIWW alternative	WA

Given the levee height described above in section 7.2, the overtopping volumes have been computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). The overtopping volumes are used as input to establish the interior stage frequency curve for the planning subunits.

The grids that are used for each of the drainage areas are listed in **Table 7.2** below, followed by a description of the specific variations. In those cases where several planning subunits are located within a single interior drainage area, the name of the drainage area is given. The hydraulic codes at the top line of the table relate to the various layout alternatives and corresponding sources of information; as listed in Chapter 3.

Table 7.2 - Interior drainage area to hydraulic alternative matrix

Interior Drainage Area	WT_ext	WA-ext	WT-0100	WT-0400	WT-01000	WB-0100	WB-0400	WB-1000	WA-0100	WA-0400	WA-1000
Abbeville	x		x	x	x						
Abbeville_to_Delcambre_ring	x					x	x	x			
Baldwin	x					x	x	x			
Charenton_ring	x					x	x	x			
Delcambre	x		x	x	x						
Erath	x		x	x	x						
Franklin	x					x	x	x			
New_Iberia	x		x	x	x						
New_Iberia_ring	x					x	x	x			
Patterson	x								X	x	x
South_of_Franklin_ring	x								X	x	x
GIWW_PU3b_ring	x								X	x	x
Other External Areas	x	x									

Note - as there are no levees in the Base model, all interior drainagae areas also have a WT-ext value.

Whereas in Planning Units 1 and 2 the interior drainage areas are separate (i.e. they don't overlap) in Planning Unit 3b the interior drainage areas may nest or intersect.

Specific comments with respect to the polders in Planning Unit 3b are given in **Table 7.3**. Additional area specific information is given in Annex B.

Table 7.3 - Specific planning unit comments

Polder Ring	Polder(s)	Comment
Patterson	Patterson	For the WA model the back levees at Patterson are 3 feet lower than the levee at the front/sea side. For the WB model the difference is slightly larger (4.5 to 5'), but the levels were adopted at the lower levels for consistency.The overtopping calculations are based upon the assumption that overtopping takes place only from the front/sea side.
GIWW alignment	GIWW_PU3b	The discharge of the river Vermillion is not included in the stored volume, pumps have been assumed to be installed to handle this additional volume.

7.4 Hydraulic results of planning alternatives

A number of planning alternatives have been considered for Planning Unit 3b. They consider a range of design standards (100-year, 400-year, and 1000-year) together with a range of variations in alignment and in the areas included within the protected areas. All of these alternatives are considered for interior stage frequency and results have been developed to be used as input for the economic evaluation. For the West, all the alternatives and the baseline have been considered in more detail.

The key alternatives are as follows:

- GIWW plan
- Franklin to Abbeville Alignment plan
- Ring levee plan

These alternatives use a different combination of levee height designs and interior drainage results. **Table 7.4** shows the linkage of the interior drainage areas to the planning alternatives, codes are explained in Chapter 3:

Table 7.4 - Internal drainage area to alternative sets

	Baseline	GIWW Plan	Franklin to Abbeville Alignment	Ring levee Plan
Abbeville				WT-xxxx
Abbeville_to_Delcambre_ring			WB-xxxx	
Baldwin				WT-xxxx
Charenton_ring			WB-xxxx	
Delcambre				WT-xxxx
Erath				WT-xxxx
Franklin				WT-xxxx
New_Iberia				WT-xxxx
New_Iberia_ring			WB-xxxx	
Patterson		WA-xxxx	WA-xxxx	WT-xxxx
South_of_Franklin_ring		WA-xxxx		
GIWW_PU3b_ring		WA-xxxx		
Other External Areas	WT-ext	WA-ext	WB-ext	WT-ext

The alignments and levee heights for each of the alternatives are shown on the summary maps in the Evaluation Results Appendix. These maps all go along with an accompanying table showing stage frequency comparisons (baseline to alternative) for a selection of planning subunits.

8 PLANNING UNIT 4

8.1 Introduction

This chapter presents the results of the calculations of the levee heights and stage frequency curves for Planning Unit 4.

Section 8.2 describes the development of the 100-year, 400-year, and 1000-year design heights in feet for the potential levee alignments in Planning Unit 4. These have been developed for the 2007 base West (WT), West A (WA) and the West B (WB) model grids.

The heights computed during the process are used as input for costing design options and for establishing overtopping rates. The combination of the heights to given planning alternatives is coupled with the stage frequency analysis as is described in section 8.3. **Figure 8.1** gives the planning subunits of Planning Unit 4. For evaluation purposes the planning subunits are grouped into drainage areas.

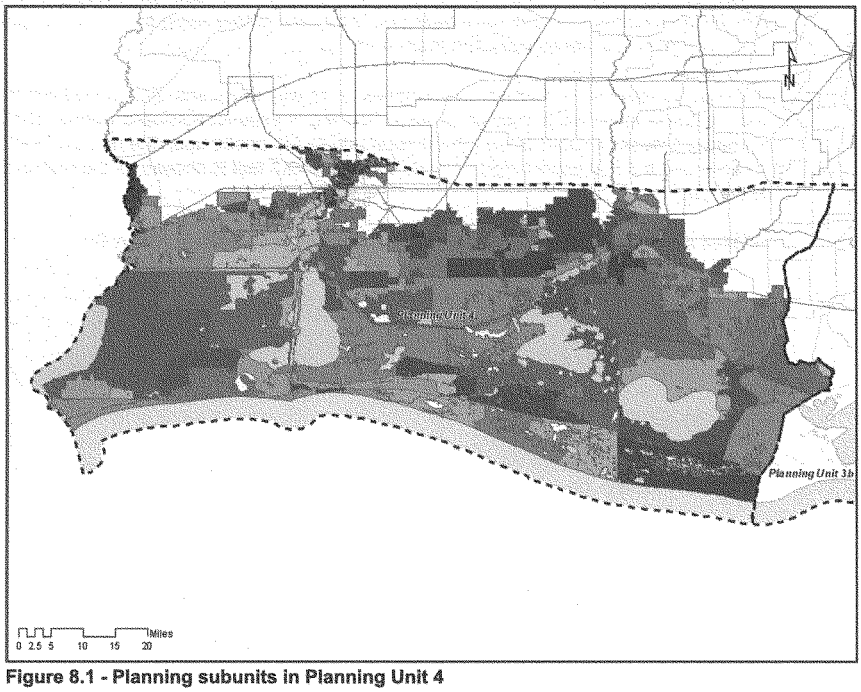

Figure 8.1 - Planning subunits in Planning Unit 4

8.2 Levee height

8.2.1 General

The levee height design has been carried out using the methodologies described in Volume I of this report. Levee heights were computed for the 2007 base West (WT) and West A (WA) model grids, for three separate design standards (100-year, 400-year, and 1000-year) for all levees included in the alternatives. The design elevations are based on the wave heights and periods obtained from the no bed friction STWAVE models for the west.

The 2007 base West (containing no levees) model grid has been used for designing a series of localized ring levees, while the West A model grid has been used to design levee heights along the GIWW alignment.

The results of levee height designs from these two models for the 100-, 400- and 1000-year levels of risk reduction are given in the following figures (**Figures 8.2 – 8.9**). In addition levees were designed considering a low (12-foot high) levee along the GIWW together with ring levees further inland. Heights for these levees were based on hand calculations using the interior stages based on overtopping of the levees using the West A model.

The Calcasieu River flows through Lake Charles on its way to the sea. Within the town there are a number of lower spots which may be at risk of flooding in extreme hurricane events. The heights of the proposed levees located along the Calcasieu River are based upon the modeled sea water elevations and a 3-foot wave height. A moderate wave of 3 feet is chosen as the levees do not face the sea.

A detailed overview of the design heights is given in the fact sheets in Annex B.

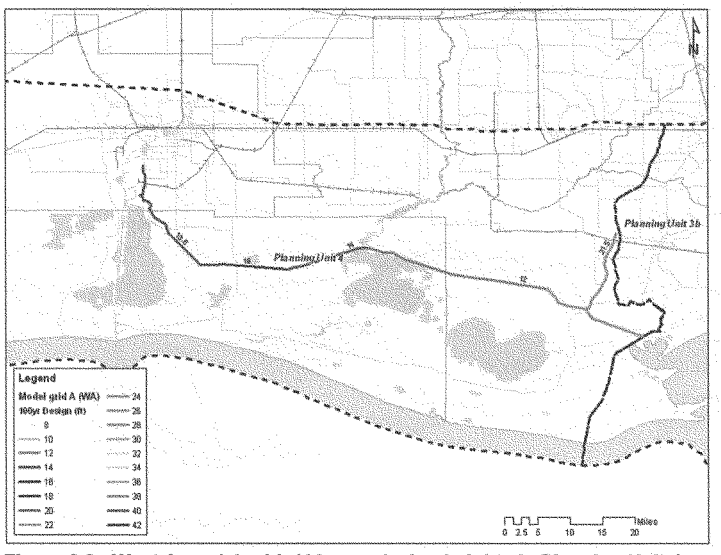

Figure 8.2 - West A model grid, 100-year design heights in Planning Unit 4

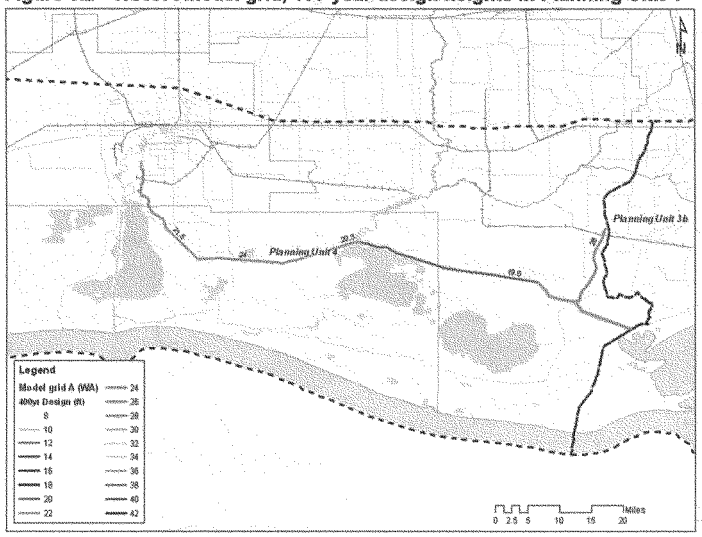

Figure 8.3 - West A model grid, 400-year design heights in Planning Unit 4

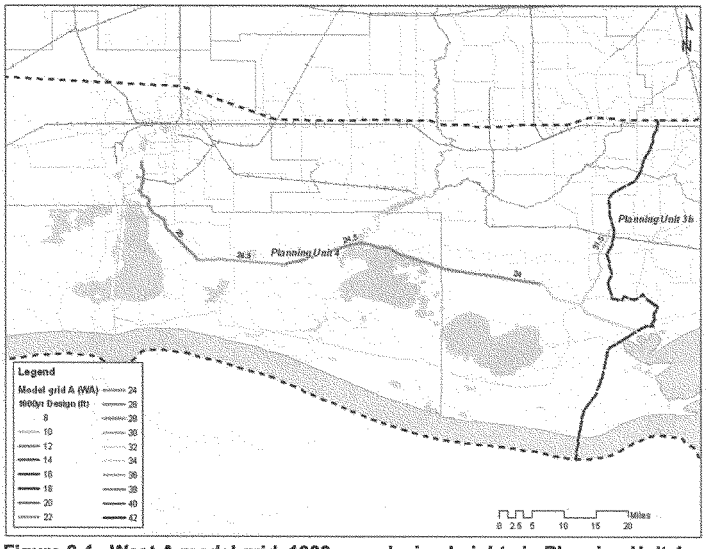

Figure 8.4 - West A model grid, 1000-year design heights in Planning Unit 4

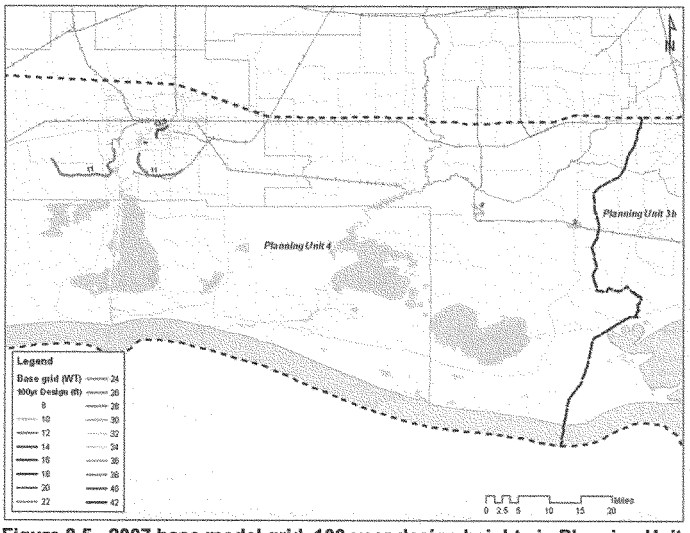

Figure 8.5 - 2007 base model grid, 100-year design heights in Planning Unit 4

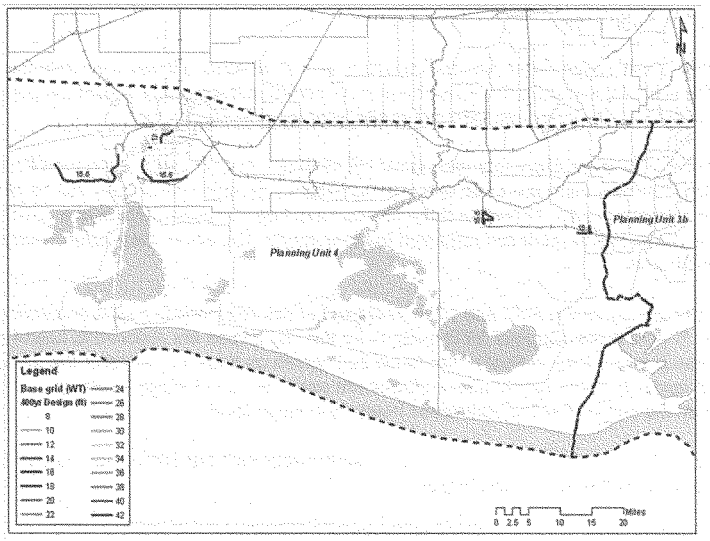

Figure 8.6 - 2007 base model grid, 400-year design heights in Planning Unit 4

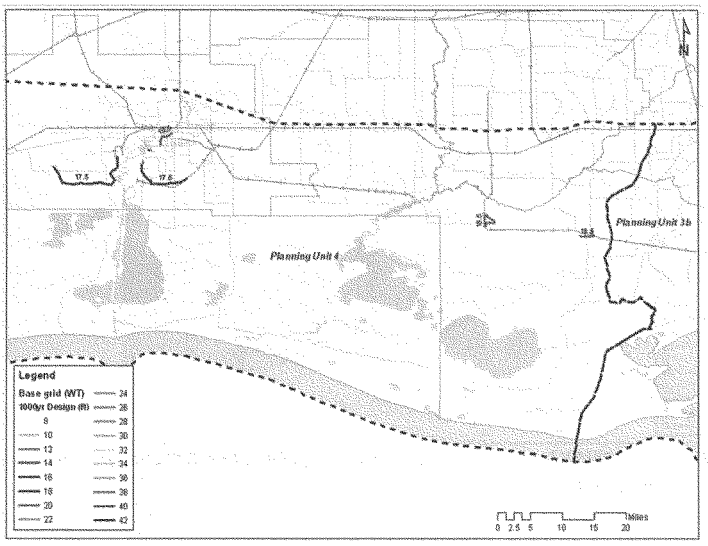

Figure 8.7- 2007 base model grid, 1000-year design heights in Planning Unit 4

8.2.2 12 feet GIWW levee alignment with return

The concept of the 12-foot levee alternative was to look at whether a lower level defense relatively close to the sea could improve the level of risk reduction for a large proportion of the population at risk without the need to introduce extreme levees near the populace.

The 12-foot GIWW levee alignment alternative consists of a 12-foot levee along the GIWW. Behind this alignment there are three areas of significant population which may still be at risk of flooding in extreme events. Therefore three levees have been proposed which can provide either a 400-year or 1000-year level of risk reduction to these areas. The areas are Kaplan, Gueydan and East Lake Charles.

Levee height design for the 400- and 1000-year level of risk reduction is based on interior stages behind the 12-foot levee. Original (WT) designs for the ring levees are reduced in height equal to the decrease in water level in front of the levee that occurs as a consequence of the presence of the 12-foot levee. In these cases interior stages are used instead of surge levels, except those cases where the surge level exceeds levee height and stage level.

For the levees bordering the Calcasieu River, a similar approach has been used. Levee design heights are given in the two figures below.

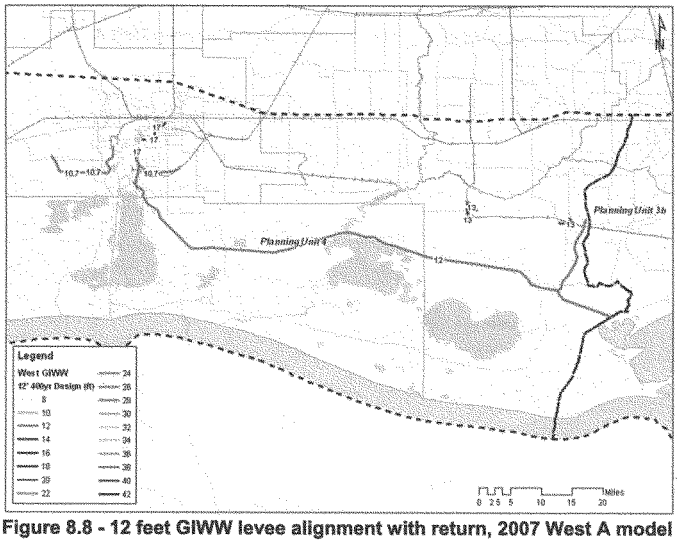

Figure 8.8 - 12 feet GIWW levee alignment with return, 2007 West A model grid, 400-year design heights in Planning Unit 4

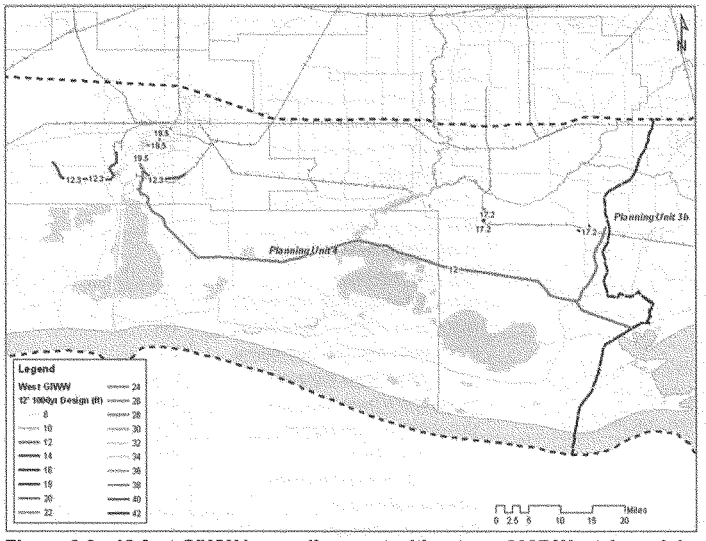

Figure 8.9 - 12 feet GIWW levee alignment with return, 2007 West A model grid, 1000-year design heights in Planning Unit 4

8.3 Interior and exterior frequency curves

There are over 200 planning subunits defined in Planning Unit 4. These fall into 2 groups, those being always outside of the levees and those falling inside the levees in one of the proposed alignments.

Similar to Planning Unit 3a and 3b, the planning subunits are grouped together to form areas considered for interior drainage. This grouping is based on the risk reduction being afforded to the area from a particular levee alignment.

The planning subunits are grouped into twelve internal drainage areas. These are listed in **Table 8.1** below, while, details of the areas are given in the Evaluation Results Appendix.

Table 8.1 - Drainage areas - relationship to alternative sets and hydrodynamic models

Name	Used in	Model
West_Lake_Charles	GIWW levee alignment 12 feet GIWW levee alignment Ring levee alternative	WT
East_Lake_Charles	12 feet GIWW levee alignment Ring levee alternative	WT
Gueydan	12 feet GIWW levee alignment Ring levee alternative	WT
Kaplan	12 feet GIWW levee alignment Ring levee alternative	WT
South_of_Lake_Charles_ring	GIWW levee alignment (with return)	WA
Central_PU4_ring	GIWW levee alignment (with return)	WA
GIWW_to_Veterans_ring	GIWW levee alignment (with return)	WA
South_of_Lake_Charles_ring_12	12 feet GIWW levee alignment	WA
Central_PU4_ring_large_12	12 feet GIWW levee alignment	WA
GIWW_to_Veterans_ring_large_12	12 feet GIWW levee alignment	WA
Prien	GIWW levee alignment 12 feet GIWW levee alignment Ring levee alternative	WT
Inner_Lake_Charles	GIWW levee alignment 12 feet GIWW levee alignment Ring levee alternative	WT

Given the levee height (see section 8.2) the overtopping volumes are computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year).

The grids that are used for each of the drainage areas are listed in **Table 8.2** below, followed by a description of the specific variations. In those cases where several planning subunits are located within a single drainage area, the name of the drainage area is given. The hydraulic codes at the top line of the table relate to the various layout alternatives and corresponding sources of information; as listed in Chapter 3.

Table 8.2 - Interior drainage area to hydraulic code matrix

Interior Drainage Area	WT_ext	WA-ext	WT-0100	WT-0400	WT-01000	WA-0100a	WA-0400a	WA-1000a	WA-0100c	WA-0400c	WA-1000c	WA-0100b	WA-0400b	WA-01000b
West_Lake_Charles	x		x	x	x									
East_Lake_Charles	x		x	x	x								x	x
Gueydan	x		x	x	x								x	x
Kaplan	x		x	x	x								x	x
South_of_Lake_Charles_ring	x	x				x	x	x						
Central_PU4_ring	x	x				x	x	x						
GIWW_to_Veterans_ring	x	x				x	x	x						
GIWW_to_Veterans_inc_ret_ring	x	x							x	x	x			
GIWW_PU3b_ring (PU4-ext)	x	x				x	x	x						
South_of_Lake_Charles_ring_12	x											x		
Central_PU4_ring_large_12	x											x		
GIWW_to_Veterans_ring_large_12	x											x		
Prien	x		x	x	x									
Inner_Lake_Charles	x		x	x	x									
Other External Areas	x	x												

Note - as there are no levees in the Base model, all interior drainage areas also have a WT-ext value.

Whereas in Planning Units 1 and 2 the interior drainage areas are separate (i.e. they don't overlap) in Planning Unit 3b the interior drainage areas may nest or intersect.

Because of the large areas of potential flooding and that these areas may be reaching the maximum potential length of coastline affected by a single hurricane approaching the GIWW levee, the area behind the GIWW levee has been split into three drainage areas even though they might act together. To evaluate the flooding in these areas they have been modelled as both independent, and as linked. When linked, the overtopping has been considered as coming from just one of the three sections (representing an extreme hurricane), but the volume has been distributed between all three areas.

For the GIWW as a primary defence, where it was required to ensure that the potential highest residual flood levels were recorded, the drainage areas are considered as acting independently, while when the levee was set at 12' and a more conservative approach could be considered, the areas were considered interlinked. These alternatives are described as the "large" internal drainage areas.

Specific comments with respect to the polders in Planning Unit 4 are given in **Table 8.3**. Additional area specific information is given in Annex B.

Table 8.3 - Specific planning unit comments

Polder Ring	Drainage area(s)	Comment

Polder Ring	Drainage area(s)	Comment
GIWW alignment	North of Veterans	The discharge of the river Mermentau is not included in the stored volume, pumps will handle this additional volume.
12 feet GIWW alignment	North of Veterans and Central PU4 ring	For the 12 feet alignment the stages within the polders are calculated based upon large Polder Rings.
12 feet GIWW alignment	South of Lake Charles	The stages for South of Lake Charles are based on overtopping of the levee protecting the Polder South of Lake Charles or based on flooding spreading form the Central PU4 polder. Due to higher grounds between Central PU4 and the South of Lake Charles polder (higher ridge at 8 feet) stages are set to a minimum of 8 feet when overtopping occurs from the South of Lake Charles side. Otherwise, stages within South of Lake Charles follow the stages caused by the flooding of Central PU4 if these stages exceed the threshold of 8 feet.

8.4 Hydraulic results of planning alternatives

A number of planning alternatives have been considered. They considered a range of design standards (100-year, 400-year, and 1000-year) together with a range of variations in alignment and in the areas included within the protected areas. All of these alternatives are considered for interior stage frequency and results developed for inclusion in the economic evaluation. For the West all the alternatives, plus the baseline were taken forward for more detailed consideration.

The key alternatives are as follows:

- GIWW plan;
- 12 feet GIWW plan;
- Ring levee plan.

These alternatives use different combinations of levee height designs and interior drainage results to produce sets of stage frequency results. **Table 8.4** shows the linkage of the interior drainage areas to the planning alternatives:

Table 8.4 - Internal drainage area to alternative sets matrix

	Alternative Sets			
Internal Drainage Area	**Baseline**	**GIWW Plan**	**12' GIWW Plan**	**Ring levee Plan**
West_Lake_Charles		WA-xxxxa		WT-xxxx
East_Lake_Charles			WA-xxxxb	WT-xxxx
Gueydan			WA-xxxxb	WT-xxxx
Kaplan			WA-xxxxb	WT-xxxx
South_of_Lake_Charles_ring		WA-xxxxa		
Central_PU4_ring		WA-xxxxa		
GIWW_to_Veterans_ring		WA-xxxxa		
GIWW_to_Veterans_inc_ret_ring				
South_of_Lake_Charles_ring_12			WA-0100b	
Central_PU4_ring_large_12			WA-0100b	
GIWW_to_Veterans_ring_large_12			WA-0100b	
Prien		WA-xxxxa		WT-xxxx
Inner_Lake_Charles		WA-xxxxa		WT-xxxx
Other External Areas	WT-ext			

The alignments and levee heights for each of the alternatives are shown on the summary maps in the Evaluation Results Appendix. These maps all go along with an accompanying table showing stage frequency comparisons (baseline to alternative) for a selection of planning subunits.

9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

The hydraulic analysis for each LACPR alternative consisted of the following consecutive steps:

1. numerical computations of surge levels and wave characteristics using ADCIRC, WAM, and STWAVE;
2. frequency analysis using the JPM-OS method and determination of exterior stage frequency;
3. determination of the levee heights and overtopping volumes;
4. determination of the interior stages including rainfall.

To provide a range of alternatives for evaluation and to enable the economic evaluation, each levee alignment alternative was evaluated for different risk reduction levels and event frequencies. A levee design has been made for three levels of risk reduction (100-year, 400-year, 1000-year). Given the level of risk reduction, the overtopping volumes have been computed for four return periods of the outside surge level and wave characteristics (100-year, 400-year, 1000-year, and 2000-year). For all alternatives, the 10-year rainfall was added to the overtopping volume to establish the interior stage frequency curve, while also pumping has been taken into account.

The ADCIRC model results and the "without friction" STWAVE results in combination with the JPM-OS method have been used for determination of levee design and overtopping quantities. This dataset has also been used for the calculation of interior and exterior stages. The method was adopted as being a robust approach for the purpose of comparison for the selection of the levee alignment plans presented in this report.

9.2 Discussion of assumptions and simplifications

From the onset of LACPR the scale of the work, both in terms of geographic area and the range of alternatives to be considered, has dictated the selection of methods and procedures used to determine results. The main assumptions and simplifications are described below.

Hydraulic Modeling and Frequency Analysis

- *Model alignments* - what has been modeled does not necessarily represent the final alternatives; this would have required an iterative process to review the results and then modify the models to optimize the levee layouts.
- *Range of storms* - for some models the suite of storms has been reduced so that the models could be completed within a practical time frame. The selection of the storms may have skewed the statistical results for the higher extremes.
- *Result presentation* - only for selected points results have been prepared for all models rather than for all model points, which would have enabled a more accurate view of the spatial variation in variables and the identification of problem areas within the model grids.
- *Wave computations with and without friction* - at present all calculations have been made using wave heights derived from a without-friction model. The with-friction models have been

developed, but the results have not been used in the analysis to date. Validation of the wave models is also required.

Levee Design

- *Use of a single point to define a long length of levee* - some of the variations in surge elevation and wave height have not been considered as the levee height design has been based on a single result point - in some cases extending some tens of miles.
- *Fixed design criteria* - the design process has only considered earthen embankments, and all levees have been designed to the same design criteria. No consideration has been made to other forms of construction, to the area at risk, or to the consequences of failure.

Stage Frequency Calculations

- *Simple stage storage approach* - no allowance has been made in the stage frequency analysis for the flow of water within areas and the time taken for areas to fill and empty.
- *Large storage areas* - the size of the storage areas has generally been large and the areas have been considered to act primarily in isolation. Only in critical areas interconnection has been considered and then only in a very simple "overflowing bucket" process.
- *No breaching of defenses* - the levees have been assumed to withstand all ranges of overtopping without breaching. This may potentially lead to higher flood depths inside a levee as the water can "pump up" due to the influences of waves.
- *Simplified pumping* - pumping has been included at the existing capacity where known. In other areas a simple relationship of 0.5 cfs/acre has been adopted without any consideration of any potential storage areas, the time for the water to reach the pumps or the distances over which pumping needed to take place. In some areas the area used for computing the pumping capacity has been fixed as there was no upper limit because of normal catchment drainage.
- *Pre-event storage*. No allowance has been made for water trapped inside areas once flood defenses have been shut prior to a hurricane.
- *Rainfall* - the rainfall rates have been taken as a synthetic distribution based on a fixed 10 year rainfall event. This has been used for all events, whereas in reality the rates of rainfall are likely to change for different return periods of event, and the distribution is unlikely to follow the uniform distribution assumed for this evaluation.
- *Joint probability of high river levels and high surges* - no consideration has been given to the potential for increased levels at the interface between tidal and fluvial flows. River flows and levels have been taken as nominal.

Many of the above issues can easily be dealt with for small areas or if sufficient time and resources are available. Within the constraints of LACPR, they have been adopted as a pragmatic approach.

However, as the range of alternatives decreases then some of these issues should be reviewed and changes to the methodology made. It is not easy to quantify the effect that addressing these issues will have on the final results, but as alternatives progress towards more detailed design then addressing the issues will improve confidence on the absolute values obtained.

In addition it will be difficult to improve all of the issues to the same degree for all of the planning units. As such, care should be taken in improving issues if the approach cannot be applied to all planning units as this may make comparing alternatives difficult, whereas at present the consistent approach adopted across all units makes alternatives comparable not only within a planning unit but also across planning units.

9.3 Recommendations for refinements to the hydraulic analysis

As the options presented in the LACPR technical report are implemented, there will be a number of opportunities to refine the work carried out to date. The types of refinements required will be defined by each specific project or study. In general, suggested key refinements of the hydraulic analysis used for future projects or studies are as follows:

- Model in ADCIRC and STWAVE the actual proposed levee alignments at their proposed heights and for the full suite of storms. Extract data for locations required for design and overtopping purposes for the proposed alignments. Ensure that a larger range of values are available for checking of the results.
- Validate the STWAVE results for the actual areas with a field measurement program of wave propagation over marshes so that the issue of no friction or with friction wave modeling can be resolved. It is without doubt that more insight and more predictive capabilities in this regard could save huge amounts of money considering the differences in design heights when waves with and without friction are applied.
- Undertake levee design specific to more localized areas and consider walls as well as embankments if necessary.
- Compute wave overtopping over shorter lengths to give variable inflow into flood areas.
- Use 2D modeling of flood flow to establish residual flood areas and depths and use the results of this modeling to site and design pump stations. 2D models should consider their interaction with adjacent areas so sequential flooding of drainage areas can be undertaken.
- In the 2D modeling consider varying rainfall and establishing relationships between rainfall and hurricane events so that the shape of the rainfall hydrograph more closely matches that which would be expected within an extreme event.
- Carry out further research to look at the parameters for defense breaching and determine if in extreme events levees should be assumed to breach. Investigate the impact of breaching on peak water levels within drainage areas.

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Annex A
Preliminary Hydrodynamic and Sensitivity Analysis Results

Summary

Hydrodynamic modeling information presented in Annex A includes storm surge results from ADCIRC runs along with STWAVE results. In order to obtain estimates for computing storm surge frequency, each node in the ADCIRC domain is monitored for its water level response as a storm progresses along it track during a simulation. The maximum surge and wave value obtained for each point for each storm simulation is used in the JPM-OS analysis to develop a stage frequency at that point. Stage frequency relationships were computed in this manner for all ADCIRC nodes in the LACPR planning units. Statistical analysis and surfaces are not presented in this annex. Statistical analysis and surfaces are discussed in Volume II of the H&H Appendix and surfaces are included in the Evaluation Results Appendix.

In general, in Annex A only the maximum of maximum (MOM) surge results and wave results are presented when making plan comparisons. The MOM plots were obtained from maximum ADCIRC surge responses at each ADCIRC node using the entire suite of storms. For a given plan, the maximum surge elevation at each point in the ADCIRC domain was compiled into a single surface to represent the highest stages and waves that can be produced in the ADCIRC domain for the entire suite of 152 storms. The MOM water level and wave plots for a plan are not indicative of a specific water surface or wave frequency but rather represent the maximum water level and wave height obtainable from a plan with the suites of storms used to generate the surge estimates.

Plan comparisons of MOM plots to the 2007 base MOM plot are presented in Annex A. The reader needs to be aware that for most of the plans analyzed in the LACPR modeling effort, levee heights in the model were set to not overtop for even the highest storm surge generated by the 152 storm suite. This was done so that water levels on the flood side of the levee system would not be moderated by overtopping and therefore levee design heights for a specified frequency could be obtained from the surge values generated at the face of the various levee reaches in a proposed plan. Levee design elevations in the LACPR analysis were developed for the 100-, 400- and 1000-year frequency water levels. This non-overtopping levee scenario is true for all plans unless otherwise noted. The one notable exception to the non-overtopping levees is the 2007 case where levee elevations in the model were set to the existing levee elevation at the start of the June 2007 hurricane season. Therefore, when comparing the 2010 base MOM condition to the 2007 MOM case, for the 2010 MOM one sees very large increases in water levels on the outside of levee systems and very large decreases in water levels on the protected side of the levee.

Future conditions for a projected 50-year future coast landscape are also discussed in Annex A. Landscape projections were obtained from the CLEAR modeling group who modified the ADCIRC grid to reflect the projected coastal change in near shore bathymetry. MOM plots of the future surge and wave heights are compared to the 2007 base MOM plots. Statistical analysis of future degraded no-action water surfaces are not presented in Annex A. Statistical surfaces are discussed in Volume II of the H&H Appendix and Maps are shown in the Evaluation Results Appendix.

Annex A also presents a sensitivity analysis for sea level rise in which water levels in the model were increased by 1, 2 and 3 feet above the base water level. The analysis was done to support ongoing restoration work for Corps offices involved in restoring levees damaged by Hurricane Katrina in response to the Congressional Directive to rebuild the damaged levee systems to their design grades or to the 100-year design level, whichever of the two levels is the higher. Using the 1, 2 and 3 foot increases in base water levels, the inundated marsh areas covered by these increased water levels were converted to water bottoms and the appropriate friction values were applied to simulate change in resistance to flow. Twenty-seven storms from the 152 storm suite were chosen for the sensitivity analysis. These storms were selected since their surge generating characteristics closely approximated the 100-year surge value in the area of on-going restoration work. The results from the sensitivity analysis were used to estimate future conditions for the restoration work in the New Orleans area but were not used in LACPR.

ANNEX A. PRELIMINARY HYDRODYNAMIC AND SENSITIVITY ANALYSIS RESULTS

A.1 Introduction

The purpose of the hydrodynamic modeling was to estimate the surge and wave conditions for the base conditions and various alternatives for storms in the JPM-OS suite for the calculation of stage-frequency curves. This involved an examination of the entire spatial domain every time step (1 sec) to determine if water levels exceeded the previous time steps maximum water level at any point in the domain. The result of this analysis is a maximum envelope of water level for a given simulation. From the ensemble of results, the expected return periods for those surge and wave conditions is calculated to assist in the quantification of risk. Example output generated from the ADCIRC model results are provided in the figures and discussion below. The results provided in this section are maximum surge elevations for each alternative or base condition for all storms simulated. The results are for illustrative purposes and as presented here do not provide information on levels of protection, which is discussed in subsequent chapters of this report. As discussed in Volume I of this report, five STWAVE grids were utilized. Results from only the southeast grid are presented here as an example of those results.

For each alternative presented herein, results for the suite of storms simulated for each alternative were scanned for peak elevations at each node, from which a "peak of peaks" data file and plot were produced (hereafter referred to as "peak" or "maximum" values/plots). The peak values for the same storm suite run on the 2007 base grid were then subtracted from the alternative maximum surge values to produce difference files and plots. Each figure title indicates the alternatives being compared to the 2007 base. Because the 2007 peak values are always subtracted from the alternative, a positive number indicates that peak surge values for the alternative are higher than the 2007 peaks, while a negative number indicates that peak surge values for the alternative are lower than the 2007 peaks. For surge results, the upper range of the legend ends at +12 feet (except for the Barrier Island cases), the maximum difference observed in any of the comparisons. Areas colored mauve indicate regions that were wet for at least one storm in the alternative simulations but were dry during all storms in the 2007 simulations. The lower range of the legend ends at -12 feet for the Marsh, and Plaquemines alternatives, the lowest observed difference for those scenarios. For the EA-ED levee system options, as well as the 2010 configuration, the lower range of the legend ends at -22 feet, the maximum observed difference for those scenarios. Areas colored dark green indicate regions that were dry during the entire storm suite in the alternative simulations but were wet for at least one storm in the 2007 base case. For these graphs, the range -12 to -22 feet was colored pale green; this color indicates regions that have a small depth of water present for at least one storm for that alternative (e.g., a marsh-like area), and they had a large peak surge value for at least one storm in the 2007 base case. The Barrier Island surge results legend had an upper and lower end of +/- 6 feet, respectively. For the STWAVE results, a legend with an upper range of +6 feet and a lower range of -6 feet was used for all alternatives, except the Barrier Islands, which used +8 feet at the upper end and -8 feet at the lower.

A.2 2007 Base Condition - East

The 2007 Base condition was created to represent South Louisiana as it was projected to exist at the start of the 2007 hurricane season. Post Hurricane Katrina and Rita topographic and bathymetric conditions were combined with levee definitions to reflect the system repairs and upgrades that were implemented as part of the USACE Task Force Guardian and by the USACE HPO and MVN.

Simulations were completed for the 152 storms for eastern Louisiana in order to define water levels and corresponding wave conditions. This information serves as a base condition to which alternative levee systems, barrier island variations, marsh improvements and/or degradation, and sea level rise can be compared. Section 2.4 of Volume 1 of this report (and section A.4 of this Annex) describes the motivation for study and changes to the physical system for each alternative inspected.

Figure A.2-1 represents the maximum surge level recorded for the East 152 storms simulated for Southeastern Louisiana. It is important to note that the displayed water levels are not stochastic representations of the 100-year or other return period water elevations, but rather are the maximum surge levels for all 152 specific storms simulated for the JPM-OS method. The highest surge levels occurred in three locations: along the east side of the Plaquemines Parish levee system from Belle Chasse to Port Sulphur; along the Mississippi coast near Biloxi and Gulfport; and northwest of Terrebonne Bay near Houma.

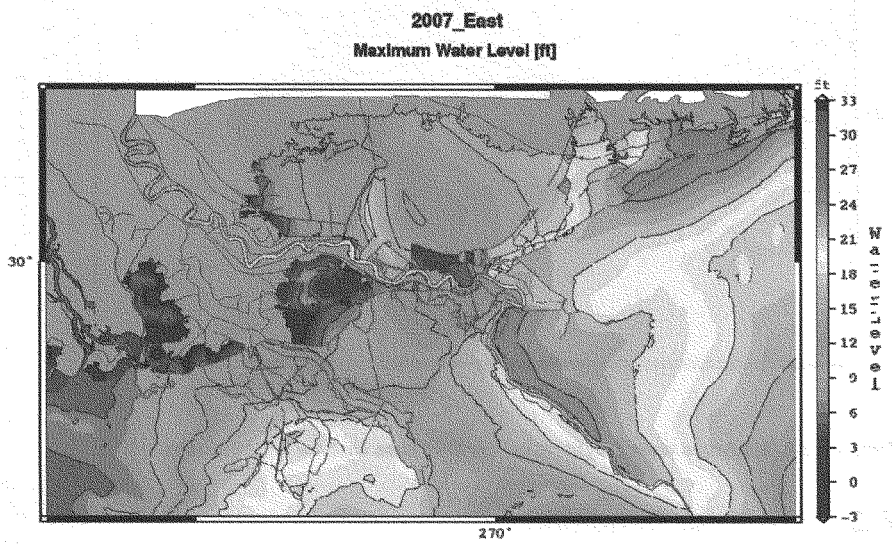

Figure A.2-1. Maximum surge level (ft) for the 2007 base case for all East 152 storms.

East of the Mississippi River, peak surge levels along the Plaquemines Parish levees all the way to the Mississippi Coast are on the order of 25 ft. These high surges develop as water is blown by easterly and then southerly winds onto the shallow Mississippi-Alabama shelf and is then stopped by the Mississippi River delta and levees and the coast of the state of Mississippi. The New Orleans metropolitan region is significantly flooded as the levees of St. Bernard Parish and New Orleans East are overtopped. Mean water levels also rise in Lake Pontchartrain as water flows via the Rigolets, Chef Menteur Pass and over the Pontchartrain land bridge, especially in easterly winds. In addition, strong localized set up occurs in this large shallow lake. Maximum surge elevations on the north and south shores of Lake Pontchartrain are approximately 15 to 18 ft. A substantial populace is located in areas directly affected by the water levels in the lake. For this reason, primary alternative levee systems inspected are various closure options for Lake Pontchartrain. Maximum surge values northwest of Terrebonne Bay in the Houma area are over 20 ft due in large part to surge being locally trapped both as it propagates north but also east and west by local levees and

roads. Maximum surge elevations along the West Bank levees are more modest reaching about 12 ft. Much like the regions adjacent to Lake Pontchartrain, a high populace resides in this portion of the state. Maximum 2007 wave heights outside of the east New Orleans levee system, as predicted by STWAVE simulations (Figure A.2-2), are on the order of 6 to 8 feet near Caernarvon, on the order of 6 to 8 feet near the MRGO/GIWW, and on the order of 8 to 10 ft near the Pontchartrain land bridge.

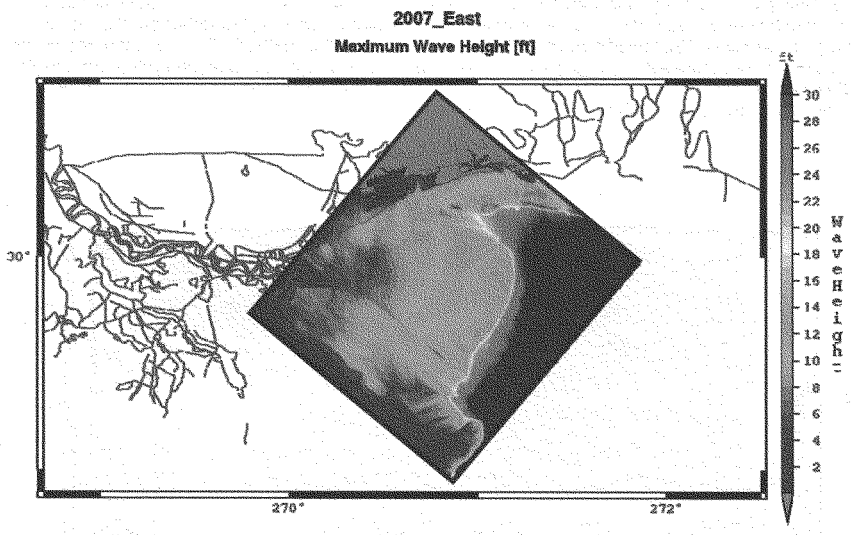

Figure A.2-2. Maximum wave height (ft) for the 2007 base case for all 152 storms.

A.3 2010 Base Condition

The 2010 condition represents the levee configuration that would exist if the proposed hurricane protection system was built to currently-authorized levels and also includes a levee that runs along the proposed Morganza to the Gulf alignment raised so that it does not overtop. The 2010 system also raises levee heights around the existing system in and around metropolitan New Orleans on both the east and west banks (with the exception of the Belle Chase) to approximate 100 year levels. In addition, the system includes a levee to close the MRGO/GIWW east of Paris Road to stop the propagation of surge into the heart of New Orleans. Figure A.3-1 shows the difference in the envelope of maximum water level between the 2010 and 2007 configurations. Initially, also included in the 2010 base run was the proposed levee for the Morganza to the Gulf project. Since the 2007 ADCIRC model runs had already addressed the base no levee condition for the Morganza area it was decided to include this proposed levee system to maximize design output from the ADCIRC runs. The Morganza levee is not expected to be completed by the 2010 date but was included in the ADCIRC runs to expedite reanalysis of the Morganza project which at the time was awaiting authorization for construction by the Congress. Around the Morganza area (to the west of Bayou Lafourche) the model included a non overtopping levee to represent the proposed new levee

around the Morganza area. The Morganza project was considered to be sufficiently removed from the PU-1 and PU-2 basins so as to not influence surge responses in those basins. Figure 2.5 below in fact shows that the proximity of the proposed Morganza levee has no influence on water levels in PU-1 and PU-2. For this conclusion, the reader should note the large area of no change separating the two response surfaces shown in Figure 2.5. To avoid confusion when looking at the 2010 base In the final Analysis for which results are presented in the Evaluation Results Appendix the 2010 Base condition was reanalyzed without the proposed Morganza Levee and all 152 storms in the east were run in ADCIRC. The 304 JPM-OS code was used to compute the frequency analysis for the 2010 Base condition.

The inclusion of a non-overtopping barrier around Golden Meadows and from Morganza to the Gulf blocks the incoming surge, raising the peak water levels south of the levees by up to 12 feet near Cut Off and up to 3 feet near Morgan City. Behind the barrier, the peak water levels in most areas are either reduced by 10 to 22 feet (dark blue and pale green areas), or they remain completely dry for all simulated storms, whereas in the 2007 configuration, the area had peaks up to 22 feet. Similar effects are seen in the New Orleans area, i.e., the fortified 2010 system either reduces the peaks by 10 to 22 feet (dark blue and pale green areas), or eliminates the surge entirely (dark green areas). Outside of the fortified New Orleans system, the peak surge increases in most areas about 1 to 3 feet. However, near the MRGO/GIWW closure, the peak surge increases 4 to 5 feet, while the peak surge near the English turn in the Mississippi increases by about 7 to 8 feet, relative to peak 2007 conditions. Maximum surge also increases up to 6 ft in the poorly protected Belle Chase region with effective focusing of surge in this concavity in the system. There was very little change in the maximum wave heights outside the hurricane protection system (Figure A.3-2).

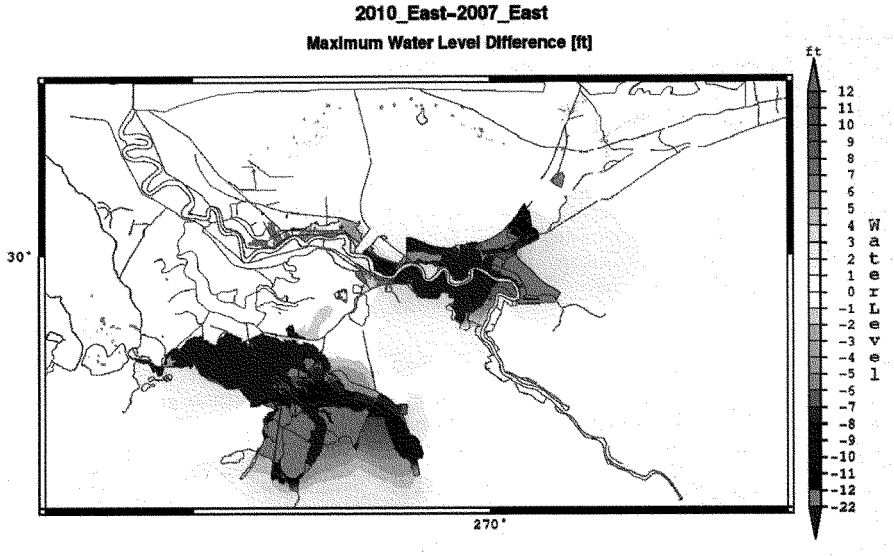

Figure A.3-1. Difference in maximum surge level between the 2010 levee configuration and the 2007 base case for the 2010 storm suite.

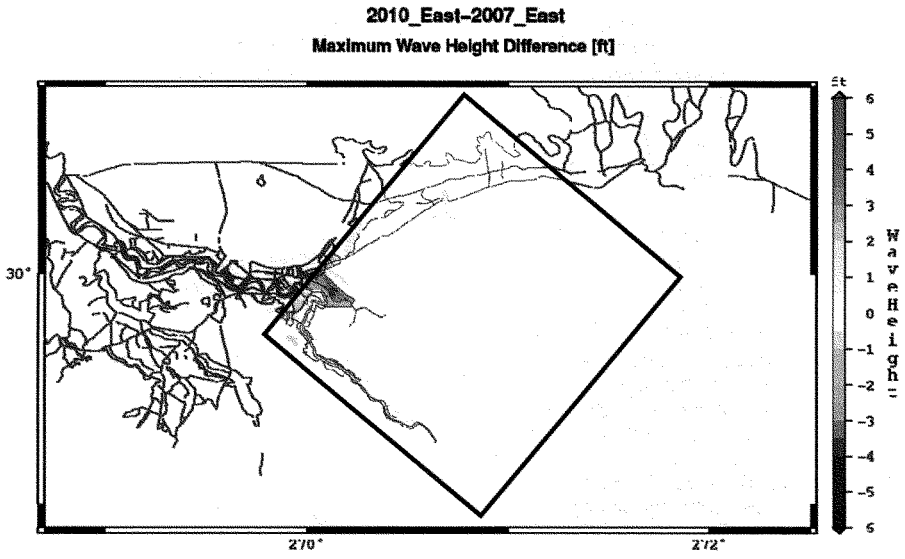

Figure A.3-2. Difference in maximum wave height (ft) between the 2010 levee configuration and the 2007 base case for the 2010 storm suite for the southeast STWAVE grid.

A.4 Closure options of Lake Pontchartrain

Various levee system alternatives were developed to understand the performance and implications of a variety of levee system improvements. A more detailed description and the suite of storms simulated for each alternative is provided in Volume I of this report, Background and Methodology. Four east levee configurations included closure options of Lake Pontchartrain. This section documents results for east alternatives A through D as they relate to the proposed closures of Lake Pontchartrain.

A.4.1 Full closure along US90 (Alternative EA)

Alternative EA involves complete closure of Lake Ponchartrain through a 30-mile non-overtopping levee extending from the junction of US11 and US90 to a point just north of Slidell near I-59. The levee generally follows the US90 corridor, extending along the west bank of Lake Borgne, the west boundary of Lake Saint Catherine, and crossing through the Chef Menteur and Rigolets Passes. Both the passes would be entirely closed during storms with gates. At the intersection of US90 and US190, the levee stretches northwestward, ultimately terminating at a point approximately one mile north of the I-10/I-12/I-59 interchange near Slidell, LA. Another feature of the alternative EA configuration is a non-overtopping levee extending along the west shore of Lake Borgne, providing full closure at the funnel for the Inner Harbor Navigation Channel and the Gulf Intracoastal Waterway.

Figure A.4-1 depicts the difference in maximum water level between the alternative EA and 2007 base case. Non-overtopping levees and channel closures prevent the filling of Lake Pontchartrain from Lake Borgne and the Mississippi Sound and thus prevent the increase in mean lake levels. Local winds still drive a localized set up in the wind direction causing high water levels along the shores of the lake. A 2 to 3 ft decrease is predicted for lakefront areas at the city of New Orleans, with maximum surge levels in the neighborhood of 10 to 12 ft. Along the north shore of Lake Pontchartrain, areas such as Mandeville and Lacombe are expected to experience a maximum surge decrease ranging between 2 and 7 ft. Maximum surge levels along the north shore of Pontchartrain are predicted to be between 9 and 14 ft. At the western shore of Lake Pontchartrain, surge reductions are predicted to be between 3 and 4 ft. In addition, the entire south levee of New Orleans East, about half the north levee of St. Bernard and the southern reach of the IHNC experience vast reductions in surge levels and/or see little or no surge against them as a result of the closure on the west shore of Lake Borgne.

Areas seaward of the levee experience water level increases. As inland surge propagation accumulates along the seaward side of the non-overtopping levee, maximum levels within Lake St. Catherine are predicted to increase by as much as 8 ft in comparison with the 2007 base levels, with a maximum predicted level of approximately as much as 30 ft. Increases in maximum surge levels are expected to be around 4 ft along the mouth of the Pearl River, and 7 ft near Bayou Savage National Wildlife Refuge within Lake Borgne. At the non-overtopping levee that encloses the funnel, surge levels are expected to increase by between 6 and 8 ft from the base 2007 levels. The effects of alternative EA may be felt as far east as Long Beach, MS, with a surge increase of 1 ft, and maximum levels between 22 and 24 ft.

In Figure A.4-2, the difference in maximum wave heights between alternative EA and the base case are presented. Wave heights increase seaward of the full-closure funnel levee between 0.5 and 1 feet. Wave heights increase seaward of the non-overtopping US90 Pontchartrain levee by up to 3 ft.

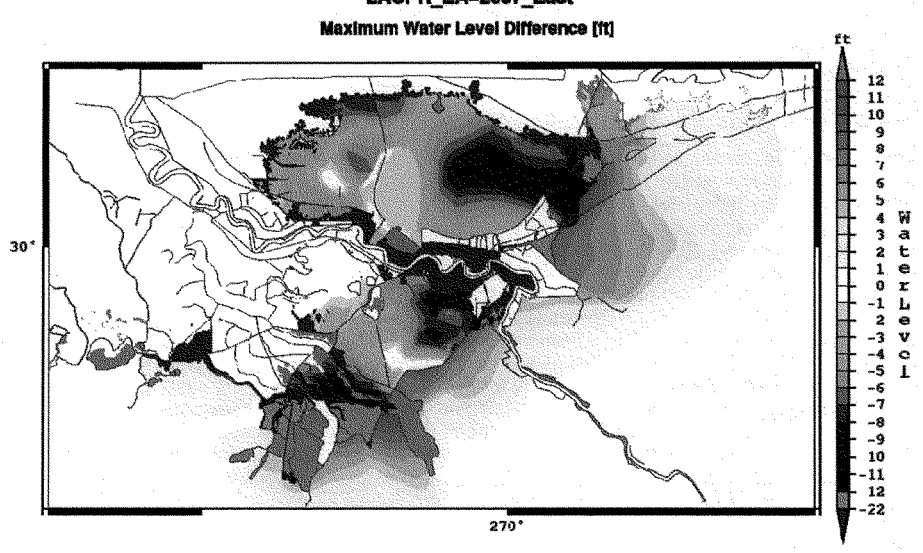

Figure A.4-1. Difference in maximum surge level between alternative EA and the base case for the EA storm suite.

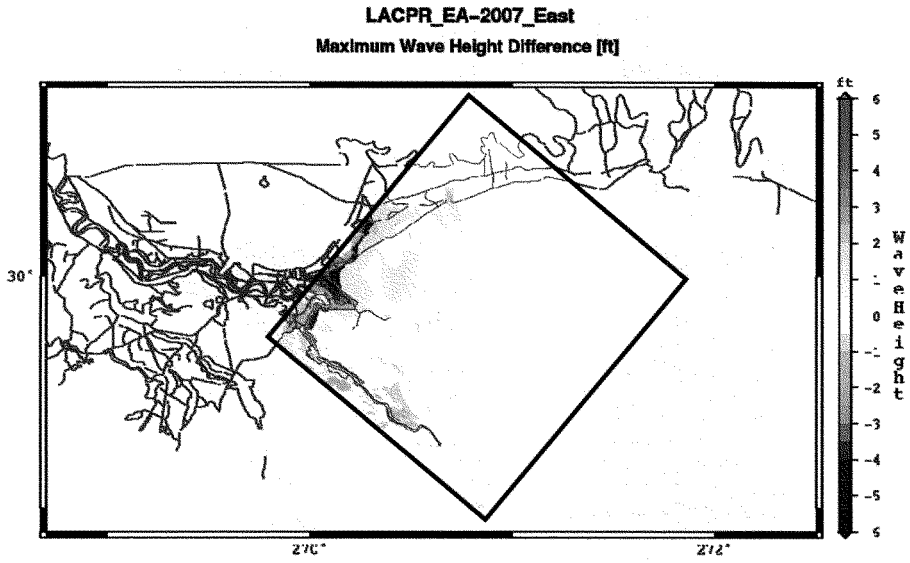

Figure A.4-2. Difference in maximum wave height (ft) between alternative EA and the base case for the EA storm suite for the southeast STWAVE grid.

A.4.2 Weir closure along US90 (Alternative EB)

Alternative EB is identical to EA with the exception that the levee across the Pontchartrain land-bridge from Chef Menteur to the Rigolets was lowered from a height not to be overtopped to 12 ft. Figure A.4-3 is the difference in the envelope of maximum water level between alternative EB and the 2007 base case. The weir levee stops the propagation of the surge into Lake Pontchartrain until its elevation exceeds 12 ft, resulting in lower peak surges in Lake Pontchartrain relative to the base case. However, for large storms on select tracks there will still be considerable infilling of Lake Pontchartrain from Lake Borgne and the Mississippi Sound. The surge on the south shore of Pontchartrain is reduced 2 to 3 ft relative to the 2007 base case and reductions of up to 3 to 4 ft are predicted on the north shore. The maximum surges on the south shore with alternative EB in place range from about 9 to 13 ft, increasing as you move from west to east. On the north shore, the peak surges for alternative EB are greater than 12 ft from about Madisonville to the east. The inclusion of the weir levee across the Pontchartrain land bridge increases water level seaward of the weir 3 to 4 ft and near Slidell 2 to 4 ft. The peak surge at the proposed levee that closes the funnel is increased 5 to 6 ft. In comparison to alternative EA, the surge reductions in Lake Pontchartrain due weir closure are less, however they are not significantly different on the south shore. The surge seaward of the structure is also less, with a maximum increase of 8 ft for alternative EA, compared to an increase of 4 ft for alternative EB. Figure A.4-4 depicts the maximum wave height differences for alternative EB. Wave height differences for areas seaward of the full-closure funnel levee range between 4 and 5 feet. Wave heights are predicted to increase inland of the US90 Pontchartrain weir by up to about 2 ft.

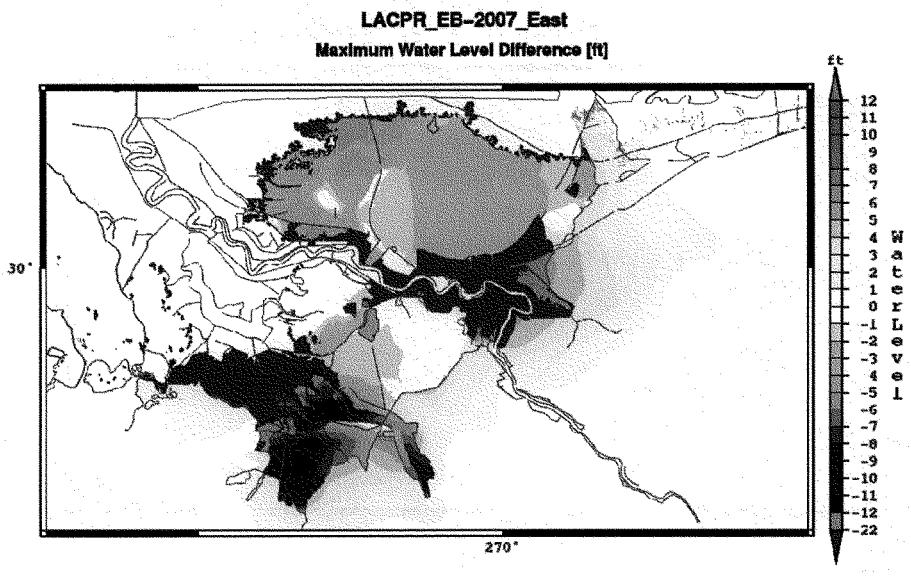

Figure A.4-3. Difference in maximum surge level between alternative EB and the base case for the EB storm suite.

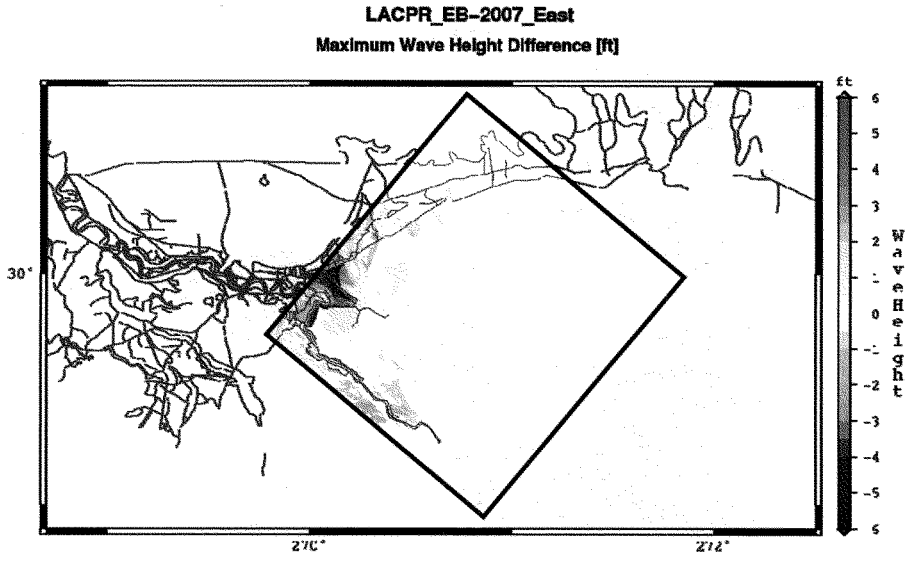

Figure A.4-4. Difference in maximum wave height (ft) between alternative EB and the base case for the EB storm suite for the southeast STWAVE grid.

A.4.3 Partial closure along US90 (Alternative EC)

The levee configuration for alternative EC involves a non-overtopping levee following the same path as alternative EA. Alternative EC differs from alternative EA in that it includes openings at Chef Menteur Pass and the Rigolets. These openings allow water to flow through these deep conveyance channels between Lake Pontchartrain and Lake Borgne. Alternative EC maintains the closure at the funnel through a non-overtopping levee placed along the west bank of Lake Borgne. Figure A.4-5 illustrates the difference between the maximum water level for alternative EC and Base 2007. Alternative EC's partial closure configuration results in a maximum increase within Lake Pontchartrain of less than 3 ft. Areas along Pontchartrain's north shore are predicted to experience a surge reduction of less than 2 ft, with similar reductions in the vicinity of New Orleans. Maximum water levels within Lake Pontchartrain range from 11 to 15 ft along the northern shore from Mandeville, LA to Lacombe, LA. Water levels as high as 18 ft are predicted for areas between Kenner, LA and Frenier, LA. Seaward of the levee, an accumulation of surge forms, resulting in an increase of alternative EC water levels in comparison with the Base 2007 levels. The greatest surge increase due to the EC configuration is 6 ft, occurring within Lake Saint Catherine. Maximum surge level increases range between 5 and 6 ft at points along the seaward side of the levee that provides closure to the funnel. The effects of the levees in alternative EC are confined to areas west of Waveland, MS, where surge increases due to the modified levees are predicted to be less than 1 ft. In comparison with alternatives EA and EB, alternative EC provides less reduction in surge levels within Lake Pontchartrain. For example, Lacombe, LA is predicted to experience only a 1 ft surge reduction due to alternative EC compared to 7 and 3 ft for alternatives EA and EB, respectively. Maximum wave height differences for alternative EC are presented in Figure A.4-5. Wave heights seaward of the full-closure funnel levee are not predicted to increase. Wave heights are predicted to increase at the partial closure US90 Pontchartrain levee by up to 1 ft.

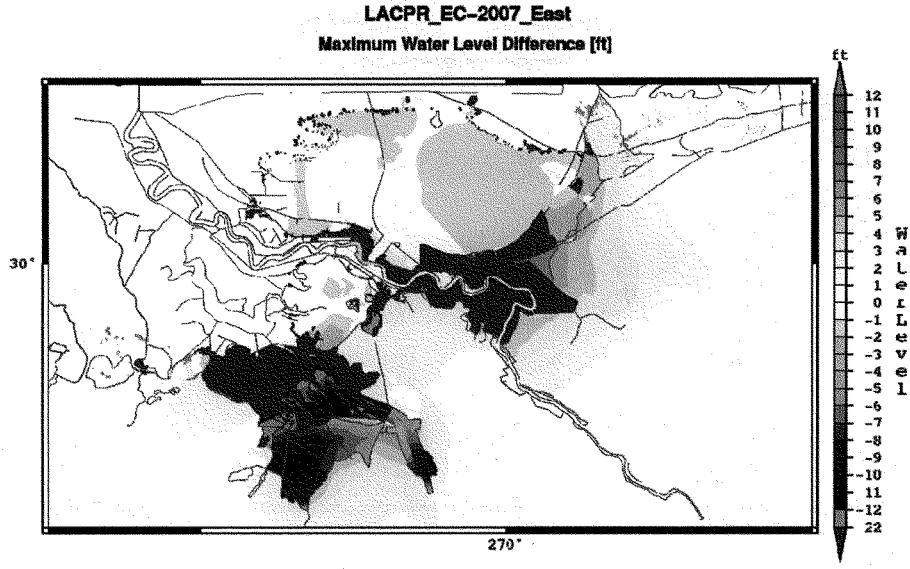

Figure A.4-5. Difference in maximum surge level between alternative EC and the base case for the EC storm suite.

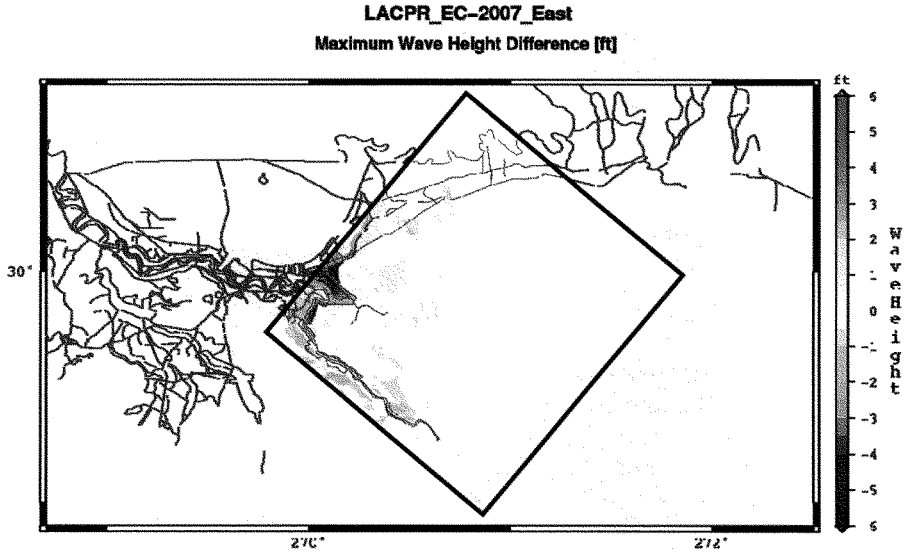

Figure A.4-6. Difference in maximum wave height (ft) between alternative EC and the base case for the EC storm suite for the southeast STWAVE grid.

A.4.4 Full closure through Lake Borgne (Alternative ED)

In alternative ED, a non-overtopping levee isolates Lake Pontchartrain from Lake Borgne. This levee extends from Verret, LA, northward through Lake Borgne along a straight line to a point along the Gulf Intracoastal Waterway just south of Little Lake. At this point, the levee extends northwestward along the Pearl River corridor, crossing I-10, and terminating at a point approximately one mile north of the I-10/I-12/I-59 interchange. This approximately 35 mile levee provides full closure to Lake Pontchartrain, Lake Saint Catherine, the Chef Menteur Pass, the Rigolets, the funnel, and portions of southeastern Lake Borgne. The non-overtopping levee in alternative ED results in an inland surge decrease accompanied by seaward accumulation of water. Figure A.4-7 illustrates the difference between the maximum water level for alternative ED and Base 2007. Maximum surge decreases inland of the non-overtopping levee are predicted to be over 12 ft at areas within Lake Borgne and Lake Saint Catherine. From Mandeville, LA to Lacombe, LA, predicted maximum surge decreases range from 3 to 6 ft along Pontchartrain's north shore. Areas along the southern shore show a 2 to 3 ft decrease from the Base 2007 levels. Maximum water levels within Lake Pontchartrain range from 3 to 15 feet, with the highest levels occurring along Pontchartrain's southwestern shore, between Kenner, LA and Frenier, LA. Maximum levels along Lake Pontchartrain at New Orleans were between 9 and 12 ft. Simulation of the alternative ED configuration depicts the maximum water level occurring seaward of the non-overtopping levees. For areas seaward of the non-overtopping levee within Lake Borgne, maximum water levels are approximately 30 ft. The effects of the alternative ED levee are predicted to be felt as far east as Gulfport, MS, where surge levels increase less than one foot due to levee implementation. In regards to surge reductions within Lake Pontchartrain, alternative ED is comparable to alternative

EA. Alternative ED is unique in that it provides dramatic reductions within Lake Saint Catherine and portions of Lake Borgne, whereas alternative EA increases levels in these areas. Figure A.4-8 depicts maximum wave height differences for alternative ED. Wave heights seaward of the full closure levee dividing Lake Pontchartrain and Lake Borgne are predicted to increase up to approximately 2 ft. Wave heights in the Caernarvon area are expected to increase by less than 1 ft.

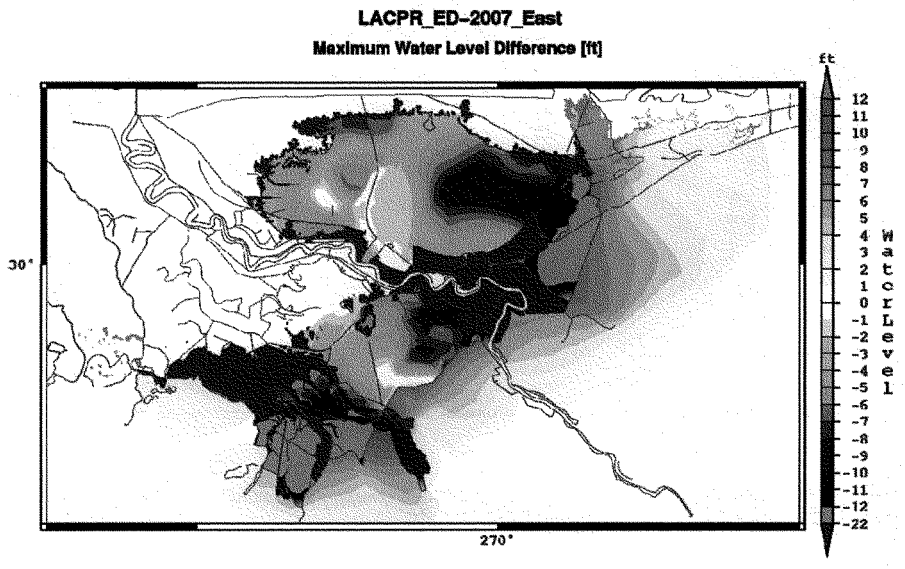

Figure A.4-7. Difference in maximum surge level between alternative ED and the base case for the ED storm suite.

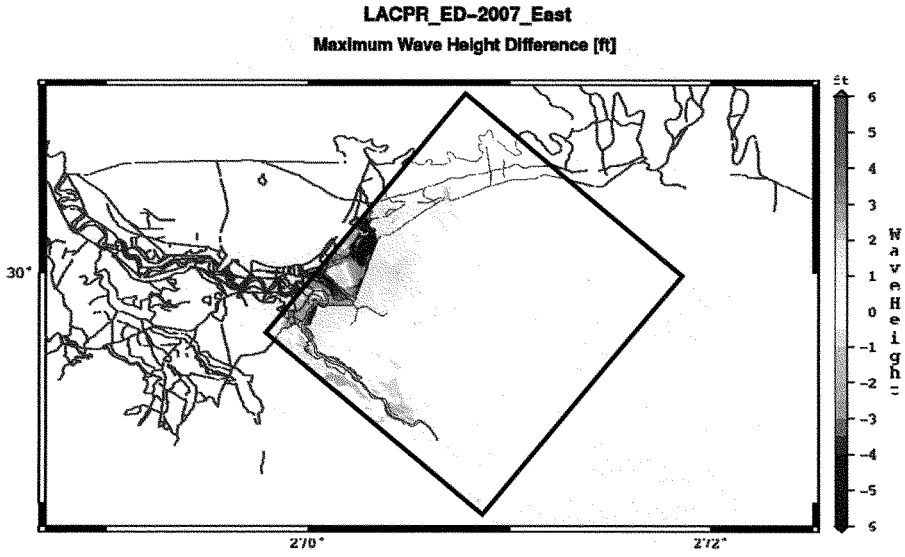

Figure A.4-8. Difference in maximum wave height (ft) between alternative ED and the base case for the ED storm suite for the southeast STWAVE grid.

A.5 Plaquemines

This section documents the results of the study that explored the influences of the lower Plaquemines Parish levee system on storm surge. The height of the levee system range from 16 to 18 ft, the natural floodplains of the Mississippi range from 3 to 8 ft. From Jesuit Bend, the levee system has a length of 24 mi on the eastside and 57 mi on the west side of the river. The total length of the delta is 81 mi from Jesuit Bend to the birds-foot. A suite of 18 hypothetical storms were simulated in order to evaluate three different configurations of the levee system: One represents the 2007 base case and serves as the reference case; the second, Plaquemines 1, introduces three spillways across the levee system with a total length of 9.5 mi; and the last one, Plaquemines 2, represents the situation of having no levees along the delta, 57 mi of levees has been removed.

A.5.1 Plaquemines 1

The purpose of the spillways is to provide a hydrodynamic connection between the west- and eastside of the delta, which should result in reduced surge levels on the upwind side of the delta. Figure A.5-1 shows the difference in peak values between Plaquemines 1 and the 2007 base case. The spillways reduce the maximum water levels 1 to 2 ft on the northeastern part of the delta, around the first spillway. At Pointe a la Hache, at the second spillway, the maximum reduction in peak water levels is about 2 ft. On the west side of the delta, around Barataria, maximum surge levels are increased 1 to 2 ft. Changes in maximum water levels due to the spillways only occur

along the delta. Surge that propagates upriver spills out through the northern spillway, reducing the surge that propagates to New Orleans. Figure A.5-2 shows the difference in maximum wave heights between Plaquemines 1 and the 2007 base case. Waves are predicted to locally increase by less than 3 ft at the spillways while there are some small decreases in waves east of the spillways.

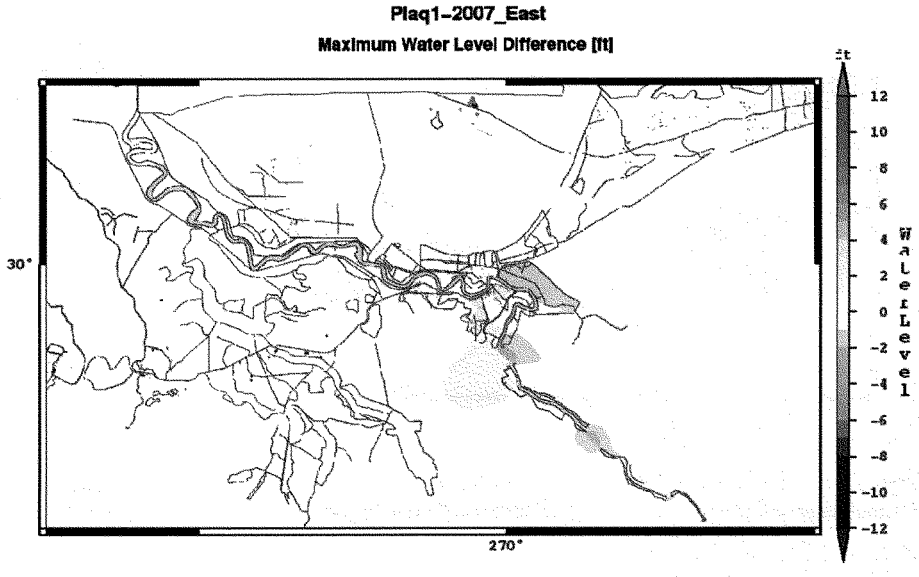

Figure A.5-1. Difference in maximum surge level between Plaquemines 1 and the base case for the Plaquemines storm suite.

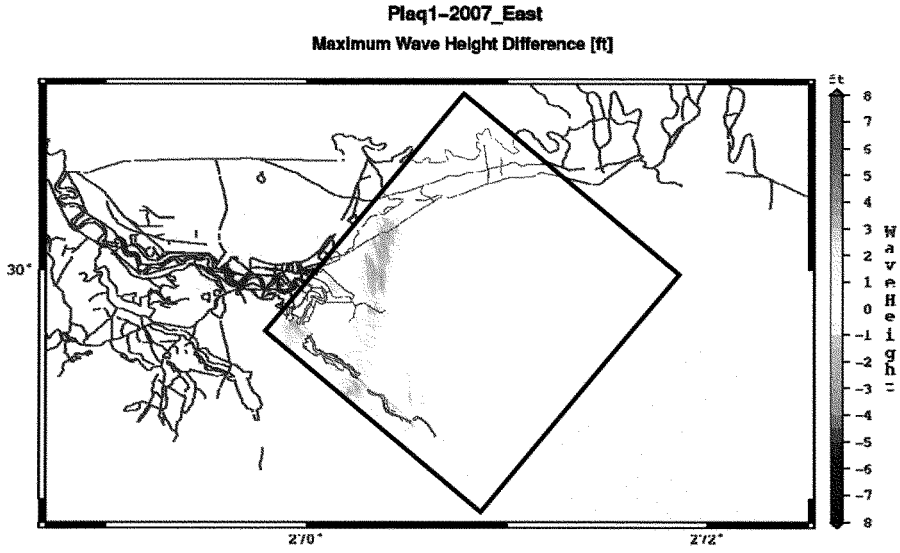

Figure A.5-2. Difference in maximum wave height (ft) between Plaquemines 1 and the base case for the Plaquemines storm suite for the southeast STWAVE grid.

A.5.2 Plaquemines 2

The configuration without having levees along the delta provides an upper limit in creating a hydrodynamic connection between the west- and eastside of the delta and represents a more natural system as well. Figure A.4-3 plots the difference in peak values between Plaquemines 2 and the 2007 base case. Completely removing levees reduces the maximum flood levels 3-8 ft along the levees of St. Bernard and Plaquemines Parish. The flood levels increase less than 1 ft at Barataria, on the westside of the delta, less than in the case with spillways. Surge that propagates upriver towards New Orleans is reduced due to the surge that propagates across the delta; and because the surge moving up the river spills out before reaching Jesuit Bend. Figure A.5-4 shows the difference in maximum wave heights between Plaquemines 2 and the 2007 base case. Waves are predicted to decrease 2 to 3 ft along the levees of St. Bernard and Plaquemines Parish as well as in the Caernarvon and Biloxi Marsh areas. Waves on the east bank of the river are predicted to decrease up to 6 ft.

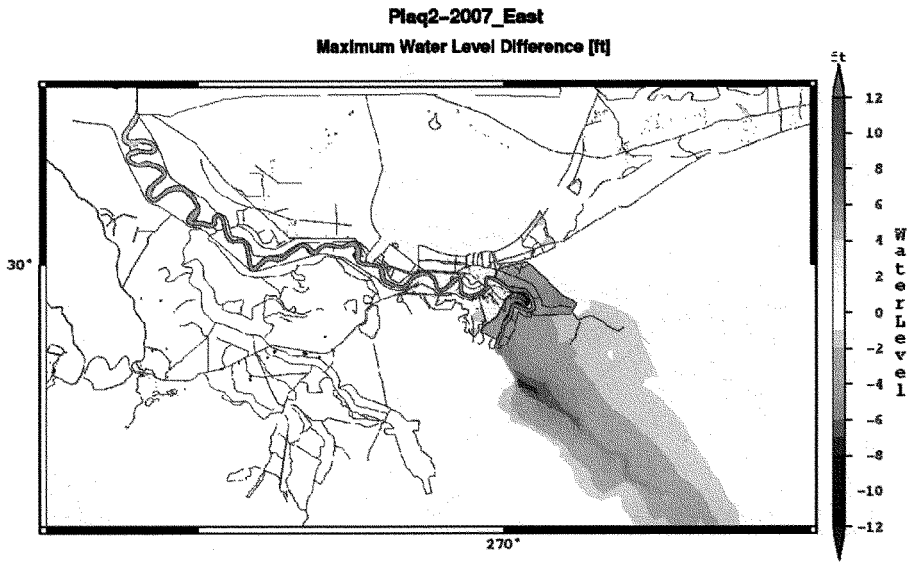

Figure A.5-3. Difference in maximum surge level between Plaquemines 2 and the 2007 base case for the Plaquemines storm suite.

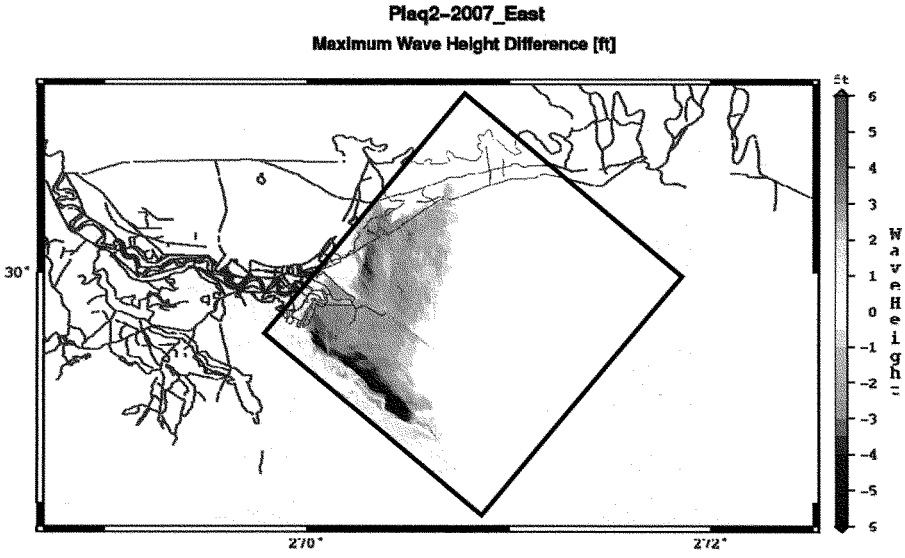

Figure A.5-4. Difference in maximum wave height (ft) between Plaquemines 2 and the base case for the Plaquemines storm suite for the southeast STWAVE grid.

A.6 Closure options for Barataria Basin

Various levee system alternatives were developed to understand the performance and implications of a variety of levee system improvements. A more detailed description and the suite of storms simulated for each alternative is provided in Volume I of this report, Background and Methodology. Two east levee configurations included closure options for the Barataria Basin. This section documents results for east alternatives EA and EB as they relate to the proposed closure of Barataria Basin.

A.6.1 Full closure (Alternative EA)

In this region, alternative EA includes the non over-topping Morganza to the Gulf levee as in the 2010 system and adds a non-overtopping levee from the west bank of the Mississippi River from Belle Chasse to Larose along the alignment of the GIWW. On the east, the closure ties into the Mississippi river levee and on the west, the closure ties into the Larose and Golden Meadow flood protection structure. The difference in the envelope of maximum water level between alternative EA and the 2007 base case is given in Figure A.4-1. The non over-topping levee effectively eliminates the propagation of the surge into the Barataria Basin where water levels in the protected region behind the levee are shown to be reduced by as much as 10 ft in some areas, including along the West Bank. However, the inclusion of the non over-topping levee across the opening of the Barataria Basin does increase water levels seaward of the structure by up to 12 ft because the hurricane wind and waves force the water to pile up against the levee structure. Water surface elevations to the west of Golden Meadow are essentially the same as in the 2010 configuration.

A.6.2 Weir closure (Alternative EB)

Alternative EB includes a lower 20 ft overtopping Morganza to the Gulf levee and in addition lowers the closure across Barataria Basin from a height not to be overtopped to 12 ft. The difference in the envelope of maximum water level between alternative EB and the 2007 base case is given in Figure A.4-3. While the weir reduces the flooding from smaller storms, it is over-topped by larger storm events. The Barataria levee does slow surge propagation northward, but the additional defenses along the west bank increase the surge in this area. Consequently, maximum surge values north of the Barataria closure are not changed much relative to the base case. However, the inclusion of the weir across the opening of Barataria Basin still stops or reduced northerly flows and therefore does increase peak water levels seaward of the weir by up to 4 ft. Surge is also amplified in this region by raising of the Mississippi river levees (see 2010 Base condition) which prevents water from escaping from west bank to the Mississippi River. Without this relief, the water piles up higher against the seaward side of the Barataria Basin levee. Since the weir in this configuration will allow over-topping, some regions immediately landward from the structure show increases in maximum surge levels as a result of the water which piles up on the landward side of the structure. The increases within the protected region are very localized. A benefit that is not revealed by the maximum plots is that many smaller storms will generate significantly less surge behind the weir compared with the base case, for instance a reduction in the 100-yr water level is expected and this is the intent of such a weir structure. Water surface elevations to the west of Golden Meadow are similar to the 2010 configuration although more flow propagates northward behind the lowered system.

A comparison of difference in the envelope of maximum water surface elevations between alternatives EA and EB reveals that Barataria Basin is significantly better protected against large events by the non-overtopping levee. Reducing the crest elevation of the levee to permit over-topping in EB results in minimal protection from large storm events, but does provide protection for storms near and below the 100-year level. On the seaward side of the levee, the higher levee of EA induces a much greater increase in maximum surge along the structure. Conversely, the lower EB weir allows relief during the larger storms that can overtop it, thus its impact seaward of the structure is not as pronounced. In summary, the EA structure is more effective at reducing surge in Barataria Basin but causes greater increase in surge on the seaward side, while the EB structure reduces surge in Barataria Basin from smaller storms.

A.7 West Alignments

Various levee system alternatives were also developed for the western part of the state. A description and the suite of storms simulated for the west and each alternative is provided in Volume I of this report, Background and Methodology. Three west levee configurations were simulated and this section documents results for west alternatives A through C.

2.7.1 2007 Base Condition

The 2007 Base condition was created to represent South Louisiana as it was projected to exist at the start of the 2007 hurricane season. Post Hurricane Katrina and Rita topographic and bathymetric conditions were used. Simulations were completed for the 152 storms for western Louisiana in order to define water levels and corresponding wave conditions. This information serves as a base condition to which alternative levee systems, marsh improvements and/or degradation, and sea level rise can be compared. Section 2.4 of Volume 1 of this report (and section A.4 of this Annex)

describes the motivation for study and changes to the physical system for each alternative inspected.

Figure A.7-1 represents the maximum surge level recorded for the West 152 storms simulated for Southeastern Louisiana. It is important to note that the displayed water levels are not stochastic representations of the 100-year or other return period water elevations, but rather are the maximum surge levels for all 152 specific storms simulated for the JPM-OS method. The highest surge levels in Louisiana occurred along the coast of Cameron Parish. Peak surge levels along the Cameron Parish levees are on the order of 24 ft. South of Lafayette, LA, peak surge levels reach approximately 21 ft. Maximum 2007 wave heights, as predicted by STWAVE simulations (Figure A.7-2), are on the order of 18 feet near the coast but break there and are greatly reduced inland.

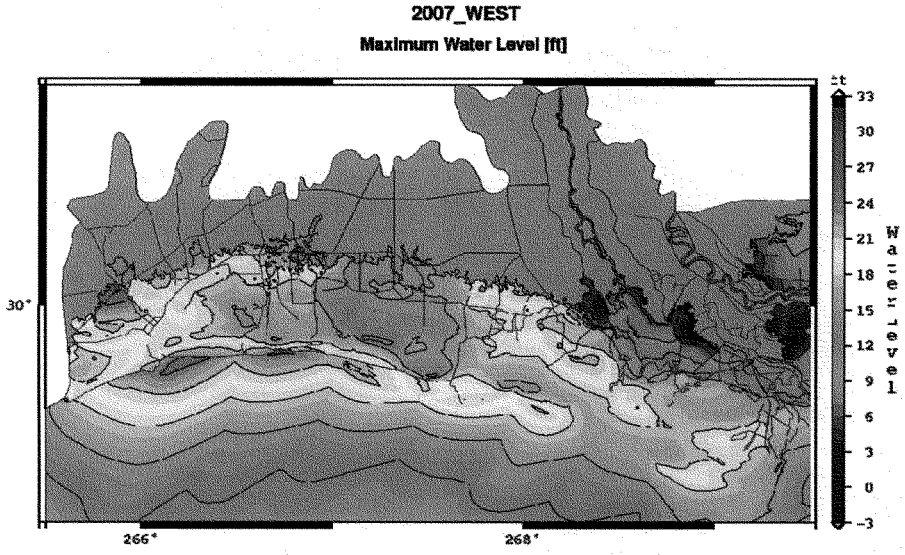

Figure A.7-1. Maximum surge level (ft) for the LAWEST 2007 base case for all West 152 storms.

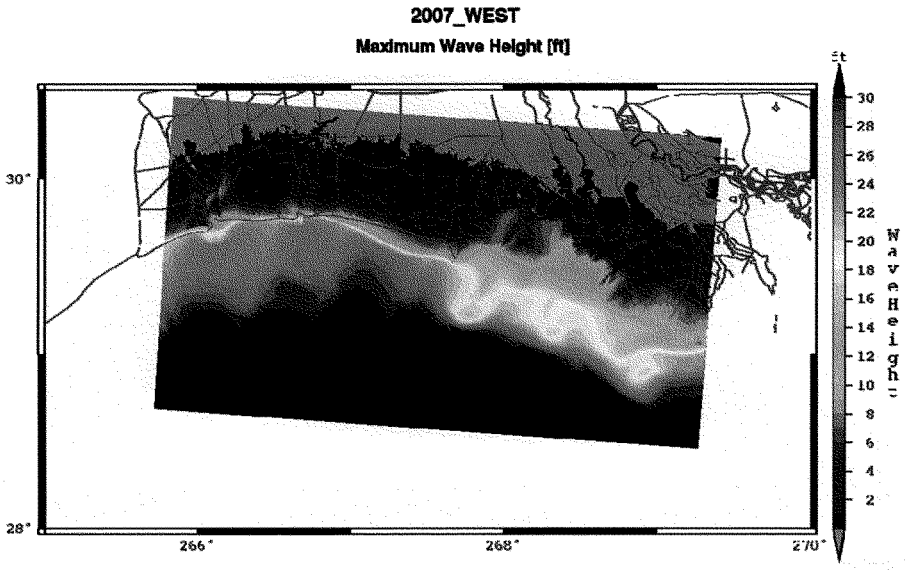

Figure A.7-2. Maximum wave height (ft) for the LAWEST 2007 base case for all 152 storms.

A.7.2 Alternative WA

Alternative WA includes a non-ovetopping levee that runs along the GIWW across all of western Louisiana from the Atchafalaya River to Vinton. West of Vinton, the levee turns north and runs to higher ground. The difference in the envelope of maximum water level between alternative WA and the 2007 base case is given in Figure A.7-3. The non over-topping levee eliminates the propagation of the surge north of the levee where water levels in the protected region behind the levee are shown to be reduced by greater than 10 ft in some areas. However, the inclusion of the non over-topping levee does increase water levels seaward of the structure by up to 6 ft because the hurricane wind and waves force the water to pile up against the levee structure. Maximum wave heights (Figure A.7-4) are reduced by 1 to 3 ft in some areas north of the proposed levee. Less than a 1 ft increase in wave height is predicted seaward of the levee.

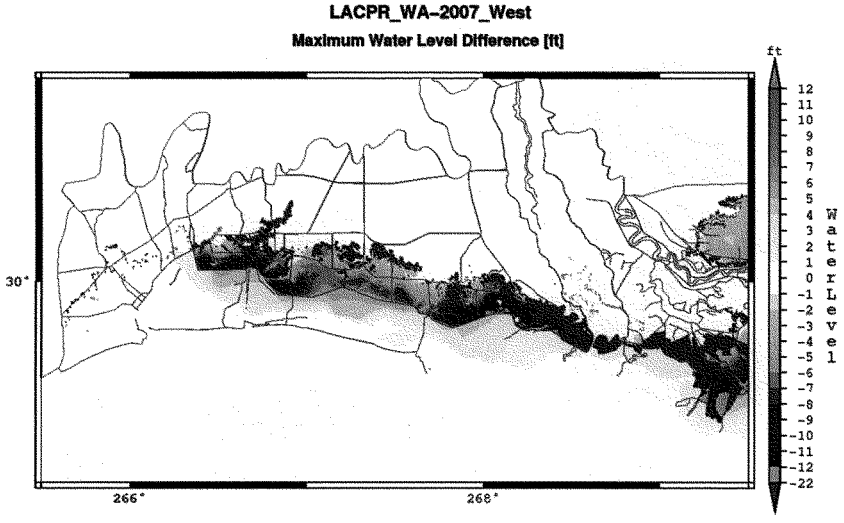

Figure A.7-3. Difference in maximum surge level between alternative WA and the LAWEST base case for the WA storm suite.

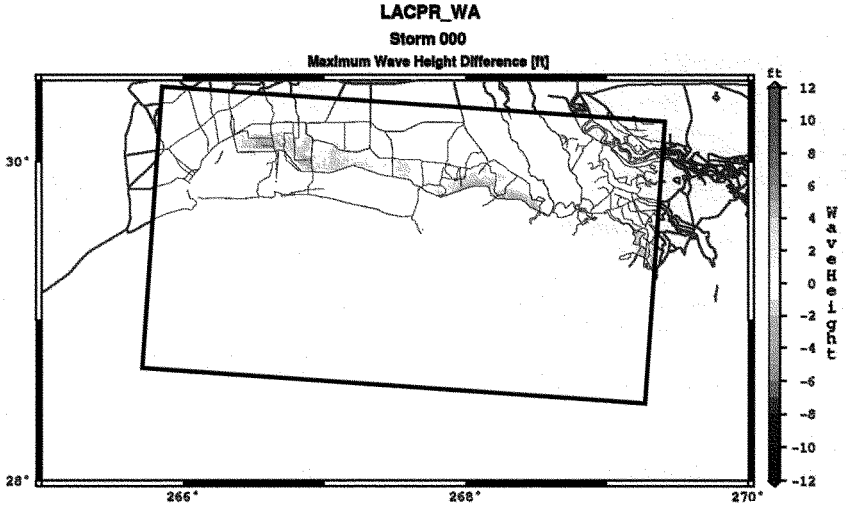

Figure A.7-4. Difference in maximum wave height (ft) between alternative WA and the LAWEST base case for the WA storm suite for the west STWAVE grid.

A.7.3 Alternative WB

Alternative WB includes a non-ovetopping levee that runs from the Atchafalaya River to Abbeyville, north of the GIWW. At Abbeyville, the levee turns north and runs west of Lafayette to higher ground. This alternative also includes small ring levees around Gueydan and Kaplan. Further to the west, a large ring levee extends from east of Lake Charles to west of Vinton protecting these areas. The difference in the envelope of maximum water level between alternative WB and the 2007 base case is given in Figure A.7-5. The non over-topping levees eliminate the propagation of the surge north of the levee where water levels in the protected region behind the levee are shown to be reduced by up to 10 ft in some areas and flooding is eliminated in many areas. The inclusion of the non over-topping levees does increase water levels seaward of the structures. Maximum wave heights (Figure A.7-6) are changed by less than 1 ft.

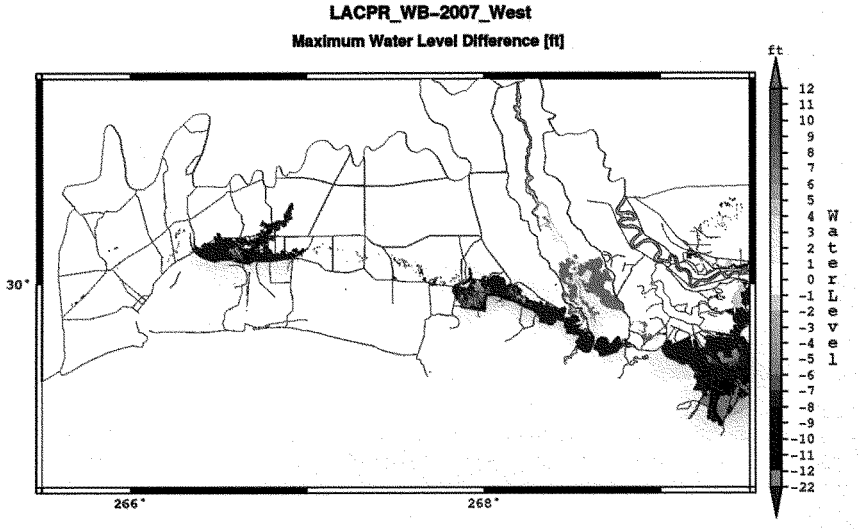

Figure A.7-5. Difference in maximum surge level between alternative WB and the LAWEST base case for the WB storm suite.

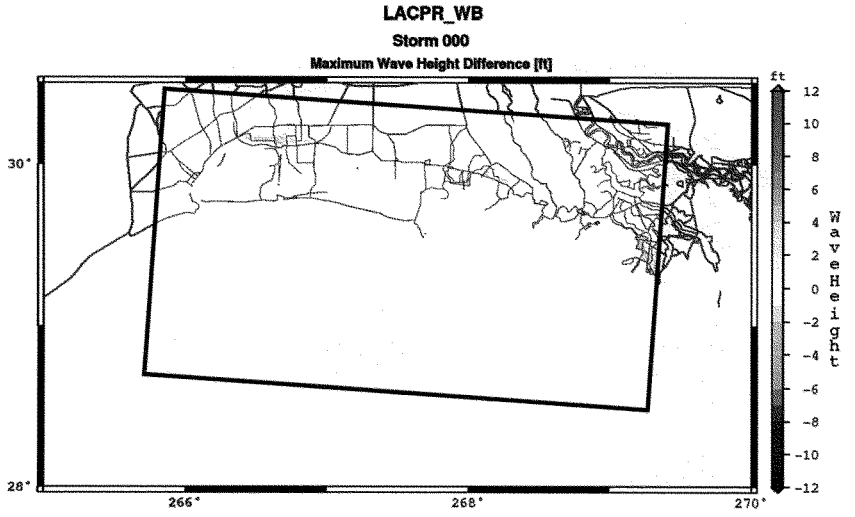

Figure A.7-6. Difference in maximum wave height (ft) between alternative WB and the LAWEST base case for the WB storm suite for the west STWAVE grid.

A.7.4 Alternative WC

Alternative WC is similar to WB except that a 100-year protection levee extends along the GIWW from Vermillion Bay to Calcasieu Lake and there are no ring levees around Gueydan and Kaplan. The difference in the envelope of maximum water level between alternative WC and the 2007 base case is given in Figure A.7-7. The results are similar to the other alternatives. The non over-topping levees eliminate the propagation of the surge north of the levee where water levels in the protected region behind the levee are shown to be reduced by up to 10 ft in some areas and flooding is eliminated in many areas. The 100-year protection levee also greatly reduces surges, with decreases from 1 to 10 ft across the protected area. The inclusion of the levees does increase water levels seaward of the structures. Maximum wave heights (Figure A.7-8) are generally changed by less than 1 ft with 1 to 2 ft reductions in some areas behind the proposed levees.

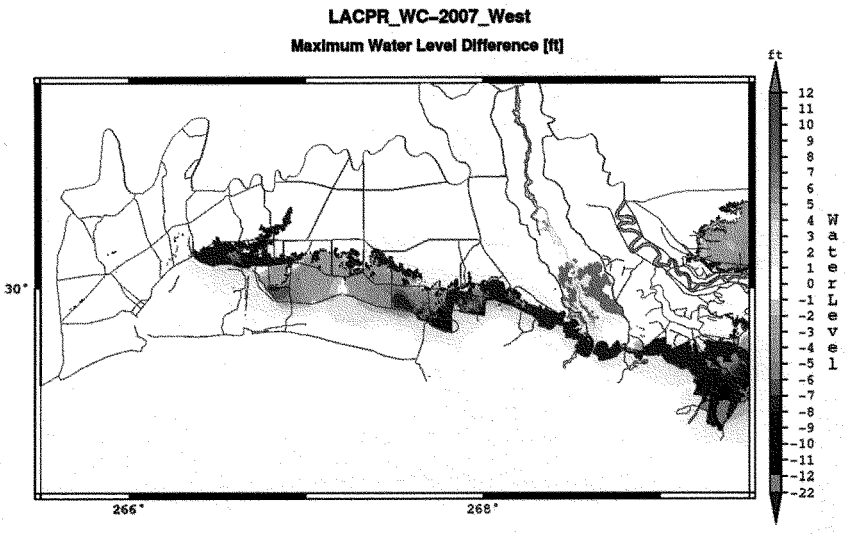

Figure A.7-7. Difference in maximum surge level between alternative WC and the LAWEST base case for the WC storm suite.

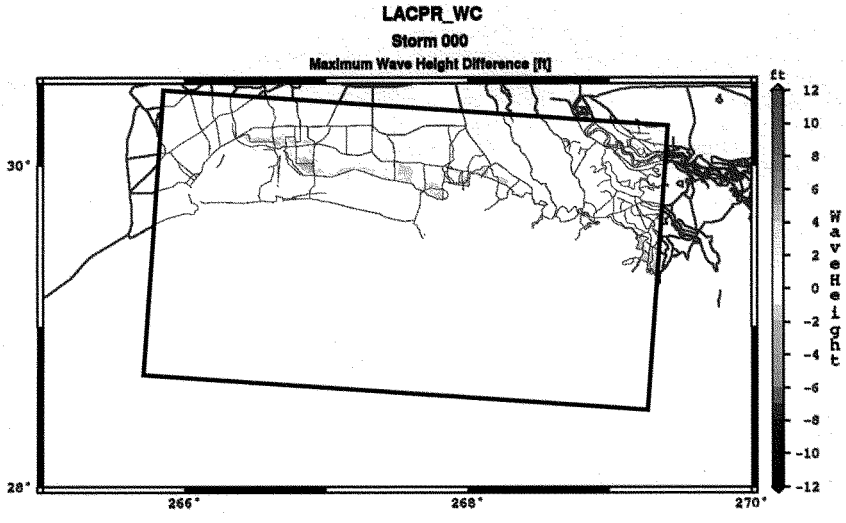

Figure A.7-8. Difference in maximum wave height (ft) between alternative WC and the LAWEST base case for the WC storm suite for the west STWAVE grid.

A.8 Future Conditions

A sensitivity analysis was performed to assess the impact of sea level rise and bathymetric and frictional resistance changes on ADCIRC-simulated peak surge elevations and STWAVE-simulated waves. Topography, landscape features, and vegetation have the potential to reduce storm surge elevations and absorb wave energy. Land elevations greater than the storm surge elevation act as a physical barrier and create bathymetric resistance for the surge and waves. Landscape features such as barrier islands also have the potential to create frictional resistance and affect storm surge and wave energy even when below the surge elevation. The influence of these features is reduced if they are lost or inundated due to sea level rise. This section is used to assess the impact of barrier island and marsh features on storm surge and wave energy at the mainland coast and to evaluate the impact of sea level rise.

A.8.1 Barrier Islands

The barrier island configurations modeled were: 1) the existing 2007 base Post-Katrina degraded condition; 2) no barrier islands with open water Manning's n value = 0.02 (BI-1); and 3) a restored barrier island configuration of 12 ft (NAVD88 2004.65) for Cat Island, Ship Island, Horn Island, Petit Bois Island, and Dauphin Island and 6 ft (NAVD88 2004.65) for the Chandeleur Islands (BI-2), 4) the existing Post-Katrina degraded condition with a forest Manning's n = 0.15 (BI-3), and 5) a restored barrier island configuration with a forest Manning's n = 0.15 (BI-4). The BI-4 configuration is identical to the restored BI-2 configuration except a Manning's n=0.15 friction has been applied to the islands to represent the increased frictional resistance associated with a forest. Results from the extreme cases, BI-1 and BI-4, are presented here.

The difference in the envelope of maximum water level between the BI-1 configuration and the 2007 base case can been seen in Figure A.8-1. The largest increase in surge seen by the removal of the barrier islands is approximately 1.7 ft in Chandeleur Sound. Outside of Chandeleur Sound, the surge increases less than a foot nearly everywhere when compared to the 2007 base case. Lake Borgne demonstrates a rise in surge of approximately 0.7 ft. The removal of the barrier islands also increases surge by as much as 1.5 ft in the Bay St. Louis, MS area. The maximum wave height differences predicted for the BI-1 configuration can be seen in Figure A.8-2. Maximum wave heights are expected to increase outside of the east New Orleans levee system up to 2 feet near Caernarvon and the IHNC/MRGO funnel. Maximum wave heights are predicted to increase by as much as 3 ft in the Bay St. Louis, MS area. The largest increase in maximum wave heights (greater than 8 ft) is observed immediately landward of the barrier islands.

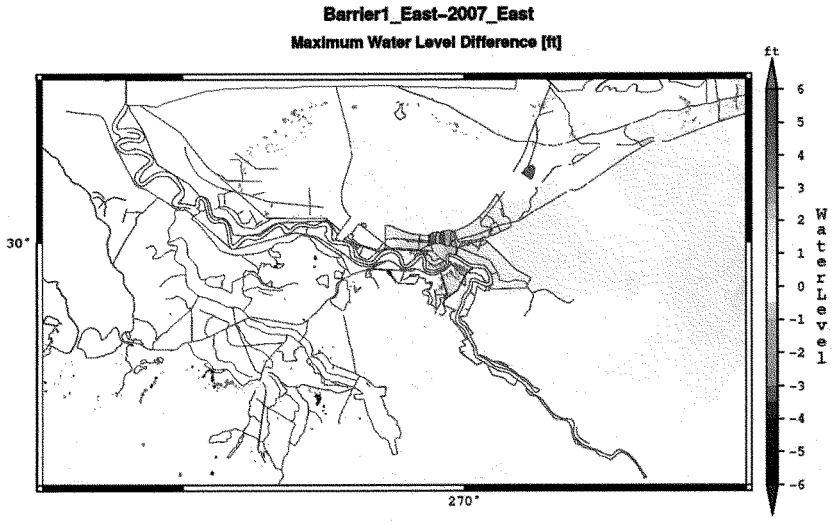

Figure A.8-1. Difference in maximum surge level (ft) between the BI-1 configuration and the base case for the Barrier Islands storm suite.

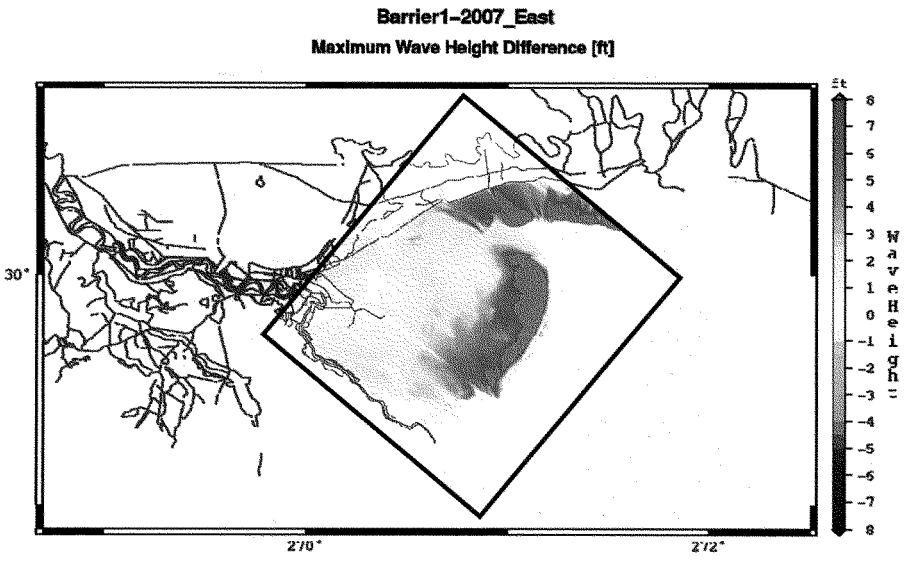

Figure A.8-2. Difference in maximum wave height (ft) between BI-1 and the base case for the Barrier Islands storm suite for the southeast STWAVE grid.

Figure A.8-3 is the difference in the envelope of maximum water level between the BI-4 configuration and the 2007 base case. The figure demonstrates the potential effects of the restored barrier islands configuration with reductions in maximum surge generally 2 ft or less at the levees when compared to the 2007 base case. There is less than 1 ft decrease in surge in Lake Pontchartrain. The restored barrier islands also decrease surge by approximately 2 ft in the Bay St. Louis, MS area. The surge in Chandeleur Sound just behind the island is reduced by as much as 6 ft with for this restoration scenario, but the reduction is less than one foot at the Plaquemines and St. Bernard levees. Figure A.8-4 shows the maximum wave height differences for the BI-4 configuration. Maximum wave heights are expected to decrease outside of the east New Orleans levee system, as predicted by STWAVE simulations, up to 2 feet near Caernarvon and 1 ft near the IHNC/MRGO funnel. Maximum wave heights are predicted to decrease by as much as 1.5 ft in the Bay St. Louis, MS area. The largest decrease in maximum wave heights (greater than 6 ft) is observed immediately landward of the barrier islands. Wave energy dissipates as a result of the restored islands and reduces the landward maximum wave heights when compared to the BI-1 and base configurations.

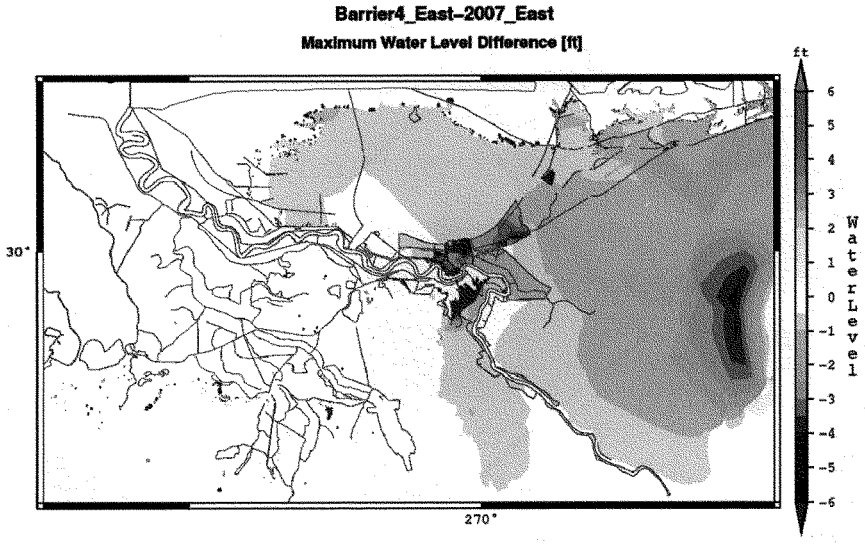

Figure A.8-3. Difference in maximum surge level (ft) between the BI-4 configuration and the base case for the Barrier Islands storm suite.

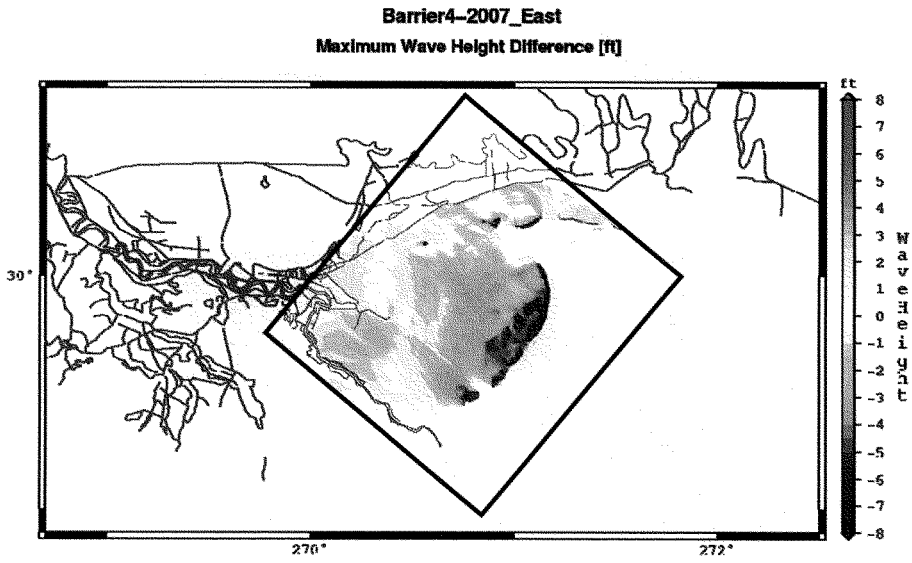

Figure A.8-4. Difference in maximum wave height (ft) between BI-4 and the base case for the Barrier Islands storm suite for the southeast STWAVE grid.

The model results indicate that the barrier islands provide some level of protection as a first line of defense. In general, raising the barrier islands caused a decrease in peak water level and wave energy landward of the barrier islands when compared to the peak water level and wave energy for the baseline 2007 Post-Katrina configuration. Degradation of the barrier islands caused a minimal increase in maximum surge level landward of Chandeleur Sound and increased waves at the hurricane protection system. With less obstruction in the Chandeleur Sound, the modeled wave heights increase, thus heightening the potential for overtopping of levees and inundation of protection areas.

A.8.2 Marsh Alternatives

The marsh alternatives included a predicted wetland definition 50 years into the future with no increased action (NIA) taken and a restored/improved marsh condition. The NIA condition was developed as part of the Coastal Louisiana Ecosystem Assessment and Restoration (CLEAR) Program. The forecasting model developed by CLEAR predicts physical processes, geomorphic features, water quality, and ecological succession. Geomorphic/bathymetric changes are based on the likelihood of discretized regions changing from open water to marsh or marsh to open water. The future condition of Coastal Louisiana predicted by CLEAR, referred to as the degraded condition, in fact does predict degradation in Southern Louisiana, but also predicts growth in the Atchafalaya basin and Breton Sound. The CLEAR future condition bathymetry was applied to the model grids and mesh and a series of storm simulations was made. Figure A.8-5 identifies the CLEAR/NIA landscape changes. Figure A.8-6 is the difference in the envelope of maximum water level between the CLEAR marsh configuration and the 2007 base case for areas to the west and the 2010 base case for the West Bank and areas east of the river. Comparison to two grids is required to isolate the impact of the wetland degradation. The degraded wetlands were incorporated into the

2010 base grid with the Morganza to the Gulf levee removed. For this reason, areas to the west were compared to 2007 which also did not incorporate the Morganza to the Gulf levee. White areas experienced less than 1 ft of increase or decrease. There is a widespread increase in surge up to 6 ft across most of the degraded areas shown in Figure A.8-6a. The surge is increased by 1-3 ft in the Lake Maurepas region, 1-6 ft in St. John/St. Charles Parishes, and 1-3 ft east of Morgan City. Less than 1 ft change in surge occurs east of the MS River and surges are increased at the West Bank 1-2 ft. Figure A.8-7 shows the maximum wave heights differences for the restored configuration. Seaward of the east New Orleans levee system, waves are expected to increase by as much as 2 ft. Maximum wave heights are also expected to increase by up to 2 ft in the Crown Point vicinity. Note that the wave height differences west of the MRGO in St. Bernard Parish are consistent with the differences observed for the 2010 vs 2007 configurations, i.e. the CLEAR/NIA bathymetry is based on the 2010 grid so the predicted differences are due to the increased 2010 levee heights and not a result of the degraded marsh conditions.

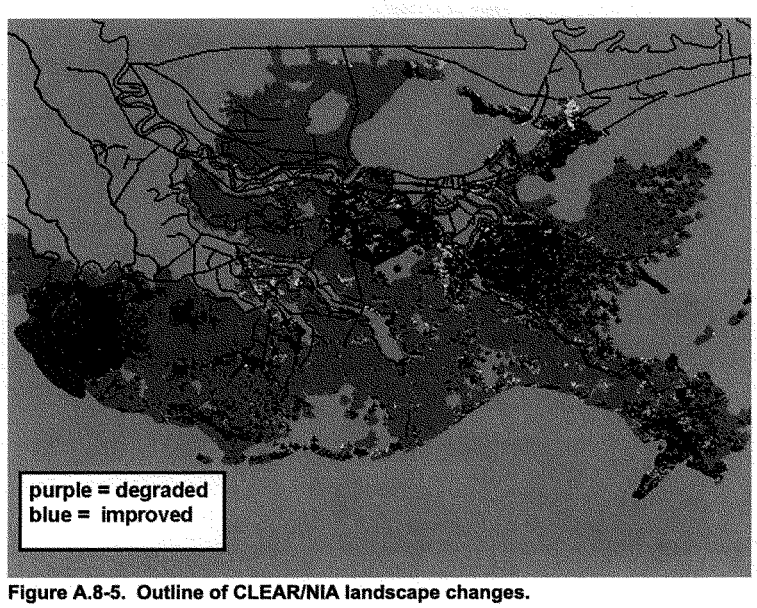

Figure A.8-5. Outline of CLEAR/NIA landscape changes.

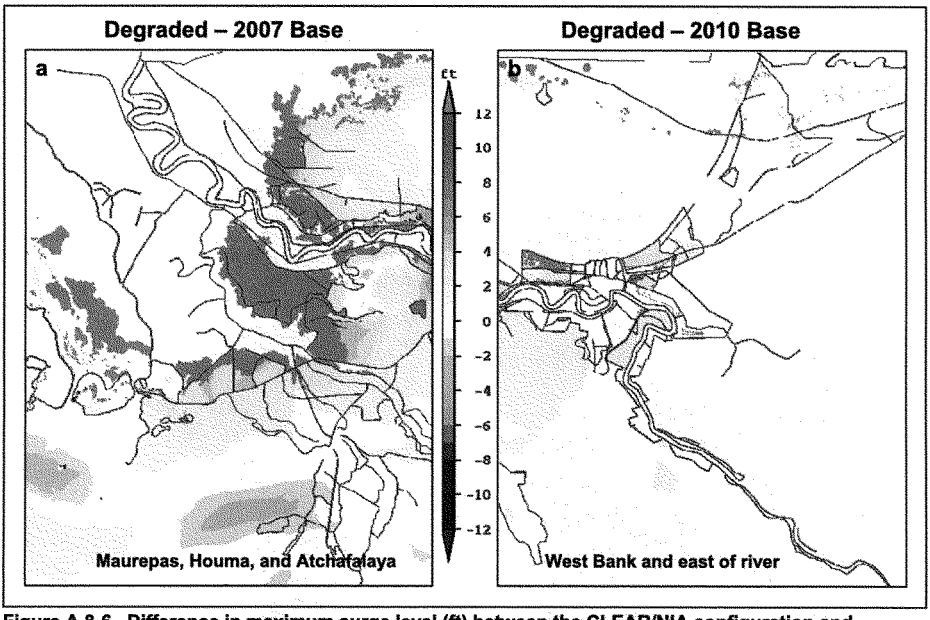

Figure A.8-6. Difference in maximum surge level (ft) between the CLEAR/NIA configuration and the 2007 and 2010 base cases for the NIA marsh storm suite.

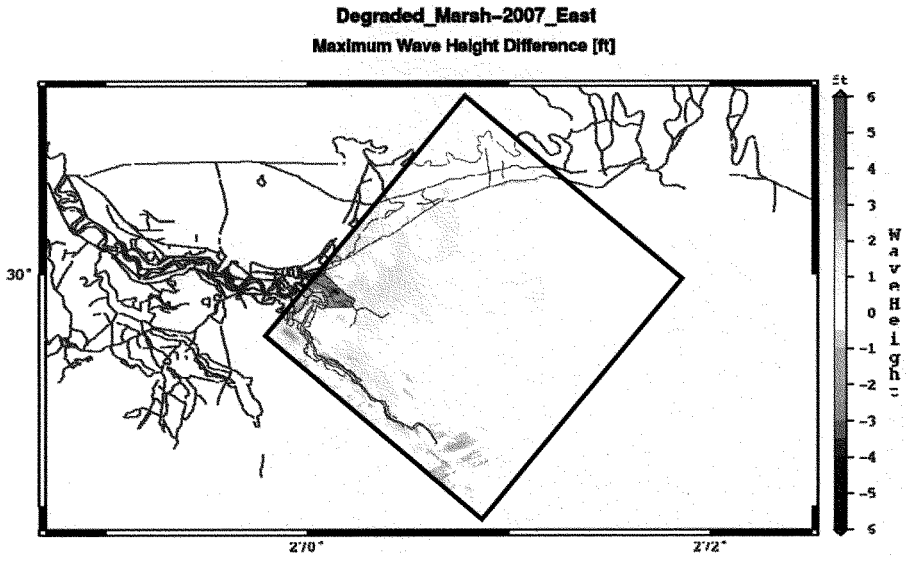

Figure A.8-7. Difference in maximum wave height (ft) between the CLEAR/NIA configuration and the base case for the NIA marsh storm suite for the southeast STWAVE grid.

The restored condition was developed by ERDC-CHL under the direction of the New Orleans District's improved action plan. The District provided CHL with marsh creation locations and type (i.e. freshwater, cypress swamp, saline, etc), freshwater diversion locations, and the volume of sediment diverted. CHL implemented these restoration features into a marsh creation program and modifications were made to the bathymetry, Manning's n values, and directional roughness lengths. These changes were applied to the model grids, mesh, and frictional files and a series of storm simulations was made. Figure A.8-8 shows an outline of the marsh restoration features for eastern Louisiana. Figure A.8-9 is the difference in the envelope of maximum water level between the restored marsh configuration and the 2007 base case and the 2010 base case. As with the degraded case, comparison to two grids is required to isolate the impact of the wetland restoration. There is a reduction in surge west of the MS River bounded by Golden Meadow on the east and mid-Terrebonne Parish on the west ranging from 1-3 ft with the largest reduction (greater than 2 ft) east of the Houma Navigation Canal and south of Route 24. This region of attenuated surge corresponds with an area that has many restoration features. There is also an area of 1-3 ft surge reduction on the West Bank east of the GIWW. Less than 1 ft change in maximum surge occurs east of the MS River. Figure A.8-10 shows the maximum wave height differences for the restored configuration. Seaward of the east New Orleans levee system, the waves are not expected to change. Note that the wave height differences west of the MRGO in St. Bernard Parish are generally consistent with the differences observed for the 2010 vs 2007 configurations, i.e. the restored bathymetry is based on the 2010 grid so the predicted differences are likely due to the increased 2010 levee heights and not a result of the restored marsh conditions.

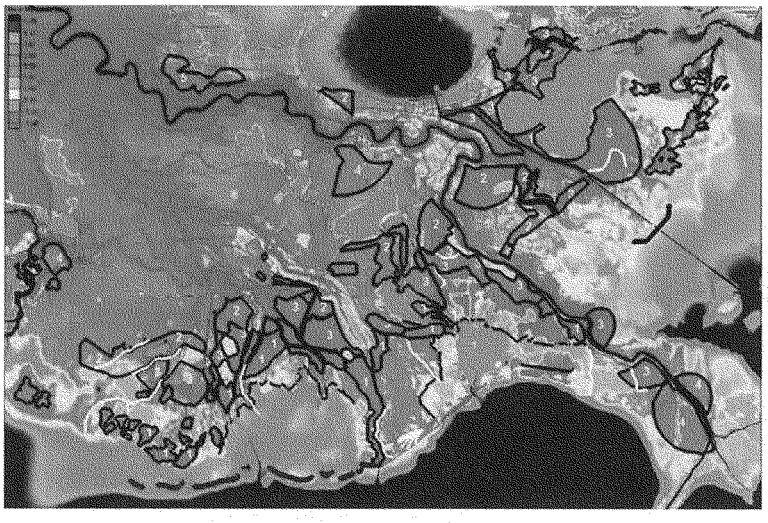

Figure A.8-8. Outline of marsh restoration features. Marsh types are outlined as follows: 1 = saline, 2 = intermediate, 3 = brackish, 4 = fresh, 5 = cypress, white lines = ridges, purple = shrub/scrub for barrier islands.

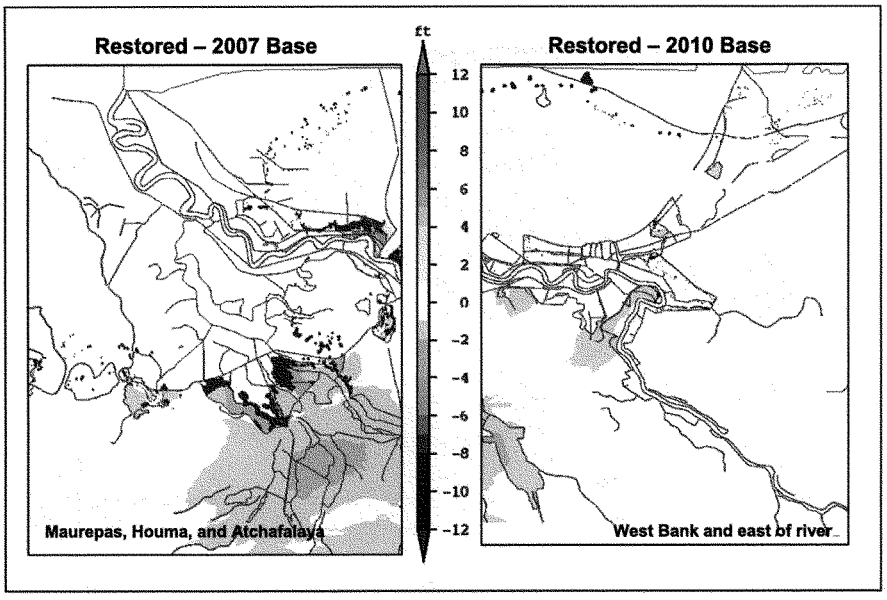

Figure A.8-9. Difference in maximum surge level (ft) between the restored marsh configuration and the 2007 and 2010 base cases for the restored marsh storm suite.

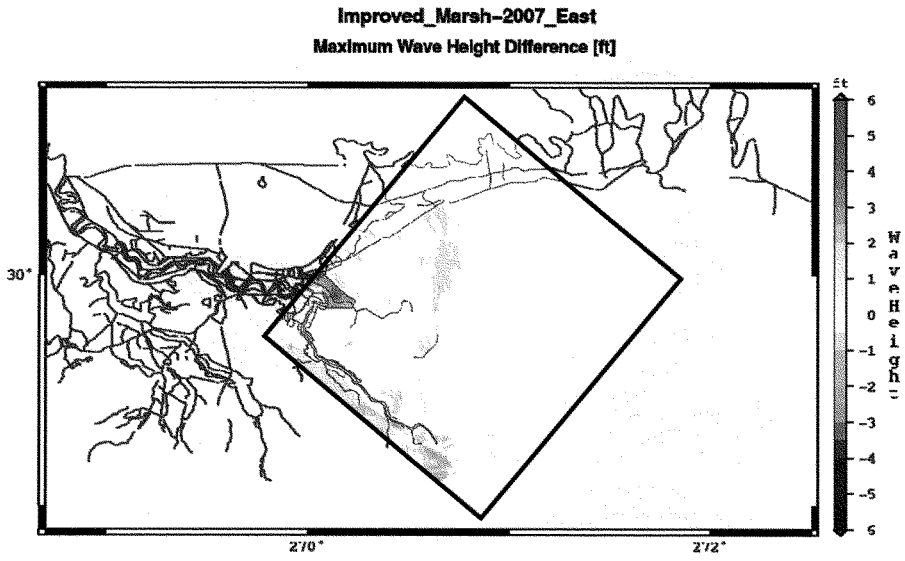

Figure A.8-10. Difference in maximum wave height (ft) between the restored marsh configuration and the base case for the restored marsh storm suite for the southeast STWAVE grid.

A.8.2.1 Marsh Alternatives for LAWEST

The improved marsh scenario for western Louisiana changed maximum surge levels and wave heights by less than 1 ft. See Figures A.8-11 and A.8-12.

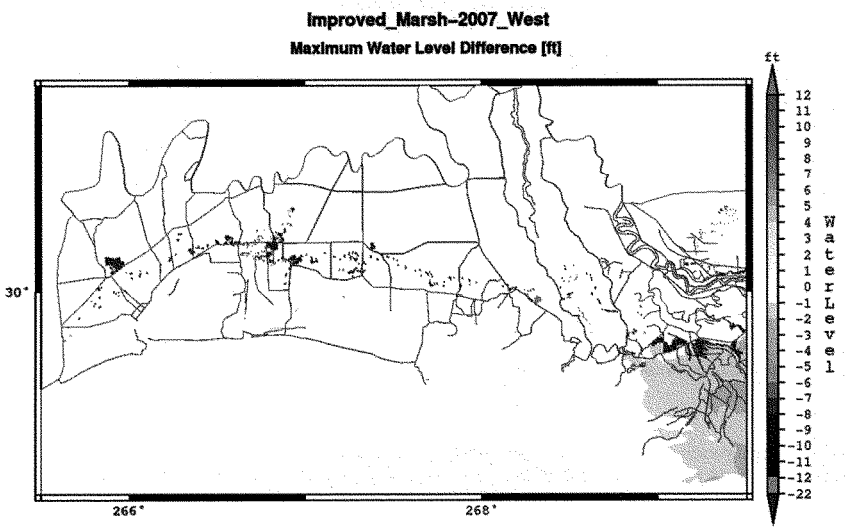

Figure A.8-11. Difference in maximum surge level (ft) between the restored marsh configuration and the LAWEST base cases for the LAWEST restored marsh storm suite.

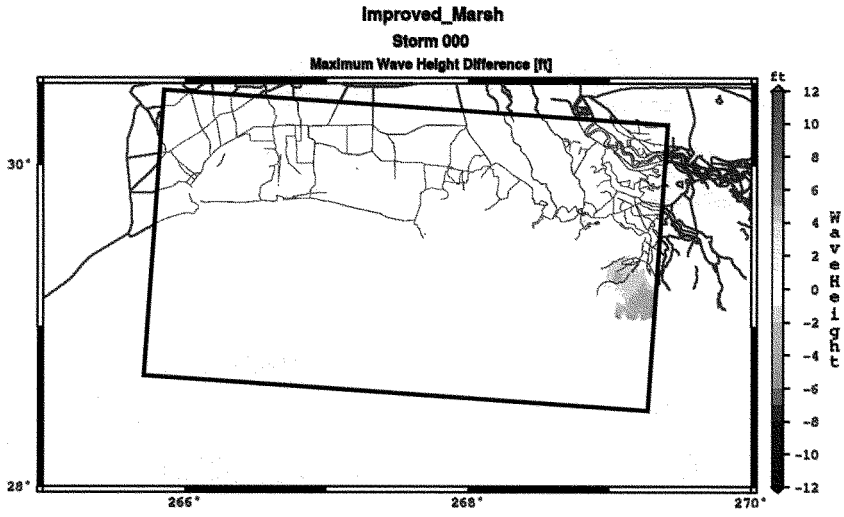

Figure A.8-12. Difference in maximum wave height (ft) between the restored marsh configuration and the LAWEST base case for the LAWEST restored marsh storm suite for the west STWAVE grid.

The impact of the future degraded landscape on peak surge levels and wave heights are given in Figures A.8-13 and A.8-14. The degraded landscape generally results in 1 to 2 ft increases in peak surges across western Louisiana with the exception of the Atchafalaya area. The future condition in this area is an improved landscape due to land building from the Atchafalaya River. Decreases in surges in this area are 1 to 3 ft. Maximum wave heights are also generally increased across west Louisiana. Some areas are greater experience greater than 1 ft change while others are less than a foot. In the Atchafalaya area, maximum wave height decreases are as much as 6 ft.

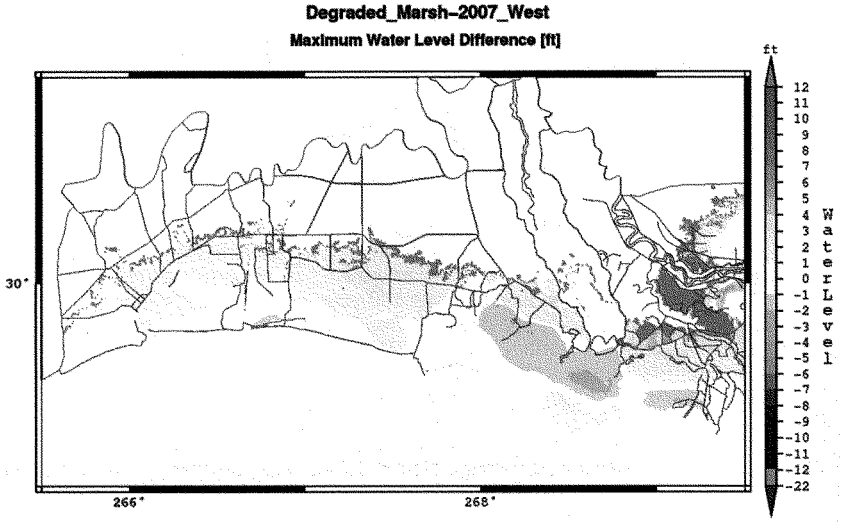

Figure A.8-13. Difference in maximum surge level (ft) between the CLEAR/NIA configuration and the LAWEST base case for the LAWEST NIA marsh storm suite.

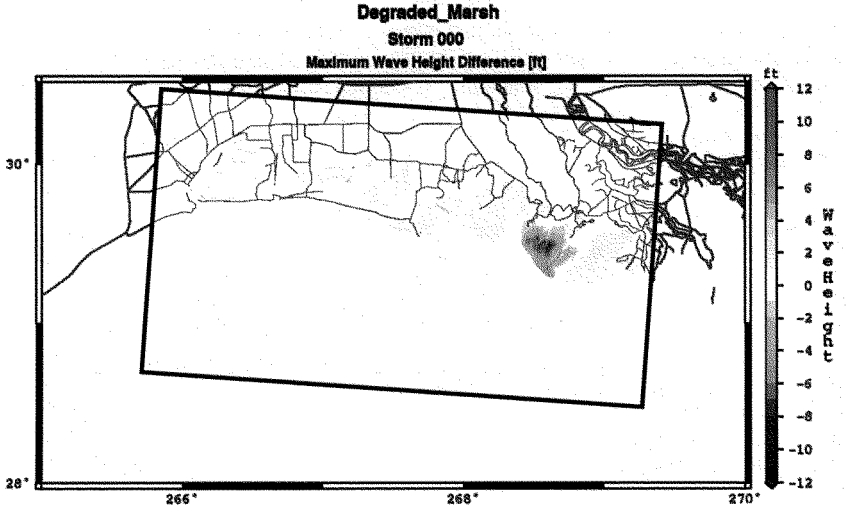

Figure A.8-14. Difference in maximum wave height (ft) between the CLEAR/NIA configuration and the LAWEST base case for the LAWEST NIA marsh storm suite for the west STWAVE grid.

A.8.3 Sea Level Rise

Sea level rise and subsidence are significant issues in the design of flood protection for southeast Louisiana. Flood walls, in particular, can not be easily raised, so future sea level rise must be considered in the initial design. The purpose of this analysis is to estimate the impact of sea level rise on surge and waves for the design of the flood defenses.

The sea level rise analysis consisted of 27 storm simulations. Nine storms were selected from the 2010 simulations and each was run with 1 ft, 2 ft, and 3 ft increase in water level. No other changes to input were made (same offshore waves, same land cover specification, same model parameters, etc.). The storms were chosen to target 100-year water levels in various areas. To summarize the results, eleven reaches are defined: South Shore of Lake Pontchartrain (SSP), East Orleans (EO), St. Bernard North (SBN), St. Bernard South (SBS), Caernarvon (C), Plaquemines East (PE), Plaquemines West (PW), South West Bank (SWB), North West Bank (NWB), Golden Meadow (GM), and Morganza to the Gulf (MtG). These areas are illustrated in Figure A.8-9.

The selection of only nine storms that give approximate 100-yr water levels provides estimates of the impact of sea level rise, but is not a rigorous analysis. For example, land cover classifications were not changed in the analysis. Vegetation types would change as water level increases, but if the increase is slow enough and sediment is available, the marsh elevation may also adjust to the change in water level. Manning-n values were not adjusted in this analysis because of the uncertainty in the values for higher sea level and so the results at each water level could be directly compared. Sea level was increased over the entire domain, which means that local impacts of subsidence are probably over estimated. The impacts of increasing sea level are two fold, the surge wave (which propagates at a speed, $c = \sqrt{gd}$, were g is acceleration of gravity and d is water depth) propagates faster, and the depth-limited wave height increases (also increasing wave setup). In general, it is expected that sea level rise increases water levels more than linearly (water level increase > sea level rise), but the complex, shallow geometry and bathymetry of Southeast Louisiana alters this trend depending on the relative speed of the storm and the surge propagation (and the relative phasing of the two).

The surge increases are calculated as the difference between the maximum water level at each grid point for the sea level rise run and the maximum water level for the base 2007 run. Similarly, the wave increases are calculated as the difference between the maximum wave height at each grid point for the sea level rise run and the maximum wave height for the base 2007 run. Surge increases were normalized by the sea level rise and are presented as multipliers below.

South Shore of Lake Pontchartrain. The SSP reach has the most consistent response to sea level rise. The multiplier is 1.0 to 1.5 (1 would be a linear response, 1 ft sea level rise = 1 ft increase is water level) with an average value of 1.3 for the target storms. The increased depth decreases the friction, allowing more water to pile up on the shore. The SSP reach has fairly consistent increase in wave height for sea level rise: 0.6 ft for 1 ft sea level rise, 1.0 ft for 2 ft sea level rise, and 1.5 ft for 3 ft sea level rise. The ratio of wave height increase to water level increase for the target storms varies from 0.23 to 0.60, with an average value of 0.43. The values are relatively high because an increase in surge results in a direct increase in depth-limited wave height in most areas.

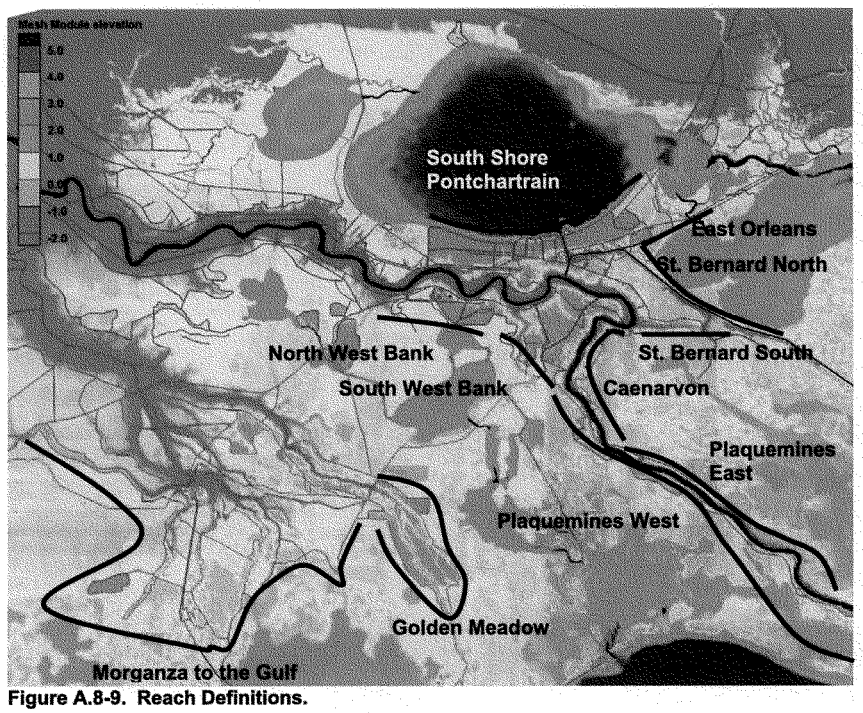

Figure A.8-9. Reach Definitions.

Back Levees of East Orleans and St. Bernard North. The response in EO and SBN has slightly more variation than SSP, with a multiplier of 1.1 to 1.6. This area forms a small pocket in the funnel area, but the reach is not as complex or shallow as areas to the south and west. The multipliers for the storms near the 100-yr water level are 1.1 to 1.6 in EO and 1.2 to 1.6 in SBN, with average values of 1.2 and 1.3, respectively. The EO and SBN behave relatively consistently with increases in wave height of 0.1 to 1.2 ft for EO and 0.1 to 1.0 ft for SBN. The ratios of wave height increase to water level increase are all less than 0.4, with average values for the target storms of 0.13 (range of 0.06 to 0.31) for EO and 0.17 (range of 0.04 to 0.38) for SBN.

St. Bernard South and Caernarvon. This reach is complex and shallow, and the results are highly variable with multipliers of 0.7 to 4.5. The large responses correspond to the storms with some of the smallest maximum surges, which have tracks that cross through Breton Sound, east of this area. As the storms pass, the larger water depth allows the surge to move in faster, as well as decreasing the frictional resistance. The "catchers mitt" of Caernarvon amplifies the surge for these storms. Storms that produce the largest surge in these areas (20-25 ft) have a sea level rise multiplier of 0.6 to 1.3 for St. Bernard South and 0.6 to 2.0 for Caernarvon. Storms that produce the 100-yr water levels have multipliers of 0.7 to 2.3 for SBS and 0.7 to 4.5 for C with average values of 1.4 and 2.1, respectively. The wave height results are highly variable with increases of 0.1 to 2.1 ft for SBS and 0.5 to 3.0 ft for C. The large responses correspond to the storms with the smallest maximum surges with tracks that cross through Breton Sound, east of this area. As the storms pass, the larger water depth allows large waves to propagate into

the area, as well as decreases the frictional resistance. The average ratio of wave height increase to water level increase is relatively large in this area, 0.45 (range of 0.4 to 0.5) for SBS and 0.50 (range of 0.42 to 0.63) for C.

Plaquemines East and West. These reaches are large with a lot of spatial variability, but the multipliers are less variable than the adjoining reaches. The multipliers for the target storms are 1.3 to 2.0 for Plaquemines East. For the Plaquemines West reach, the range of multipliers for the target storms is 1.4 to 3, with average values of 1.5 and 1.9, respectively. The wave height increases in these areas are similar to St. Bernard South and Caernarvon. The wave height increases are 0.4 to 2.8 ft for PE and 0.4 to 2.9 ft for PW. The maximum increases in wave height in the Plaquemines East reach were typically at the north end of this reach, between Phoenix and Davant. The average ratio of wave height increase to water level increase is 0.58 (range 0.38 to 0.78) for the target storms for PE. For the Plaquemines West reach, the maximum increases in wave height were typically between Empire and Buras or near Myrtle Grove. The average ratio of wave height increase to water level increase is 0.41 (range 0.23 to 0.69) for the target storms for PE.

West Bank. This reach is also complex and shallow. The multipliers range from 1.0 to 3.6. Storms near the 100-yr level for the West Bank have multipliers ranging from 1.3 to 3.6 for SWB and 1.0 to 2.9 for NWB. The largest numbers tend to be hot spots (small areas) and not large areas of high multipliers. The average multipliers for the target storms are 2.5 for SWB and 2.1 for NWB. The wave height increases are 0.1 to 1.0 ft. The ratio of wave height increase to water level increase is 0.03 to 0.3 for the target storms with average values of 0.11 for SWB and 0.15 for NWB.

Golden Meadow and Morganza to the Gulf. Multipliers in this reach are similar to the West Bank, but not as variable. Multipliers range from 1.0 to 2.5. The surges tend to be most amplified on the northeast corner of Golden Meadow and in the pocket regions. The multipliers for the storms near the 100-yr water level are 1.4 to 2.3 for Golden Meadow and 1.5 to 2.0 for Morganza to the Gulf, with average values of 1.8 and 1.7, respectively. These reaches include complex levee geometries (pockets) and bathymetry, but are more exposed than the west bank. The wave height increases are up to 2.0 ft along Golden Meadow and up to 3.0 ft along Morganza to the Gulf. The average ratio of wave height increase over surge increase for the target storms is 0.27 (range 0.14 to 0.42) for Golden Meadow and 0.37 (range 0.23 to 0.5) for Morganza to the Gulf.

Annex B

Fact sheets for sub basins

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Planning Unit 1

Laplace

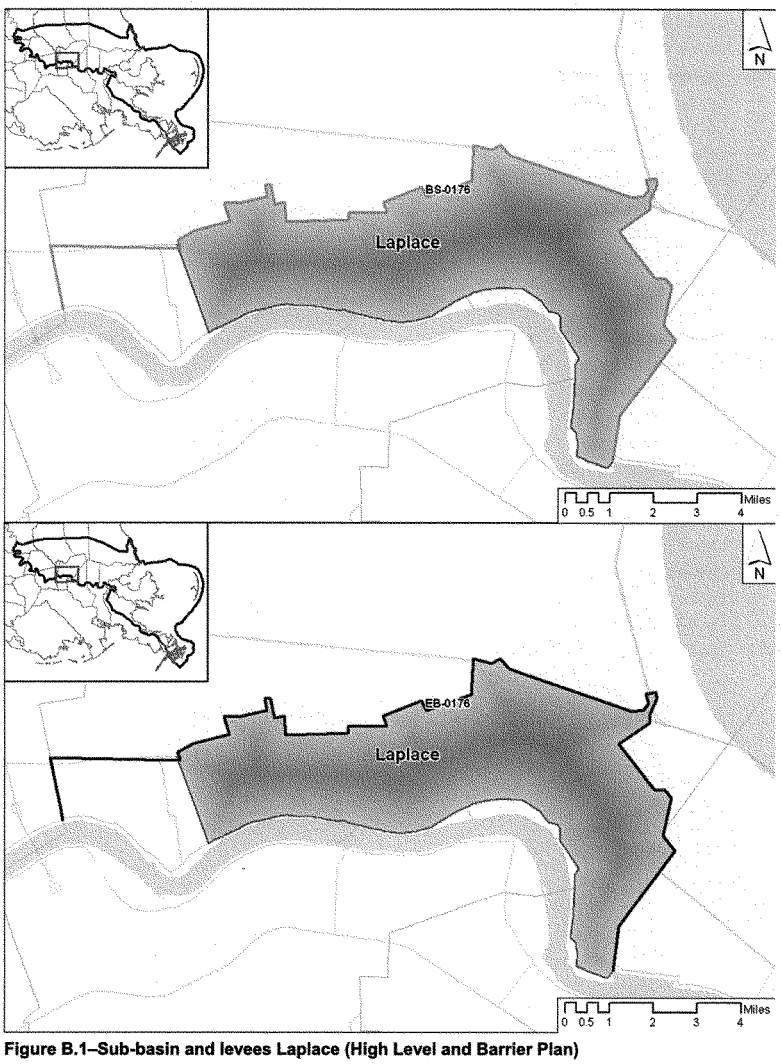


Figure B.1–Sub-basin and levees Laplace (High Level and Barrier Plan)

Table B-1 – Levee characteristics Laplace

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0176	27.3	n.a.	13.5	18.5	21	12.5	17	19
-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0176	27.3	n.a.	9	14.5	18	8	13	16
-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	10,000	Based on 0.5cfs/acre

Storage Area	Type	Notes
Laplace	Single area	Max stage is higher of surge (176) or design height (176)

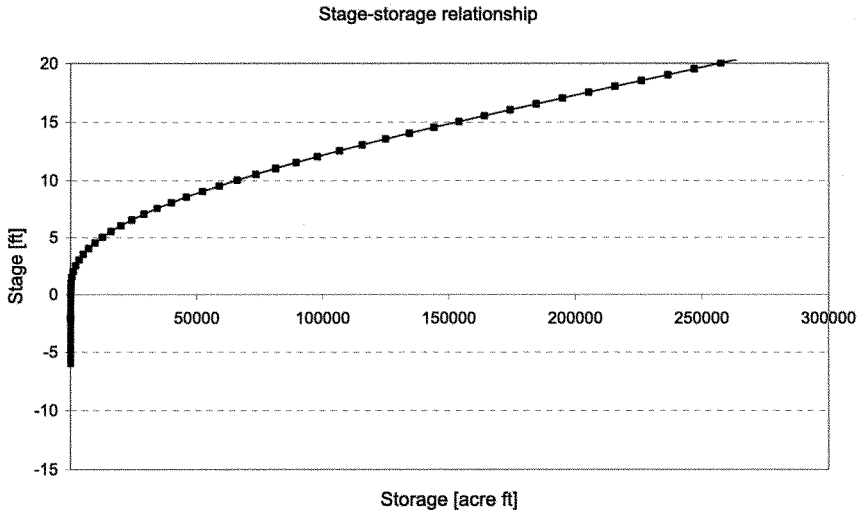


Figure B.2 – Stage storage relationship Laplace

Additional Notes

Pumping needs to be included based on the basis of 0.5cfs/acreas as development is close behind the proposed levee line. Area taken as 21000 acres and therefore assumed 10,000cfs.

A new alignment for the levee in Laplace has been suggested from a different study. At present - for consistency - the original alignment is being maintained, but reference will be made in the final documentation to the new alignments. Pump sizing has also been undertaken for both the existing and new alignments by the other project. For the existing alignment a value of 3500cfs has been developed against the 10,000cfs suggested in above. For consistency the 10,000cfs value has been used.

Initial economic runs suggested that significantly more people were affected with a levee than without. This appears to be a result of the planning sub units finishing short of the actual flood extents, therefore suggesting in the without case that some property is outside the flood extent whereas they are below the extreme surge elevations (for the higher events). To solve this exterior stage frequency values were taken from the existing planning sub units and applied to the interior units in this area. The relationships were:

Laplace 1 - exterior values from SJJO_8c
Laplace 2 - exterior values from STCH_11b

St. Charles Norco

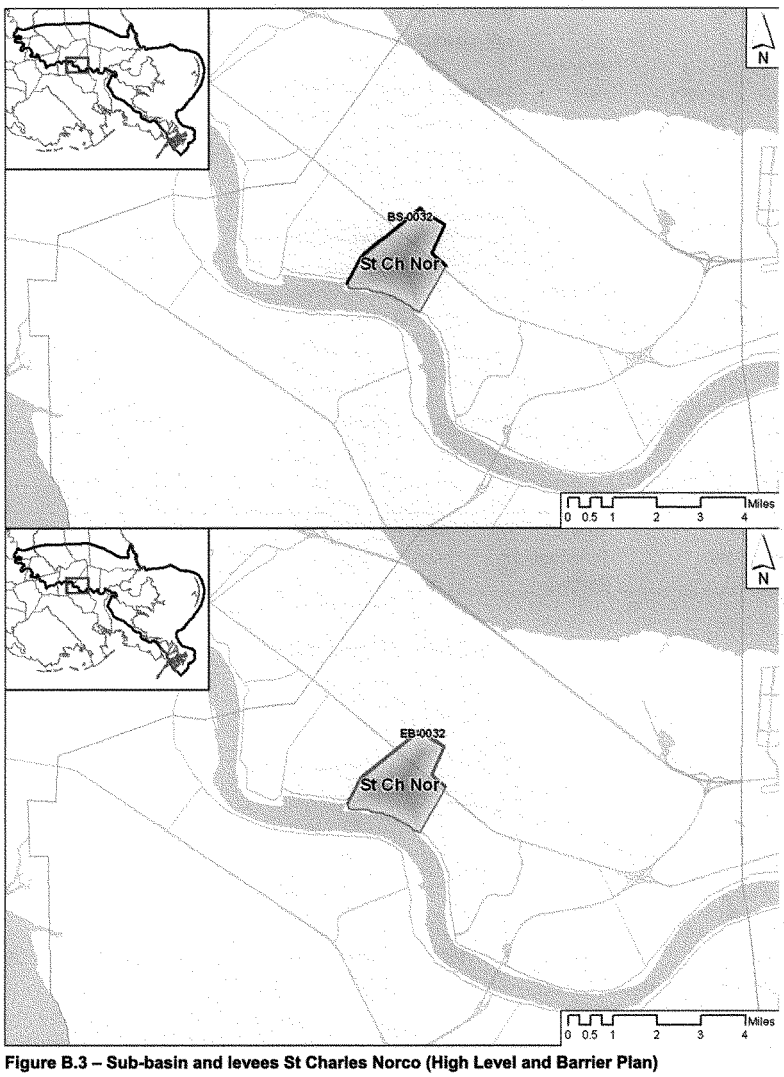


Figure B.3 – Sub-basin and levees St Charles Norco (High Level and Barrier Plan)

Table B-2 – Levee characteristics St Charles Norco

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0032	4.3	13	16	21	23.5	14.5	19.5	21.5
-	-	-	-	-	-	-	-	-

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0032	4.3	13	16[(1)]	18.5	21.5	16[(1)]	17	19.5
-	-	-	-	-	-	-	-	-

Note [(1)] Height fixed equal to High Level Plan 100 year as this should have been implemented.

	Existing	Proposed	Notes
Pump Capacity (cfs)	800	800	

Storage Area	Type	Notes
St Charles - Norco	Linked area (St Charles)	Connects to St Charles - rest (4.5'). See description of linked system in chapter 4.

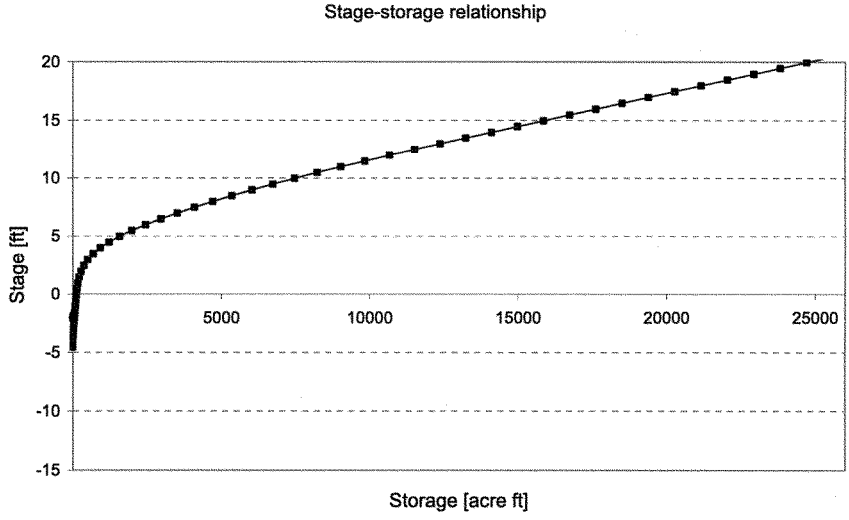


Figure B.4 – Stage storage relationship St Charles Norco

St Charles Remainder

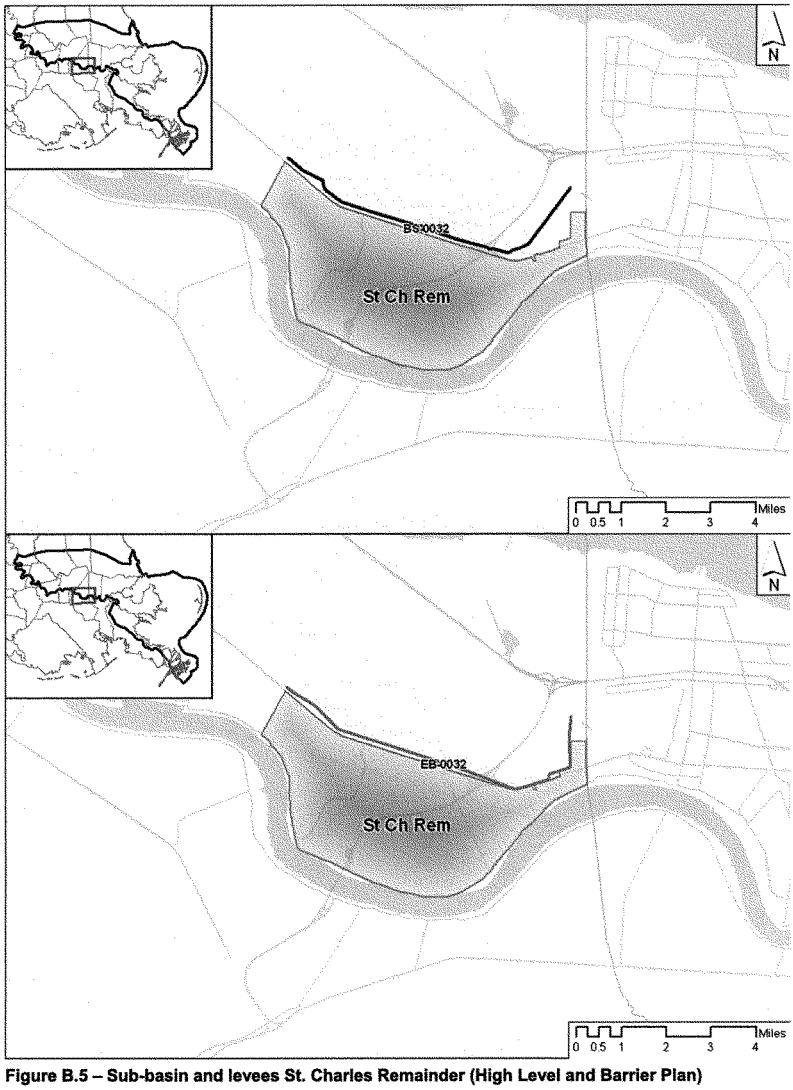


Figure B.5 – Sub-basin and levees St. Charles Remainder (High Level and Barrier Plan)

Table B-3 – Levee characteristics St. Charles Remainder

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0032	4.2	13	16	21	23.5	14.5	19.5	21.5
-	-	-	-	-	-	-	-	-

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0032	4.2	13	16[(1)]	18.5	21.5	16[(1)]	17	19.5
-	-	-	-	-	-	-	-	-

Note [(1)] Height fixed equal to High Level Plan 100 year as this should have been implemented.

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	0	

Storage Area	Type	Notes
St Charles - Rest	Linked area (St Charles)	Connects to St Charles - Norco (4.5'). See description of linked system in chapter 4.

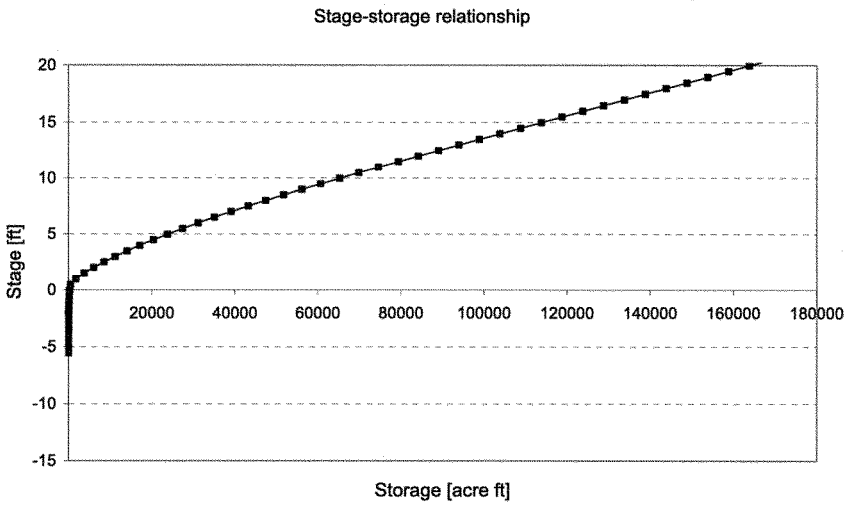


Figure B.6 – Stage storage relationship St Charles Remainder

East Jefferson

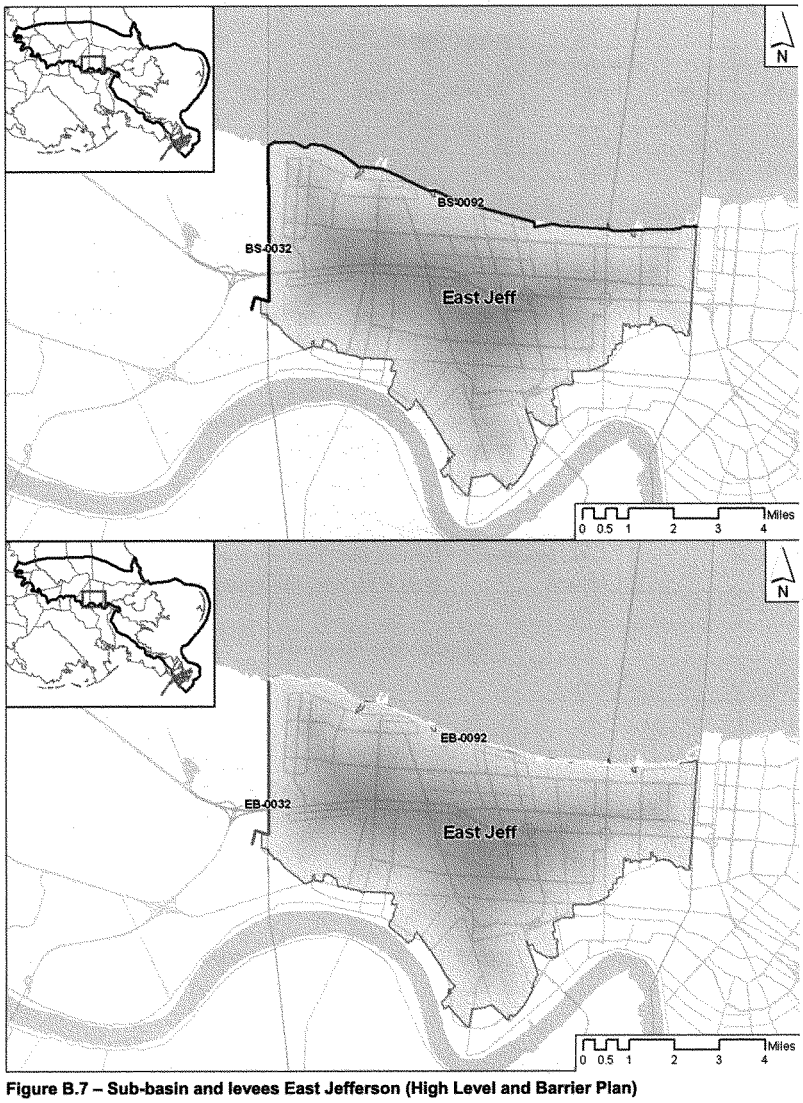


Figure B.7 – Sub-basin and levees East Jefferson (High Level and Barrier Plan)

Table B-4 – Levee characteristics East Jefferson

High Level Plan			**Design Height (waves modeled without friction)**			**Design Height (waves modeled with friction)**		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**	**100 year**	**400 year**	**1000 year**
[-]	**[miles]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
BS-0032	4.0	13	16	21	23.5	14.5	19.5	21.5
BS-0092	10.0	16.5	16.5	19.5	21.5	16.5	19.5	21.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**	**100 year**	**400 year**	**1000 year**
[-]	**[miles]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
EB-0032	4.0	13	16[(1)]	18.5	21.5	16[(1)]	17	19.5
EB-0092	10.0	16.5	16.5	16.5	18.5	16.5	16.5	18.5

Note [(1)] Height fixed equal to High Level Plan 100 year as this should have been implemented.

	Existing	**Proposed**	**Notes**
Pump Capacity (cfs)	20,590	20,590	

Storage Area	**Type**	**Notes**
East Jefferson	Linked area (New Orleans)	Connects to NO Metro with a weir at 5'. If stage < 5' then likely only floods to this level. See description of linked system in chapter 4

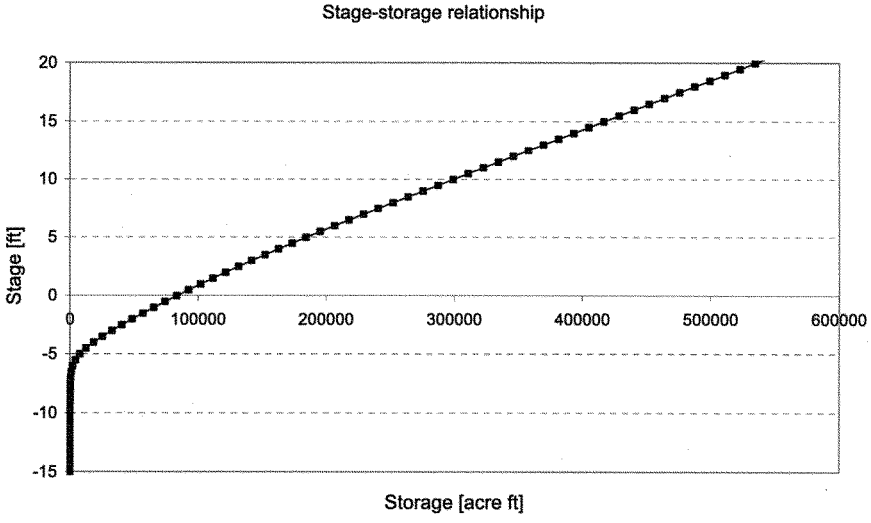


Figure B.8 – Stage storage relationship East Jefferson

New Orleans Metro

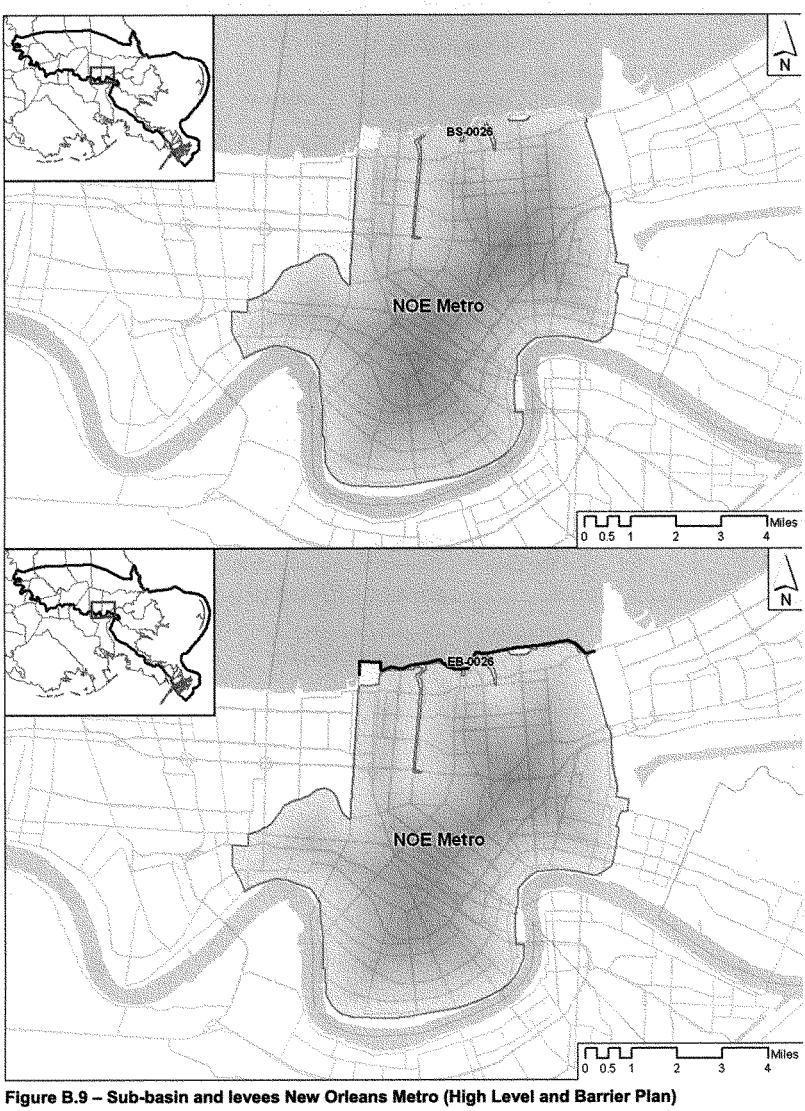


Figure B.9 – Sub-basin and levees New Orleans Metro (High Level and Barrier Plan)

Table B-5 – Levee characteristics New Orleans Metro

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0026	6.1	18.5	18.5	20	23.5	18.5	20	23.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0026	6.1	18.5	18.5	18.5	18.5	18.5	18.5	18.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	26,160	26,160	

Storage Area	Type	Notes
NO Metro	Linked area (New Orleans)	Connects to East Jefferson (5'), St Bernard (12.5') and NOE (12.5'). See description of linked system in chapter 4

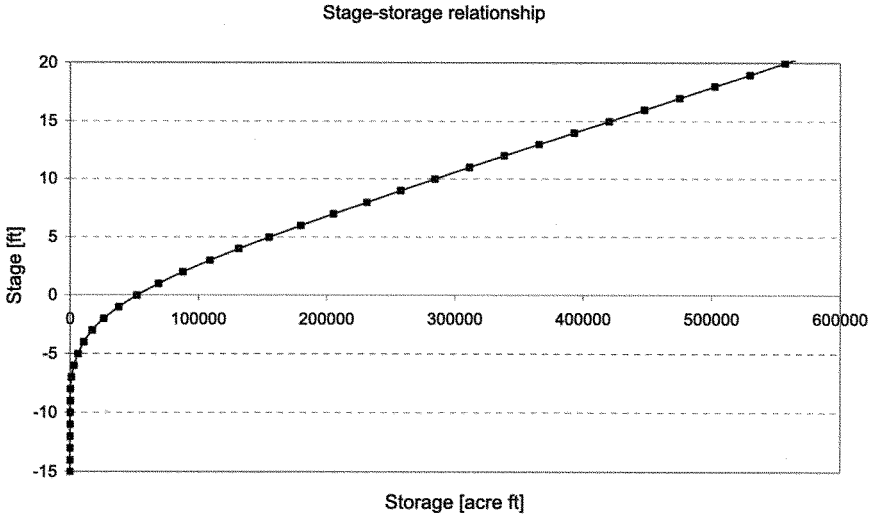


Figure B.10 – Stage storage relationship New Orleans Metro

New Orleans East

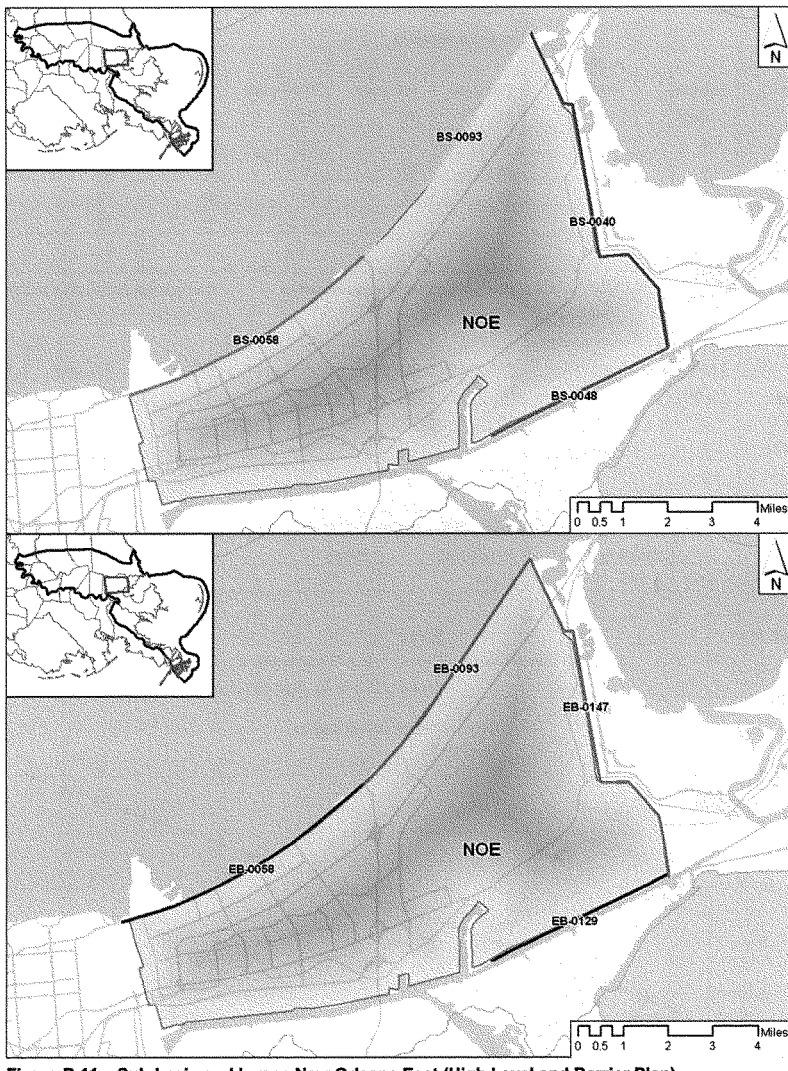


Figure B.11 – Sub-basin and levees New Orleans East (High Level and Barrier Plan)

Table B-6 – Levee characteristics New Orleans East

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0040	8.4	15	18	23.5	26	17.5	23	26
BS-0048	4.4	18	28.5	35	38	24	30	32
BS-0058	6.1	15	15	18	21	15	18	21
BS-0093	6.2	18	18	19	21.5	18	19	21.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0058	6.3	15	15	15	15	15	15	15
EB-0093	6.2	18	18	18	18	18	18	18
EB-0129	4.4	18	28.5[(1)]	33.5	37	26.5	33.5	37
EB-0147	8.4	15	18[(1)]	16.5	20.5	17.5[(1)]	16.5	20

Note [(1)] Height fixed equal to High Level Plan 100 year as this should have been implemented.

	Existing	Proposed	Notes
Pump Capacity (cfs)	6,310	6,310	

Storage Area	Type	Notes
NOE	Linked area (New Orleans)	Connects to NO Metro with a weir at 12.5'. If stage <12.5' then likely floods to this level. See description of linked system in chapter 4

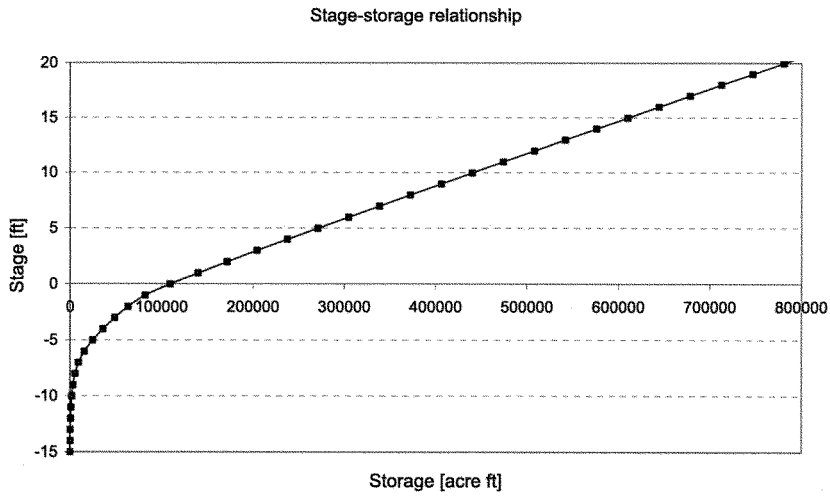


Figure B.12 – Stage storage relationship New Orleans East

St Bernard Wetland

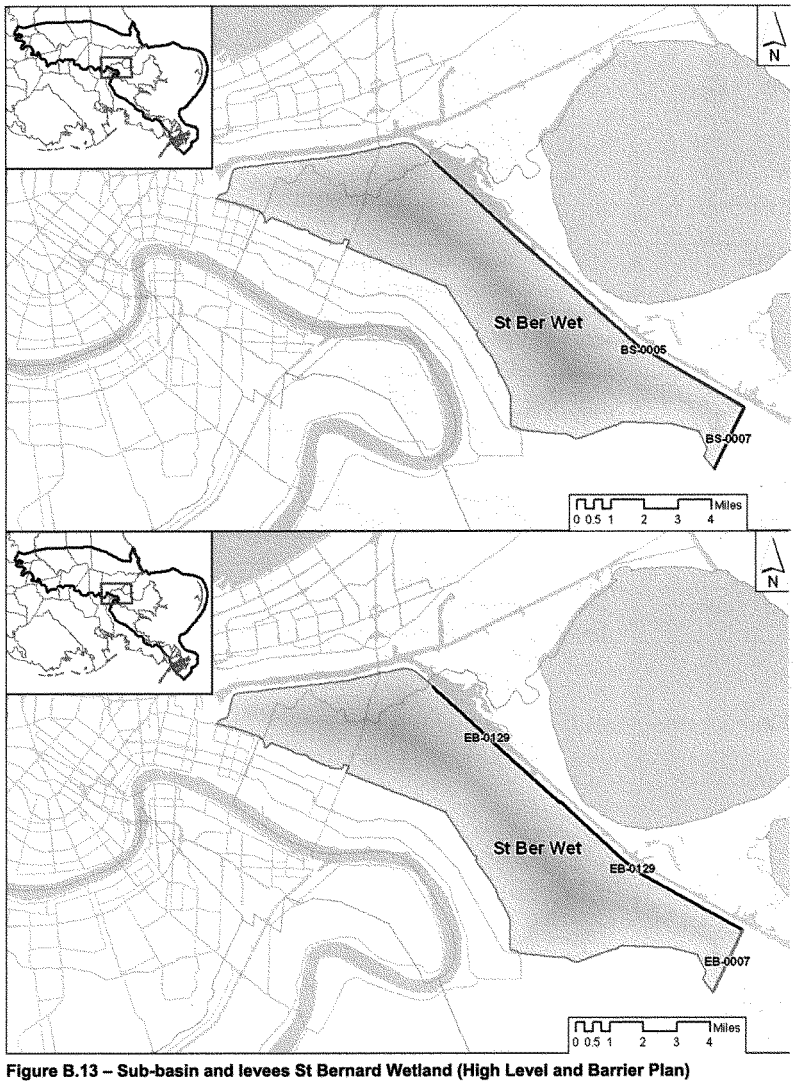


Figure B.13 – Sub-basin and levees St Bernard Wetland (High Level and Barrier Plan)

Table B-7 – Levee characteristics St Bernard Wetland

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0005	11.8	18	26.5	32.5	35.5	24	29.5	32
BS-0007	2.0	17	26	34	38	21.5	28	31.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0007	2.0	17	28	37.5	40.5	23	30.5	35
EB-0129	11.8	18	26.5	33.5	37	26.5	33.5	37

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	0	

Storage Area	Type	Notes
St Bernard Wetland	Linked area (New Orleans)	Connects to St Bernard Developed at 10.5'. If stage <10.5' then likely only floods to this level. See description of linked system in chapter 4

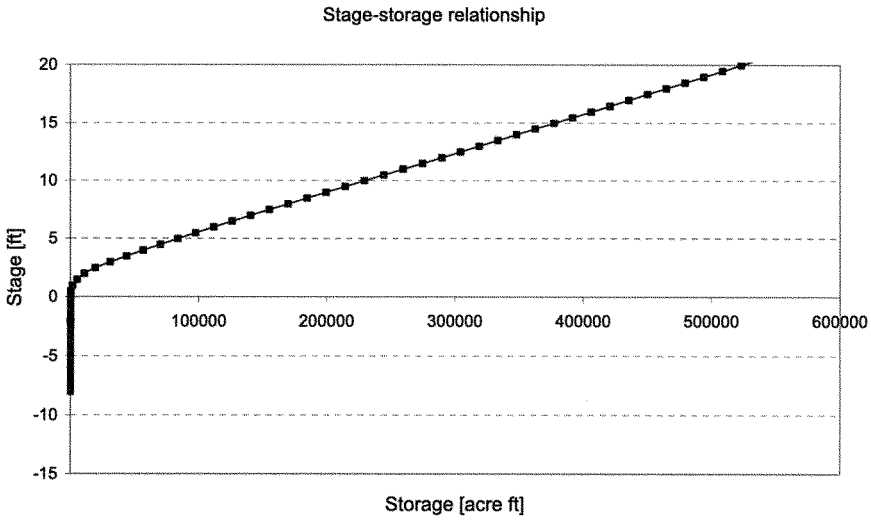


Figure B.14 – Stage storage relationship St Bernard Wetland

St Bernard Developed

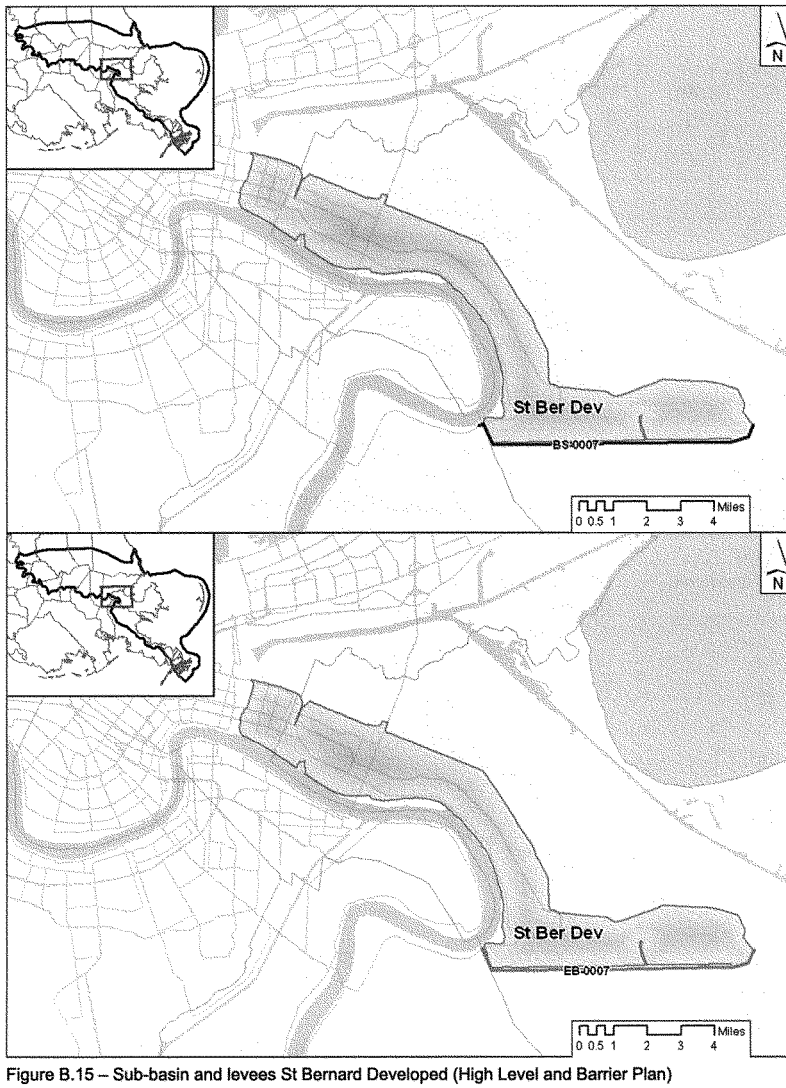


Figure B.15 – Sub-basin and levees St Bernard Developed (High Level and Barrier Plan)

Table B-8 – Levee characteristics St Bernard Developed

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0007	8.8	17	26	34	38	21.5	28	31.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0007	8.8	17	28	37.5	40.5	23	30.5	35

	Existing	Proposed	Notes
Pump Capacity (cfs)	8,190	8,190	Pumps assumed to pump out of system rather than into St Bernard Wetland

Storage Area	Type	Notes
St Bernard Developed	Linked area (New Orleans)	Connects to St Bernard Wetland (10.5') and NO Metro (12.5'). See description of linked system in chapter 4.

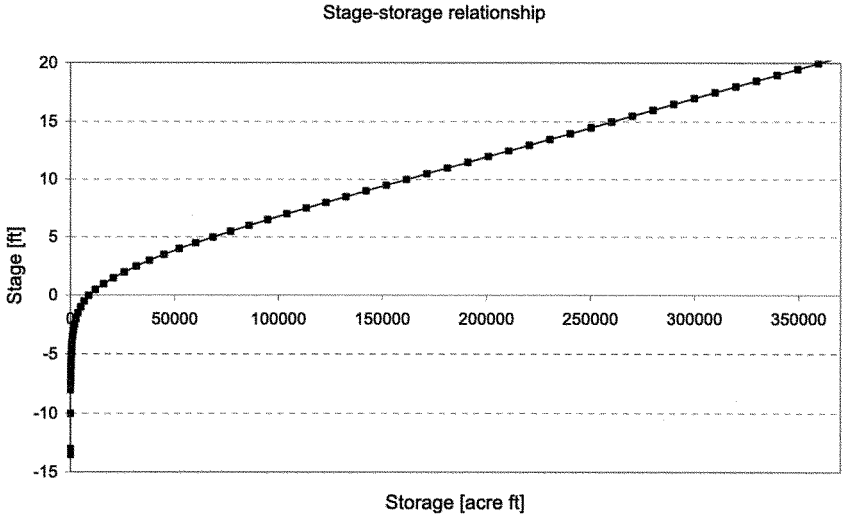


Figure B.16 – Stage storage relationship St Bernard Developed

Plaquemines - Scarsdale

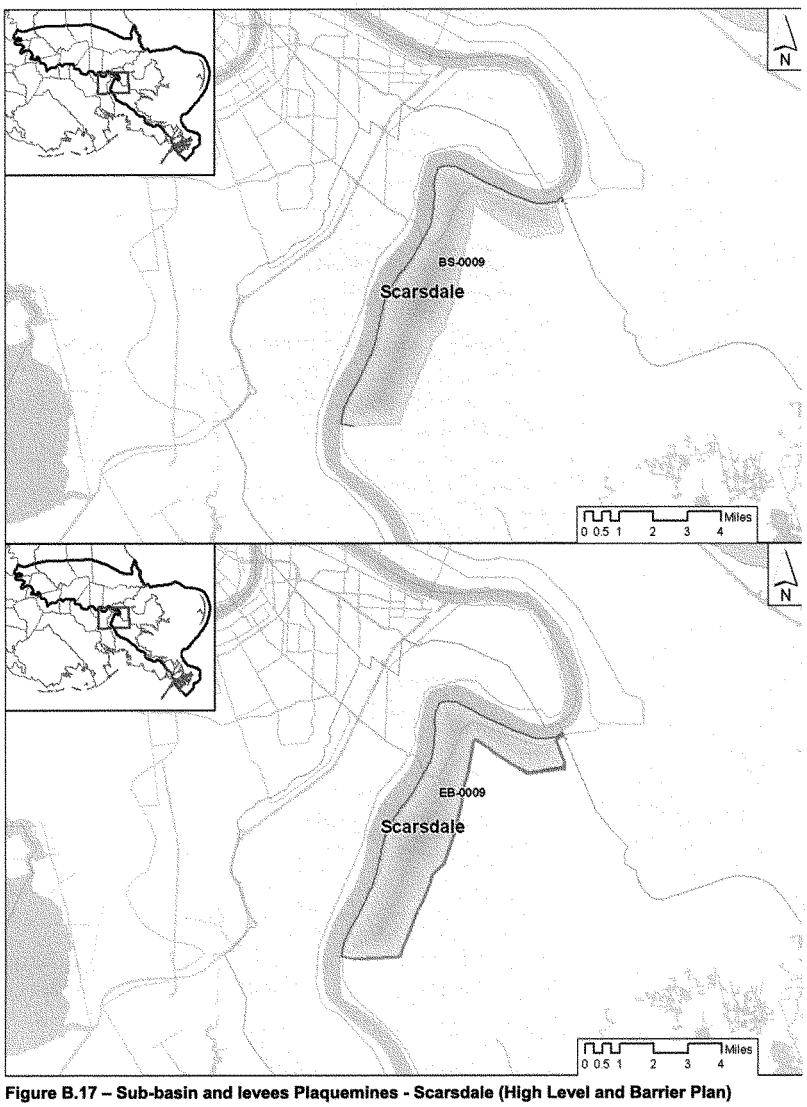


Figure B.17 – Sub-basin and levees Plaquemines - Scarsdale (High Level and Barrier Plan)

Table B-9 – Levee characteristics Plaquemines - Scarsdale

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0009	12.0	n.a.	29	38.5	42.5	24	31	35

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0009	12.3	n.a.	33	42.5	45.5	25.5	34.5	40

	Existing	Proposed	Notes
Pump Capacity (cfs)	1,780	1,780	

Storage Area	Type	Notes
Plaquemines Scarsdale	Single area	Max height is higher of surge (9) or design height (9) - upper level is also capped at 18' as the outflow level into the Mississippi River

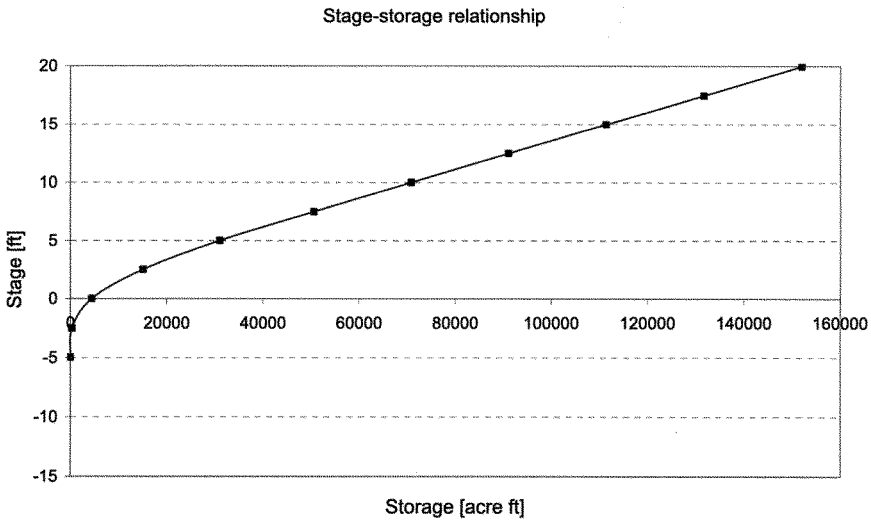


Figure B.18 – Stage storage relationship Plaquemines - Scarsdale

Additional Notes

Limit of flood depth will be based on the MRT levees rather than the hurricane protection - taken as 18' as an upper limit.

Madisonville

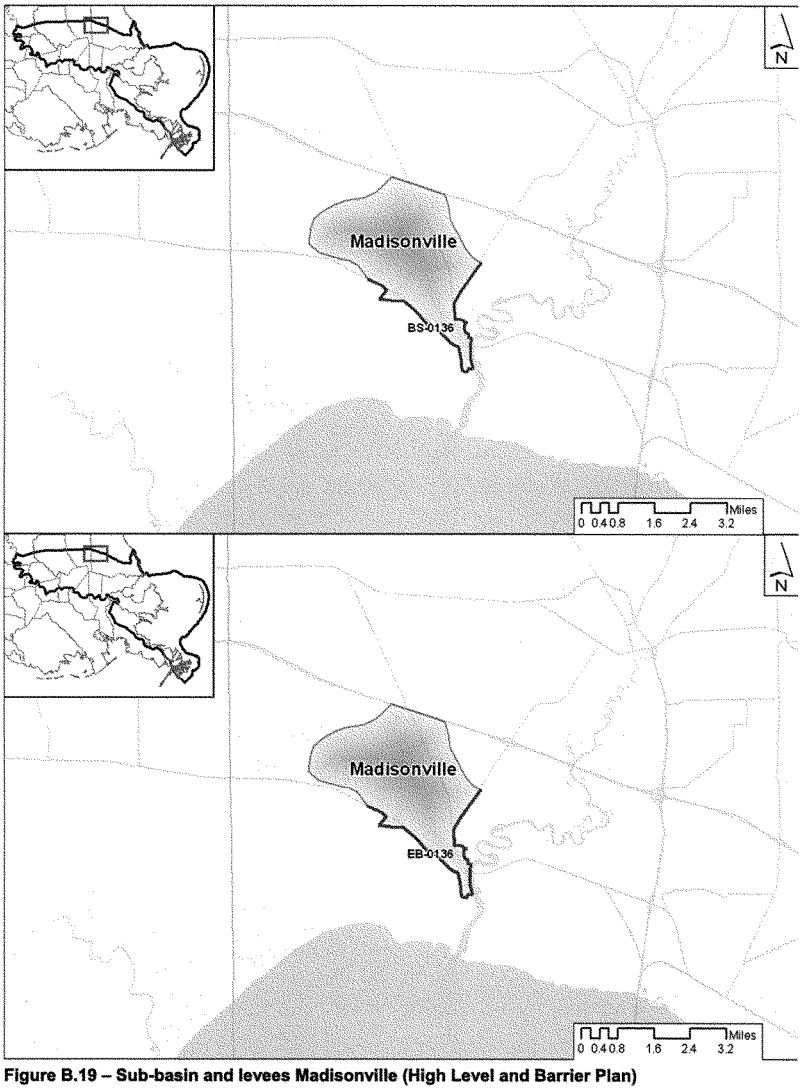


Figure B.19 – Sub-basin and levees Madisonville (High Level and Barrier Plan)

Table B-10 – Levee characteristics Madisonville

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0136	6.6	n.a.	16	20	22	16	20	22

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0136	6.6	n.a.	11	15.5	17.5	11	15.5	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	5,000	Based on 1cfs/acre

Storage Area	Type	Notes
Madisonville	Single area	Max stage is higher of surge (136) or design height (136).

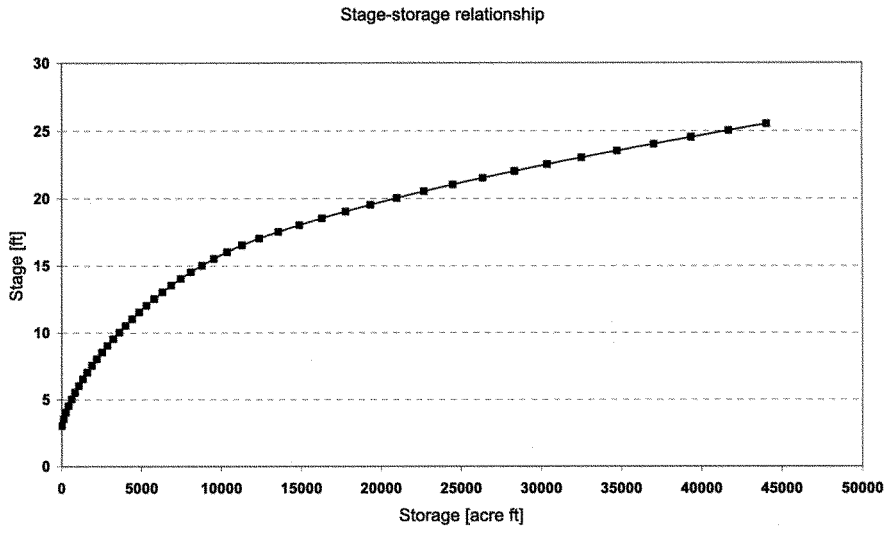


Figure B.20 – Stage storage relationship Madisonville

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

	High Level Plan				**Barrier Plan**			
	BS-0136 (miles)				**EB-0136 (miles)**			
Overtopping	4.1				4.1			
Non-overtopping	2.5				2.5			

South Covington

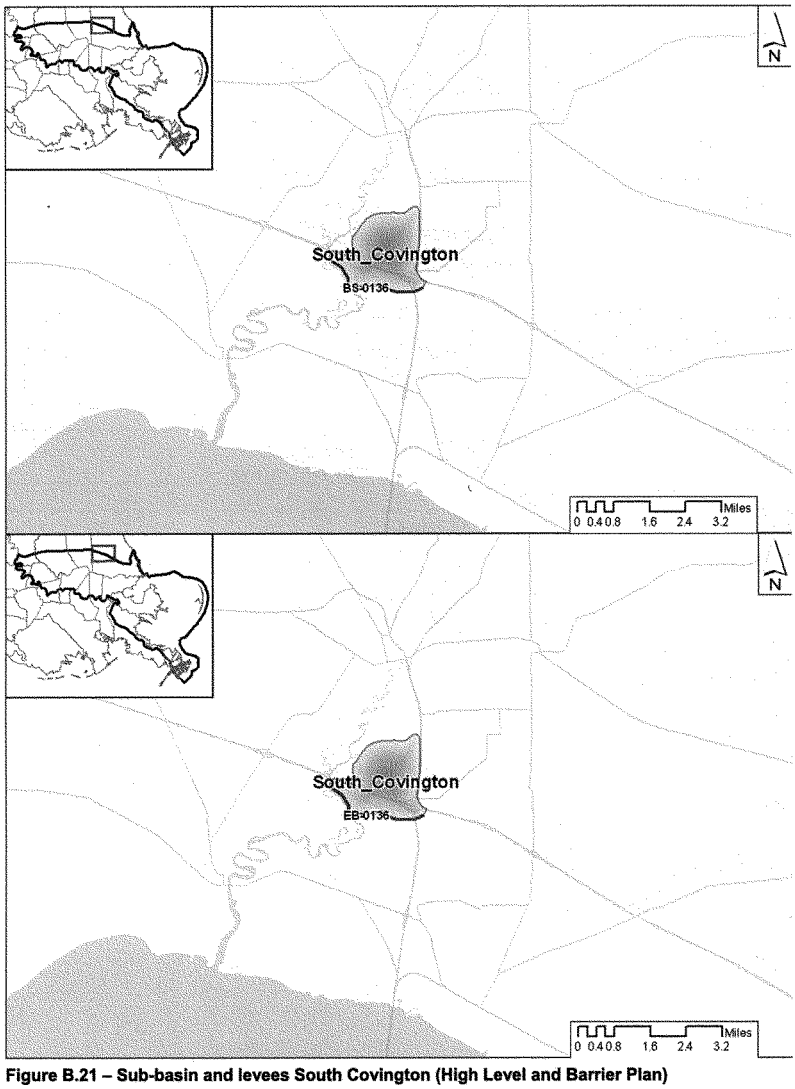


Figure B.21 – Sub-basin and levees South Covington (High Level and Barrier Plan)

Table B-11 – Levee characteristics South Covington

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0136	3.0	n.a.	13.6	16	17.4	16	20	22

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0136	3.0	n.a.	11	15.5	17.5	11	15.5	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	1,600	Based on 1cfs/acre

Storage Area	Type	Notes
South Covington	Single area	Max height is higher of surge (136) or design height (136)

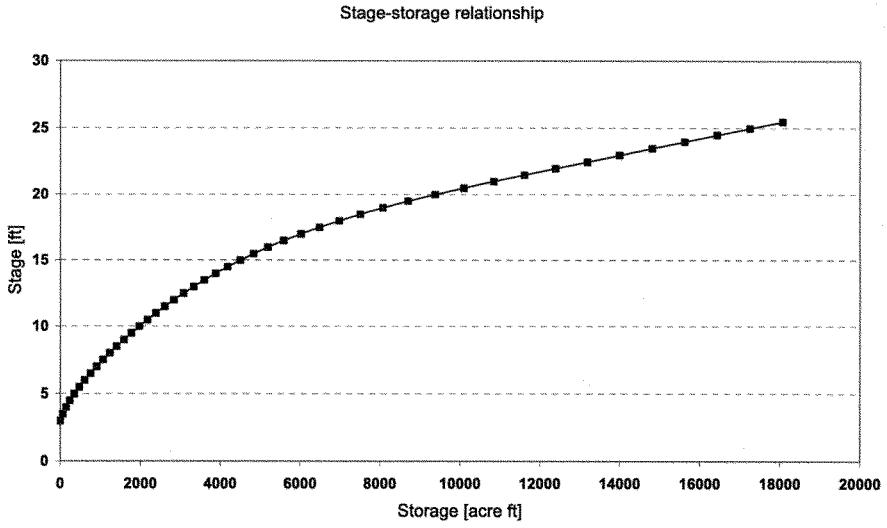


Figure B.22 – Stage storage relationship South Covington

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

None of the proposed levee system is likely to be subject to wave action as it is inland and protected from wave action buy other levees. Levee heights and internal flooding has therefore been developed by using a simple weiring process. The process and the results are given below.

It was decided to use the 90% surge level plus 3' as the design basis rather than using the levee height from the lakefront.

High Level (BS)

This results in the following levee heights

100 year	13.6'	(10.6' surge +3')
400 year	16'	(13' surge + 3')
1000 year	17.4'	(14.4' surge + 3')

The highest water level (2000yr 90%) is 15.2'. Therefore there is no significant overtopping of the 400 or 1000 year defenses and therefore only rainfall contributes to flooding.

At 100 year the 1000yr/90% and 2000yr/90% exceed the levee level and would cause additional flooding.

The peak overflow height would be 0.8' for the 1000/90 and 1.6' for the 2000/90

Overflow rates are the order of 2.4cfs/ft for 1000/90 and 6.7cfs/ft for 2000/90. This is a peak value and would only apply for a short time.

The time of overtopping has been established by developing a surge hydrograph and cutting the hydrograph at the levee height. Overtopping has been taken as occurring linearly from zero to peak and back again and an approximate volume of overtopping can be established using this assumption.

Time for 1000/90 above 13.6' is 240 mins. Therefore volume of overtopping is approx. 3151 acreft which equates to a flood level of 12.75' (nearly full to levee level)

Time for 2000/90 above 13.6' is 340 mins. This equates to a volume of approx. 25000 acre ft - this is full to surge level of 15.2'

Barrier Plan (EB)

This results in the following levee heights

100 year	10.3'	(7.3' surge +3')
400 year	12.8'	(9.8' surge + 3')
1000 year	14.1'	(11.1' surge + 3')

The highest water level (2000yr 90%) is 12.0'. Therefore there is no significant overtopping of the 400 or 1000 year defenses and therefore only rainfall contributes to flooding.

At 100 year the 1000yr/90% and 2000yr/90% exceed the levee level and would cause additional flooding.

The peak overflow height would be 0.8' for the 1000/90 and 1.7' for the 2000/90

The time of overtopping has been established by developing a surge hydrograph and cutting the hydrograph at the levee height. Overtopping has been taken as occurring linearly from zero to peak and back again and an approximate volume of overtopping can be established using this assumption.

Time for 1000/90 above 10.3' is 150 mins. Therefore volume of overtopping is approx. 3938 acre ft - this is full to surge level of 11.1'
Time for 2000/90 above 13.6' is 220 mins. This equates to a volume of approx. 17900 acre ft - this is full to surge level of 12.0'

Madisonville to Mandeville

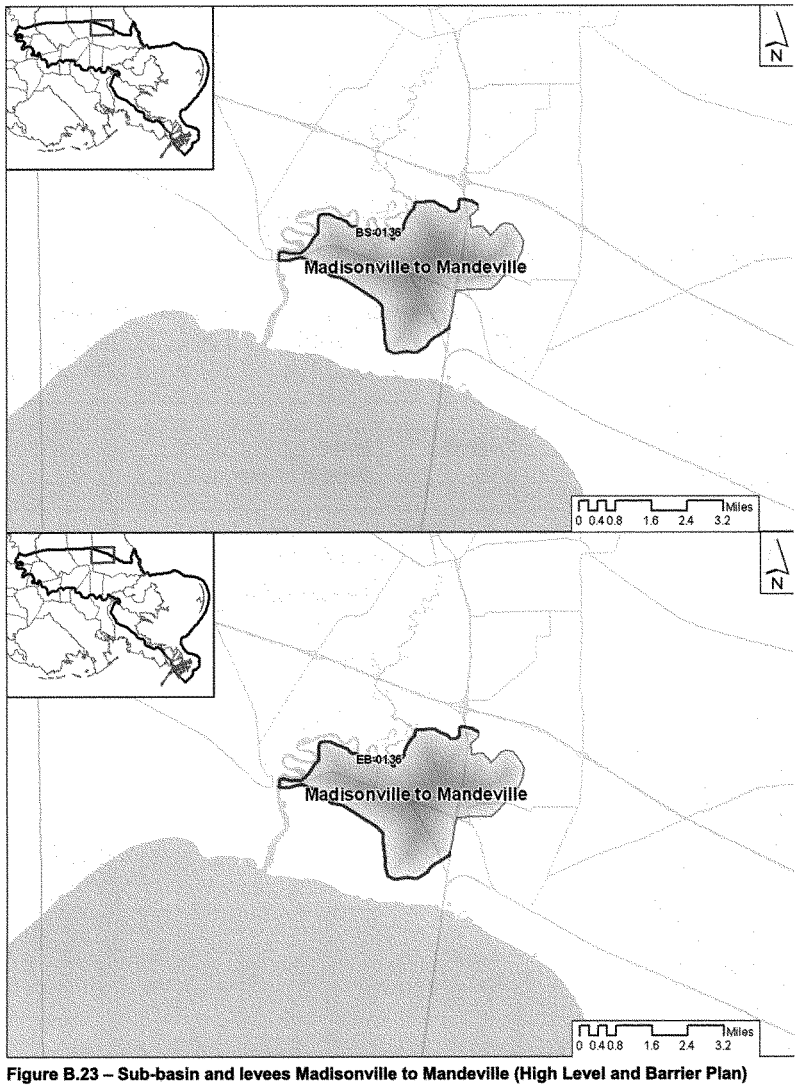


Figure B.23 – Sub-basin and levees Madisonville to Mandeville (High Level and Barrier Plan)

Table B-12 – Levee characteristics Madisonville to Mandeville

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0136	11.5	n.a.	16	20	22	16	20	22

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0136	11.5	n.a.	11	15.5	17.5	11	15.5	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	5,600	Based on 1cfs/acre

Storage Area	Type	Notes
Madisonville to Mandeville	Single area	Max stage is higher of surge (136) or design height (136)

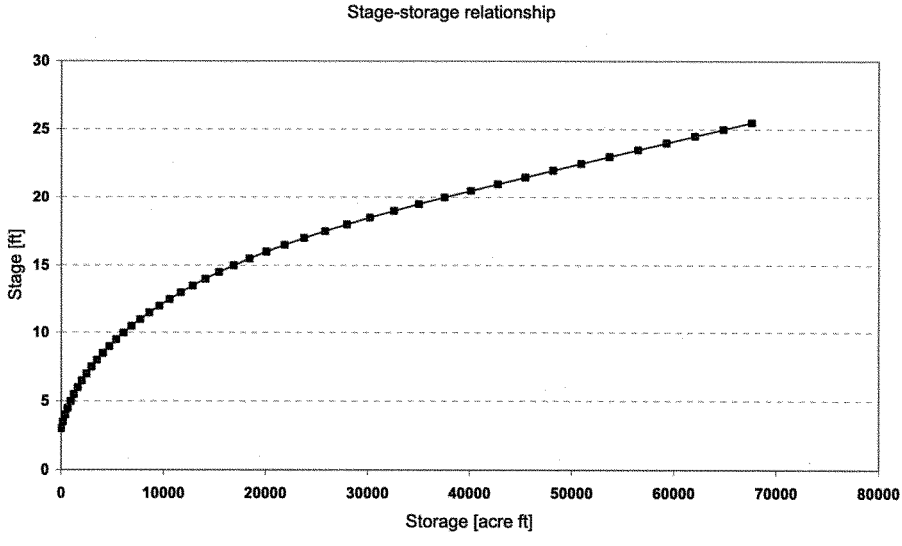


Figure B.24 – Stage storage relationship Madisonville to Mandeville

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

	High Level Plan				**Barrier Plan**			
	BS-0136 (miles)				**EB-0136 (miles)**			
Overtopping	4.1				4.1			
Non-overtopping	7.4				7.4			

Mandeville

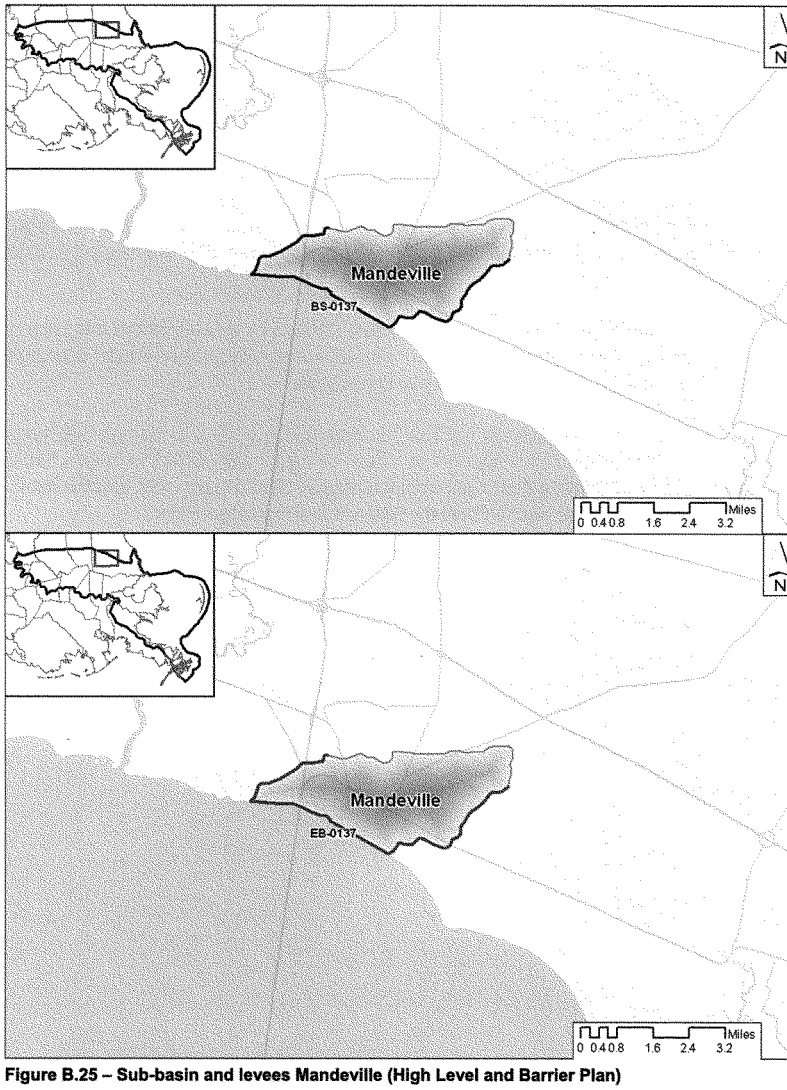


Figure B.25 – Sub-basin and levees Mandeville (High Level and Barrier Plan)

Table B-13 – Levee characteristics Mandeville

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0137	9.2	n.a.	15	19	21.5	15	19	21.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0137	9.2	n.a.	9.5	13.5	16	9.5	13.5	16

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	5,200	Based on 1cfs/acre

Storage Area	Type	Notes
Mandeville	Single area	Max stage is higher of surge (137) or design height (137)

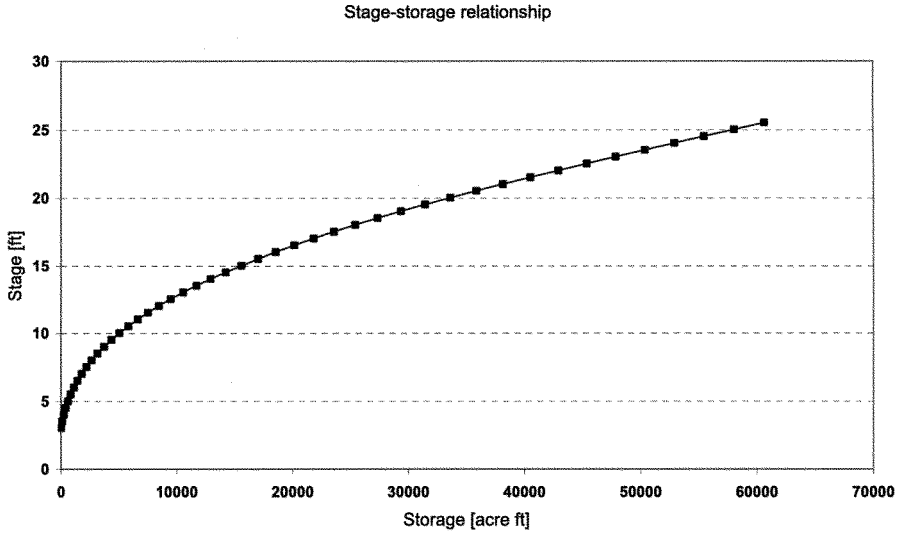


Figure B.26 – Stage storage relationship Mandeville

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

	High Level Plan				**Barrier Plan**			
	BS-0137 (miles)				**EB-0137 (miles)**			
Overtopping	5.5				5.5			
Non-overtopping	3.7				3.7			

West Lacombe

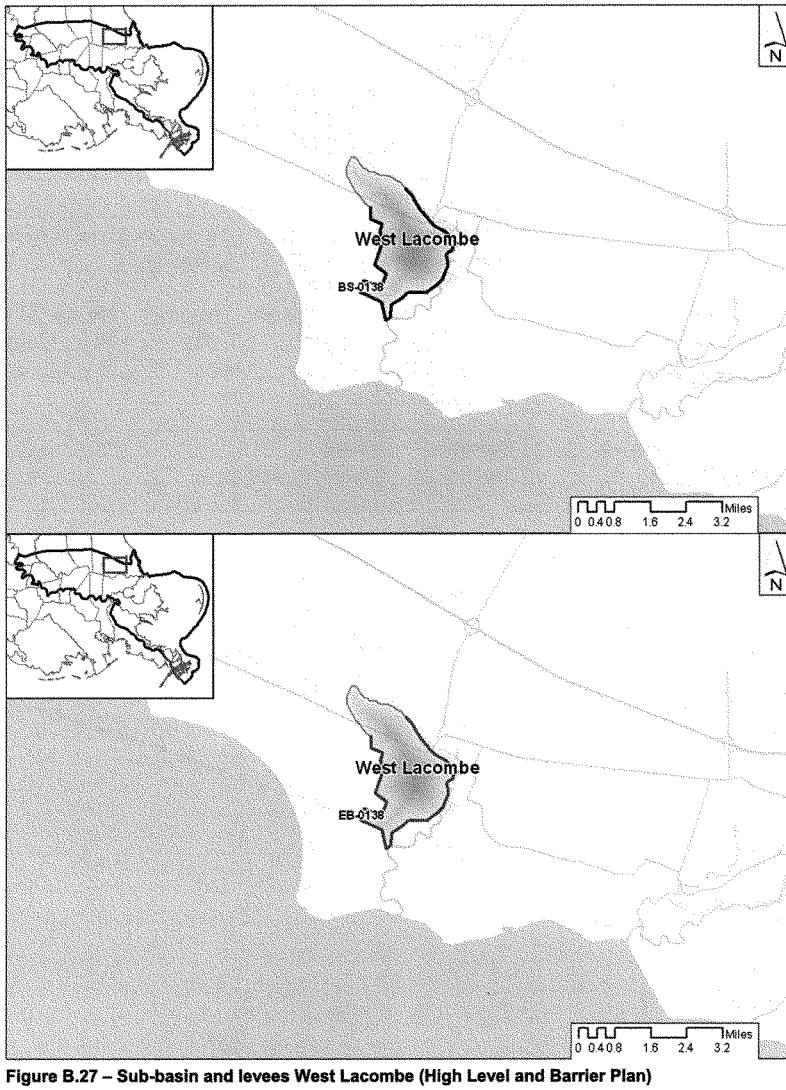


Figure B.27 – Sub-basin and levees West Lacombe (High Level and Barrier Plan)

Table B-14 – Levee characteristics West Lacombe

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0138	8.2	n.a.	15	20	22.5	15	20	22.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0138	8.2	n.a.	9.5	14.5	17.5	9.5	14.5	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	2,400	Based on 1cfs/acre

Storage Area	Type	Notes
West Lacombe	Single area	Max stage is higher of surge (138) or design height (138)

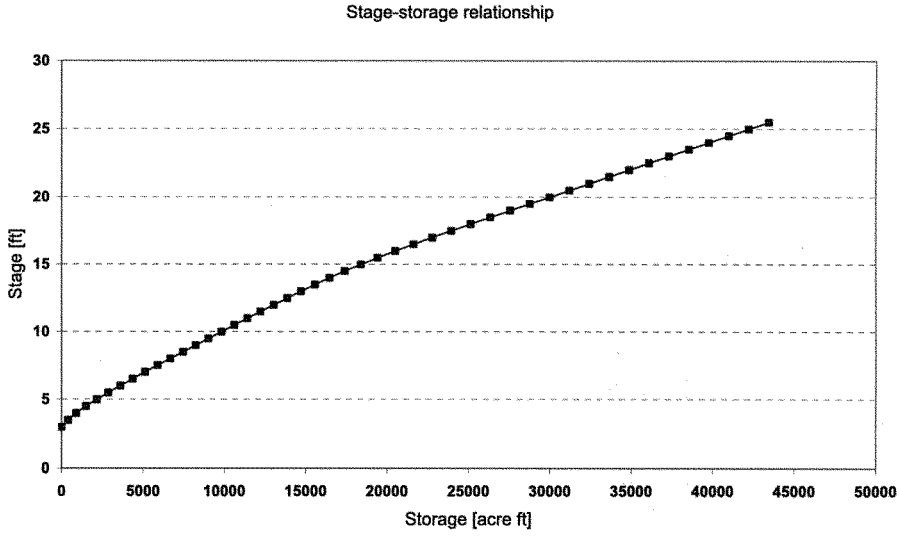


Figure B.28 – Stage storage relationship West Lacombe

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

	High Level Plan				**Barrier Plan**			
	BS-0138 (miles)				**EB-0138 (miles)**			
Overtopping	5.3				5.3			
Non-overtopping	2.9				2.9			

East Lacombe

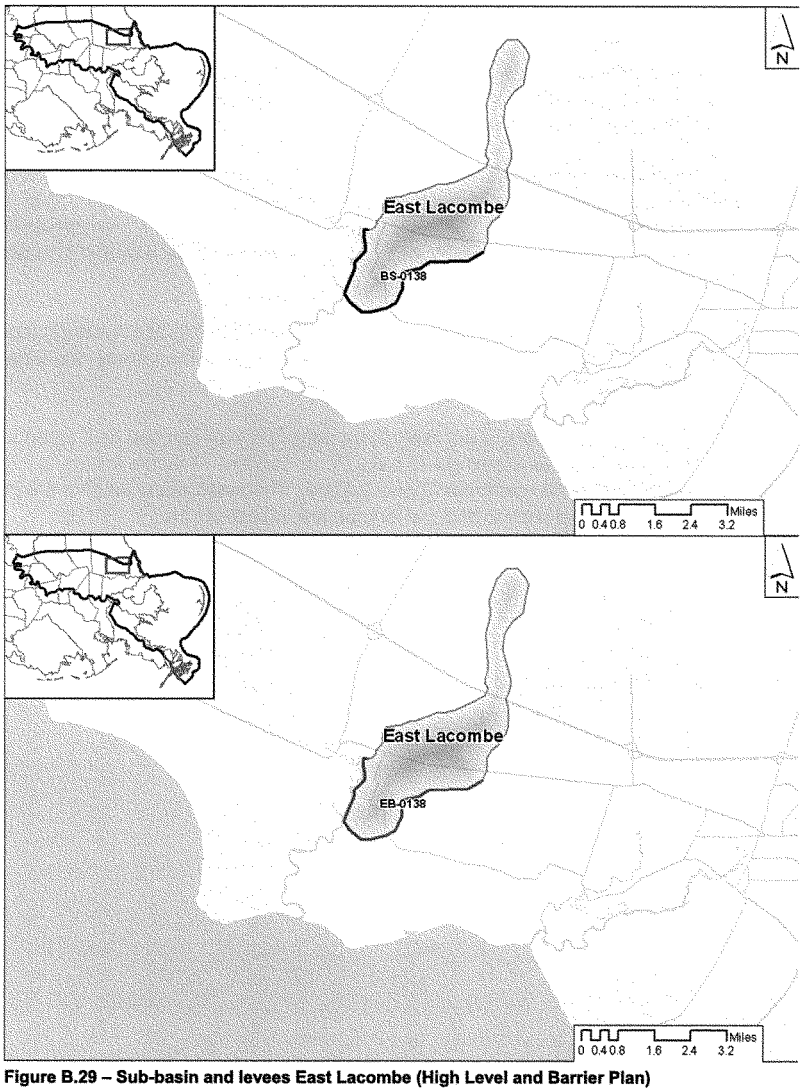


Figure B.29 – Sub-basin and levees East Lacombe (High Level and Barrier Plan)

Table B-15 – Levee characteristics East Lacombe

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0138	5.6	n.a.	15	20	22.5	15	20	22.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0138	5.6	n.a.	9.5	14.5	17.5	9.5	14.5	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	5,000	Based on 1cfs/acre

Storage Area	Type	Notes
East Lacombe	Single area	Max stage is higher of surge (138) or design height (138)

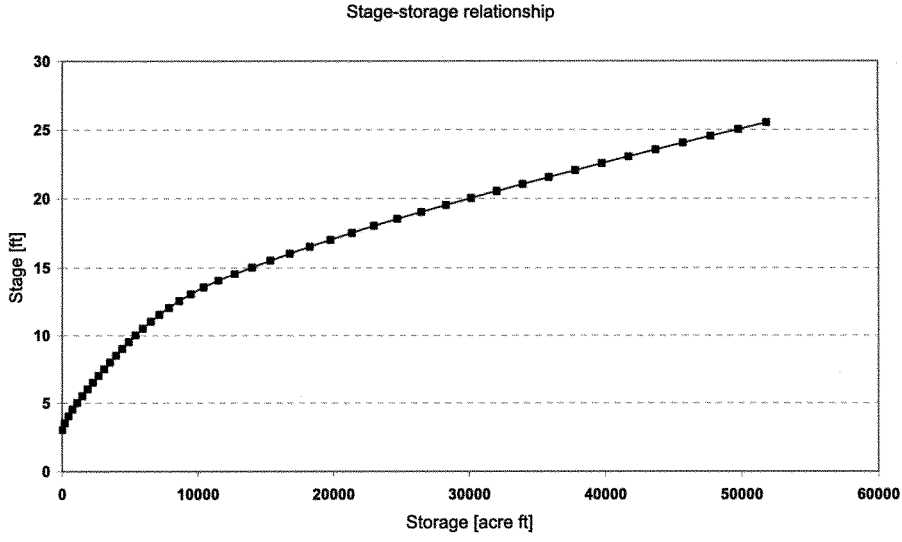


Figure B.30 – Stage storage relationship East Lacombe

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

	High Level Plan				**Barrier Plan**			
	BS-0138 (miles)				**EB-0138 (miles)**			
Overtopping	3.9				3.9			
Non-overtopping	1.7				1.7			

Slidell

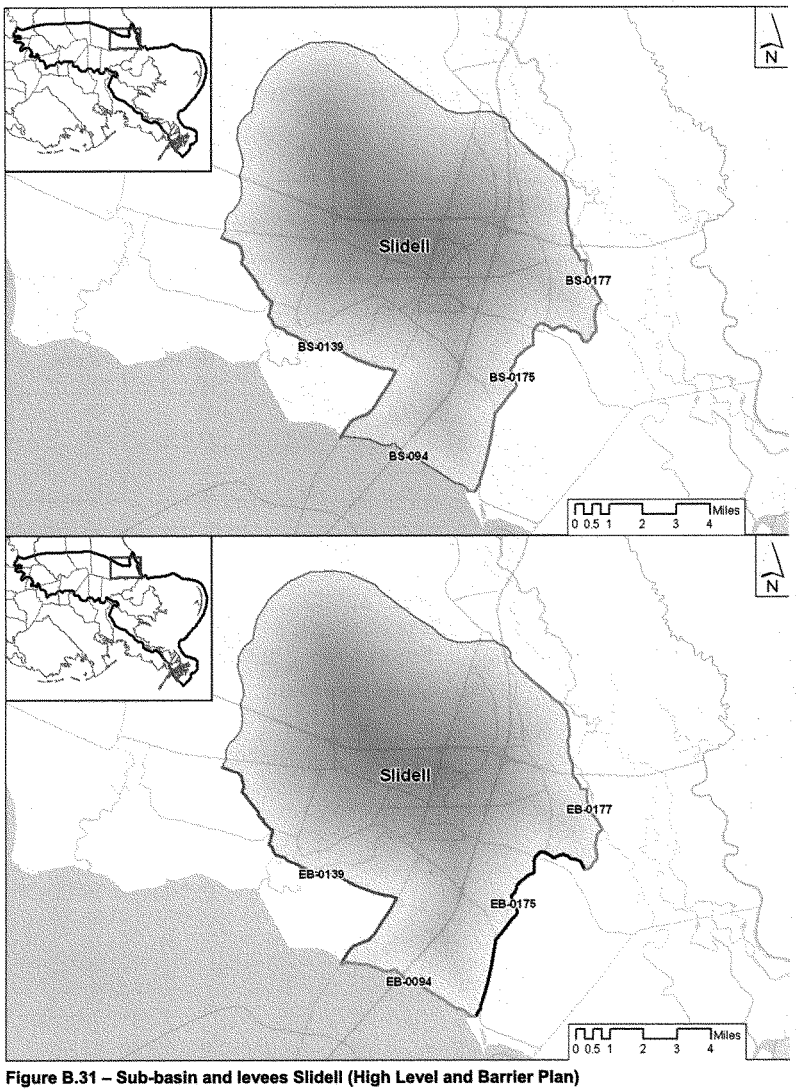


Figure B.31 – Sub-basin and levees Slidell (High Level and Barrier Plan)

Table B-16 – Levee characteristics Slidell

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0139	10.0	n.a.	16	21.5	24	16	21.5	24
BS-0094	4.6	n.a.	16.5	23	26	16.5	23	16
BS-0175	7.3	n.a.	22.5	30.5	24.5	19	25.5	29
BS-0177	7.1	n.a.	20	28	32	17.5	23.5	26.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0139	10.0	n.a.	11	17.5	21	11	17.5	21
EB-0094	4.6	n.a.	13	21	24.5	12.5	20.5	24.5
EB-0175	7.3	n.a.						
EB-0177	7.1	n.a.	26	35	39	20.5	27	30

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	54,000	Based on 1cfs/acre

Storage Area	Type	Notes
Slidell	Single area	Max stage is higher of surge (94) or design height (94) - see note

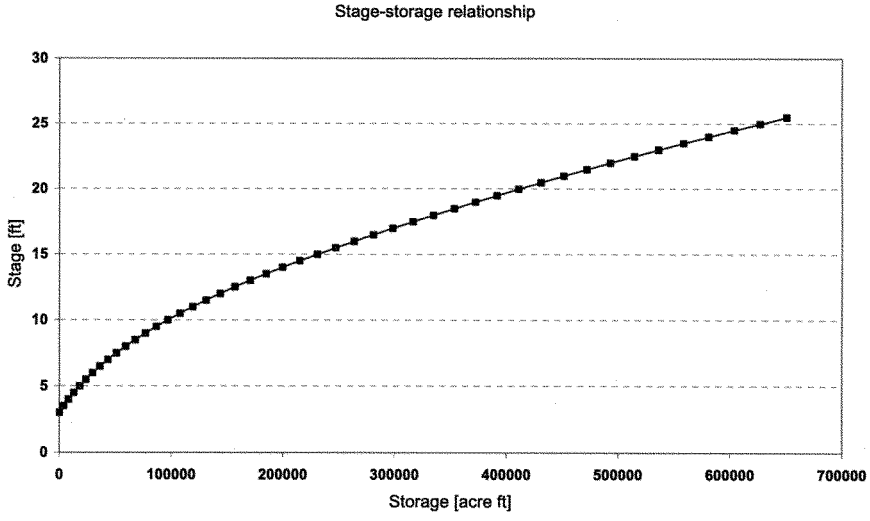


Figure B.32 – Stage storage relationship Slidell

Additional Notes

The Lidar levels on the North Shore show very low levels and the storage curves would extend to well below 0 datum. It has been decided to assume that there is no storage available below 3' in all areas on the North shore when levees are to be constructed. The stage storage relationships used in the calculations have been amended to this for both the high level and barrier plans and this is the curve shown in the graph above.

The location of the levee has resulted in only a small area of storage adjacent to the proposed levees, and as such the degree of pumping has been increased to compensate for this, compared with the levels of pumping assumed in the central New Orleans area where the drainage channels and roads absorb much more of the initial flooding. For the North Shore a rate of 1cfs/acre has been adopted.

The areas for calculating rainfall volumes have been derived by looking at the natural topography and estimating the drainage area. No allowance has been made for man-made drainage routes which may affect the total volumes within a particular area.

Much of the proposed levee system is not likely to be subject to wave action as it extends along the banks of the major tributaries draining into Lake Pontchartrain. Only the levees facing the Lake have been used within the overtopping calculations. The overtopping lengths are indicated on the plans at the start of the section and the lengths are indicated here:

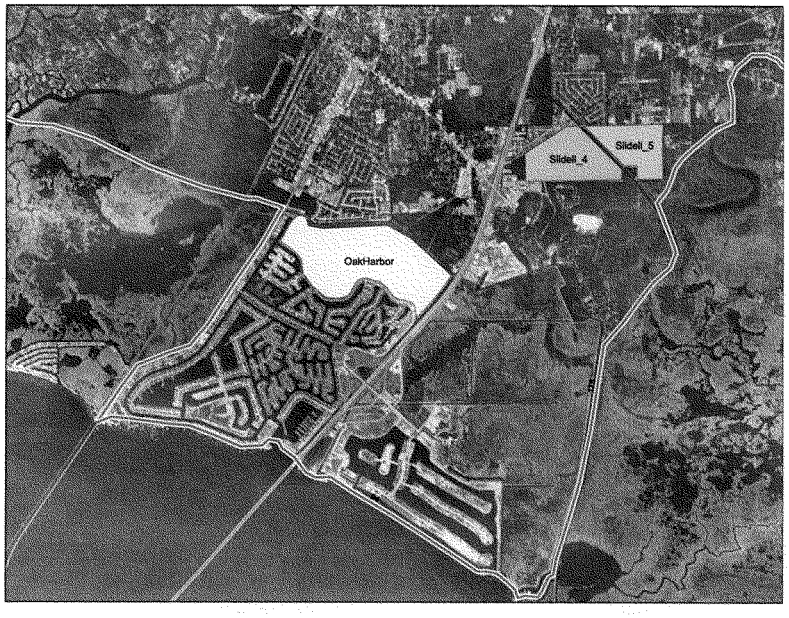
	High Level Plan				Barrier Plan			
	BS-0139 (miles)	BS-0094 (miles)	BS-0175 (miles)	BS-0177 (miles)	EB-0139 (miles)	EB-0094 (miles)	EB-0175 (miles)	EB-0177 (miles)
Overtopping	10.0	4.6	7.3	4.5	10.0	4.6	7.3	4.5
Non-overtopping	0.0	0.0	0.0	2.6	0.0	0.0	0.0	2.6

In addition there are three areas within the proposed new levee system which already have non-federal levees which will affect the flood levels. These areas have been called Oak Harbor, Slidell 4 and Slidell 5. They are shown on the figure below. For this assessment the levees have been taken as having a crest elevation of 11'.

Using stage storage curves for these areas the 10 year rainfall without pumping results in the following flood levels.

Oak Harbor 3.8ft
Slidell_4 6.2ft
Slidell_5 4.8ft

This will not cause excessive flooding so additional pumping within theses areas has not been included. These values have been used for the inside of these areas apart from when the interior stage for the rest of the area exceeds 11' (the nominal levee height) when they were assumed to fill to the same level as outside the levees.


Slidell_4
Slidell_5
OakHarbor

Planning unit 2 – Barataria Basin

Algiers

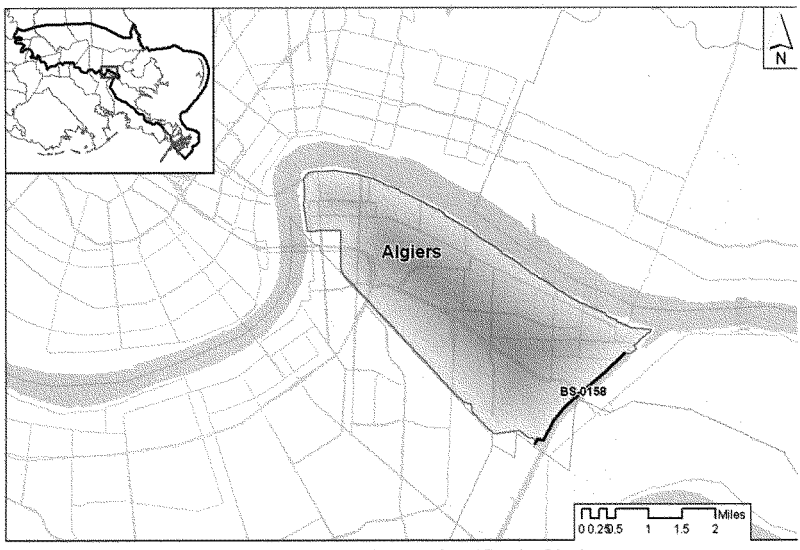


Figure B.33 – Sub-basin and levees Algiers (High Level and Barrier Plan)

Table B-17 – Levee characteristics Algiers

High Level Plan/ Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0158	2.0	10	10	15	17.5	10	15	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	4,650	4,650	

Storage Area	Type	Notes
Algiers	Single area	Max stage is higher of surge (158) or authorized (10') or design height (158)

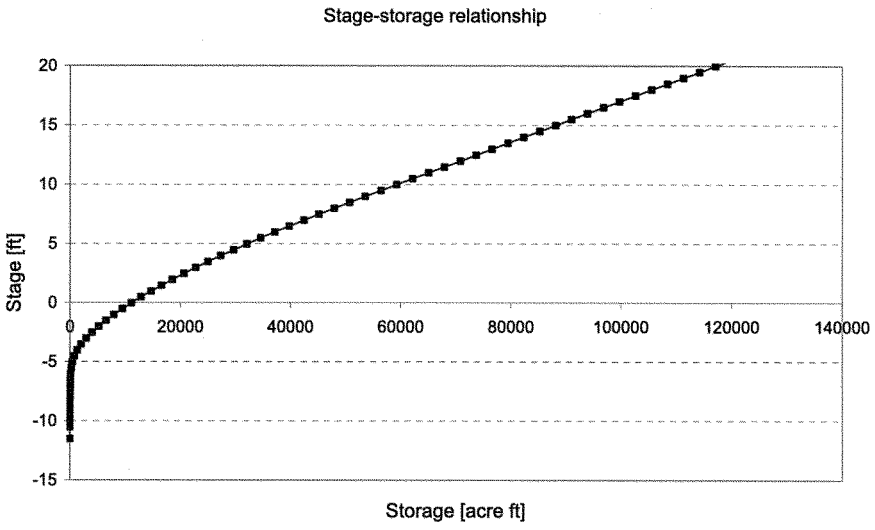


Figure B.34 – Stage storage relationship Algiers

English Turn

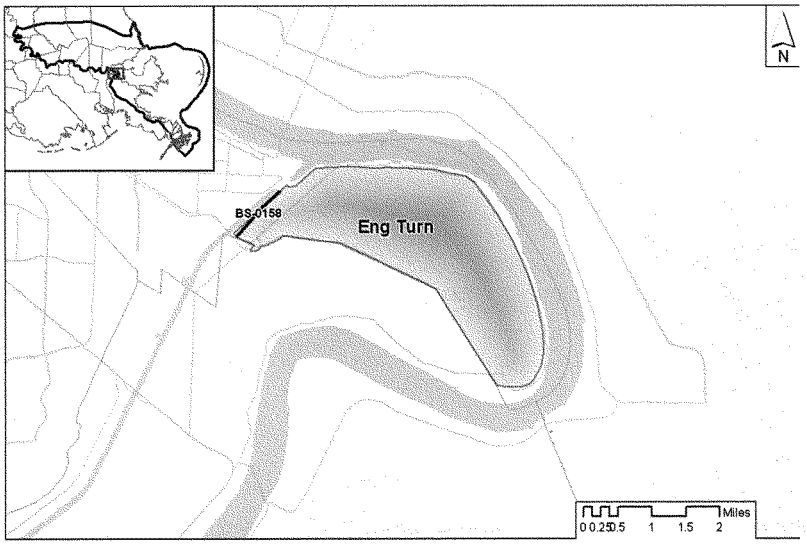


Figure B.35– Sub-basin and levees English Turn (High Level and Barrier Plan)

Table B-18 – Levee characteristics English Turn

High Level Plan/ Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0158	0.9	10	10	15	17.5	10	15	17.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	1,670	1,670	

Storage Area	Type	Notes
English Turn	Single area	Max stage is higher of surge (158) or authorized (10') or design height (158)

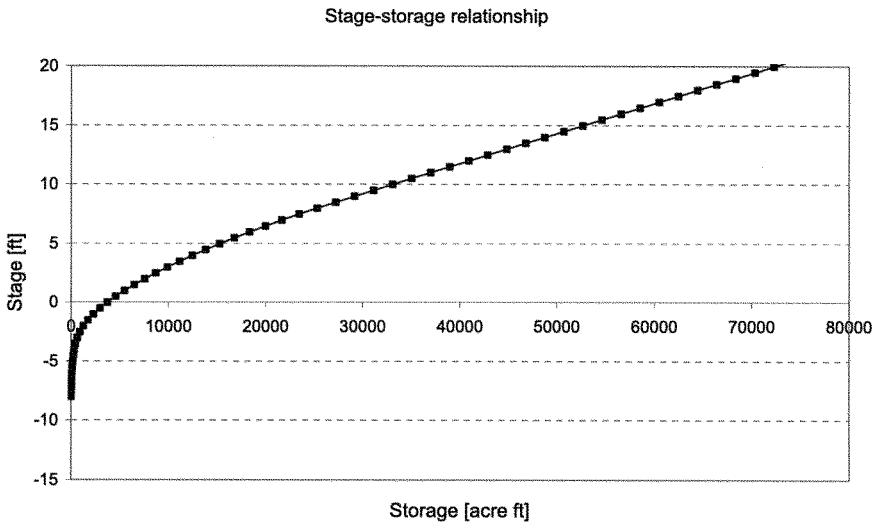


Figure B.36 – Stage storage relationship English Turn

Plaquemaines Belle Chase

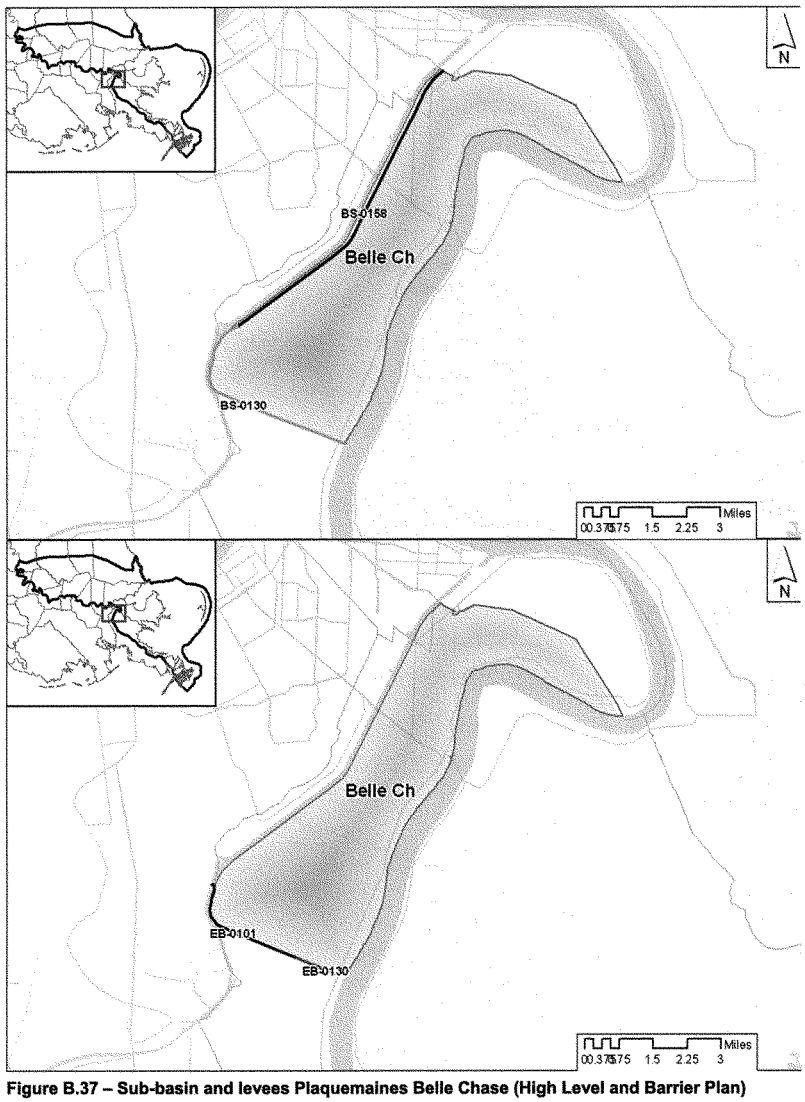


Figure B.37 – Sub-basin and levees Plaquemaines Belle Chase (High Level and Barrier Plan)

Table B-19 – Levee characteristics Plaquemaines Belle Chase

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0130	4.8	11	11	17	20	11	15	17
BS-0158	7.4	10	10	15	17.5	10	15	17.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	2.9	11	11	11	11	11	11	11
EB-0130	1.0	11	16	22	25	14	19	22

	Existing	Proposed	Notes
Pump Capacity (cfs)	4,650	4,650	

Storage Area	Type	Notes
Plaquemines Belle Chase	Single area	Max height is higher of authorized (11'), surge (130) or design height (130)

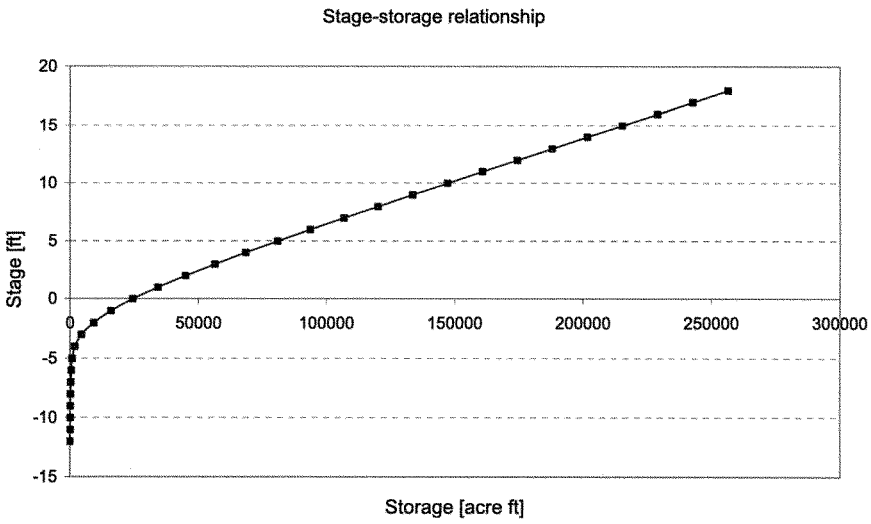


Figure B.38 – Stage storage relationship Plaquemaines Belle Chase

West Jefferson – East of Harvey

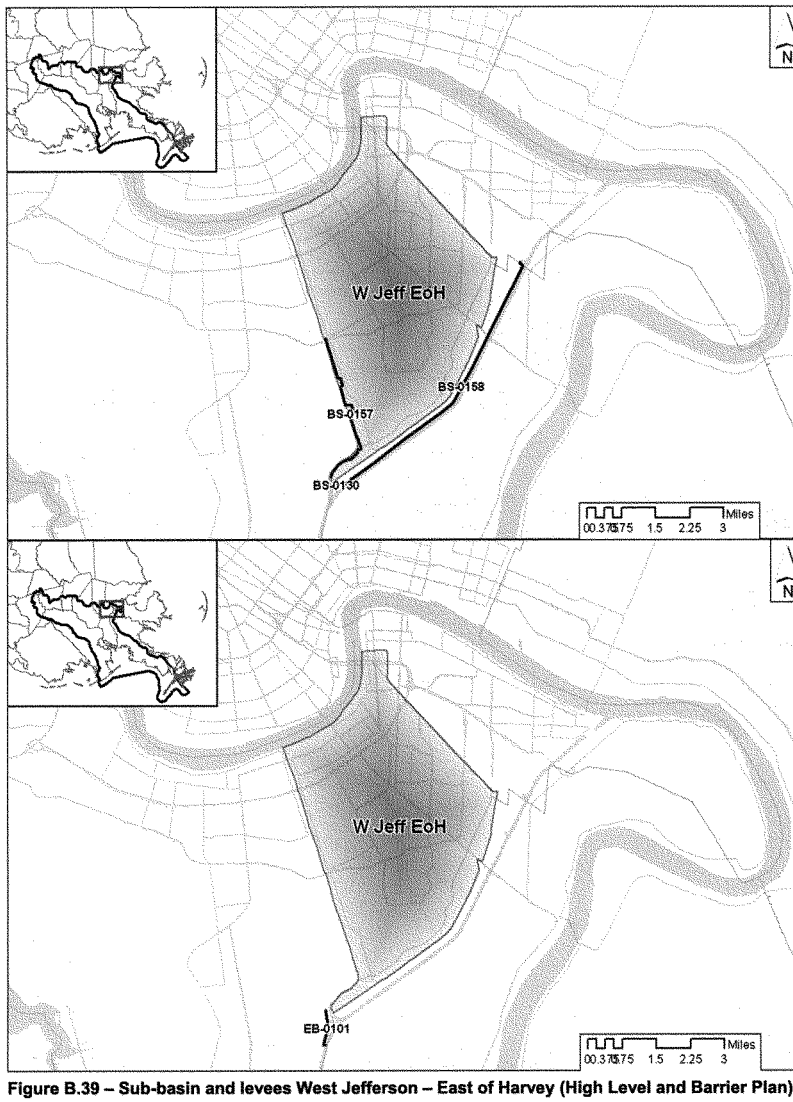


Figure B.39 – Sub-basin and levees West Jefferson – East of Harvey (High Level and Barrier Plan)

Table B-20 – Levee characteristics West Jefferson – East of Harvey

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0130	0.3	11	11	17	20	11	15	17
BS-0157	3.8	10	10	14	16.5	10	14	16
BS-0158	6.5	10	10	15	17.5	10	15	17.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	0.8	11	11	11	11	11	11	11

	Existing	Proposed	Notes
Pump Capacity (cfs)	10,430	10,430	

Storage Area	Type	Notes
West Jeff - East of Harvey	Linked area (West Jeff)	Connects to Harvey (5'). See description of linked system in chapter 5

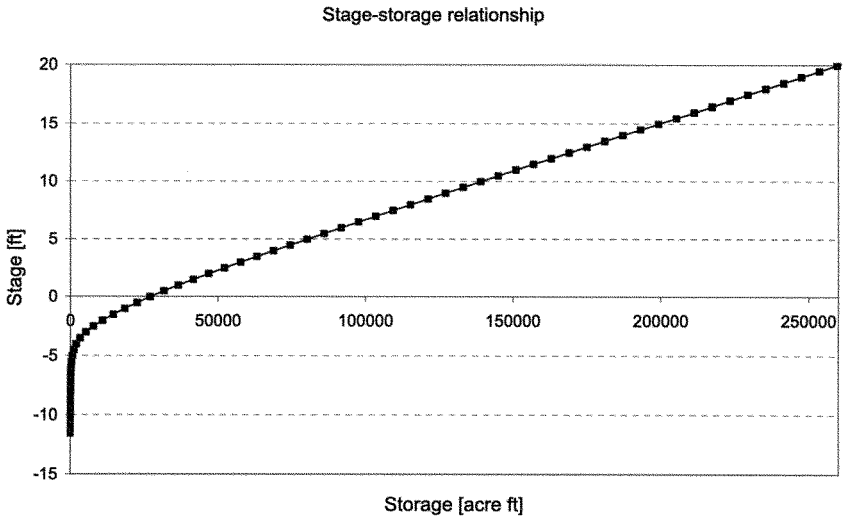


Figure B.40 – Stage storage relationship West Jefferson – East of Harvey

West Jefferson – Harvey

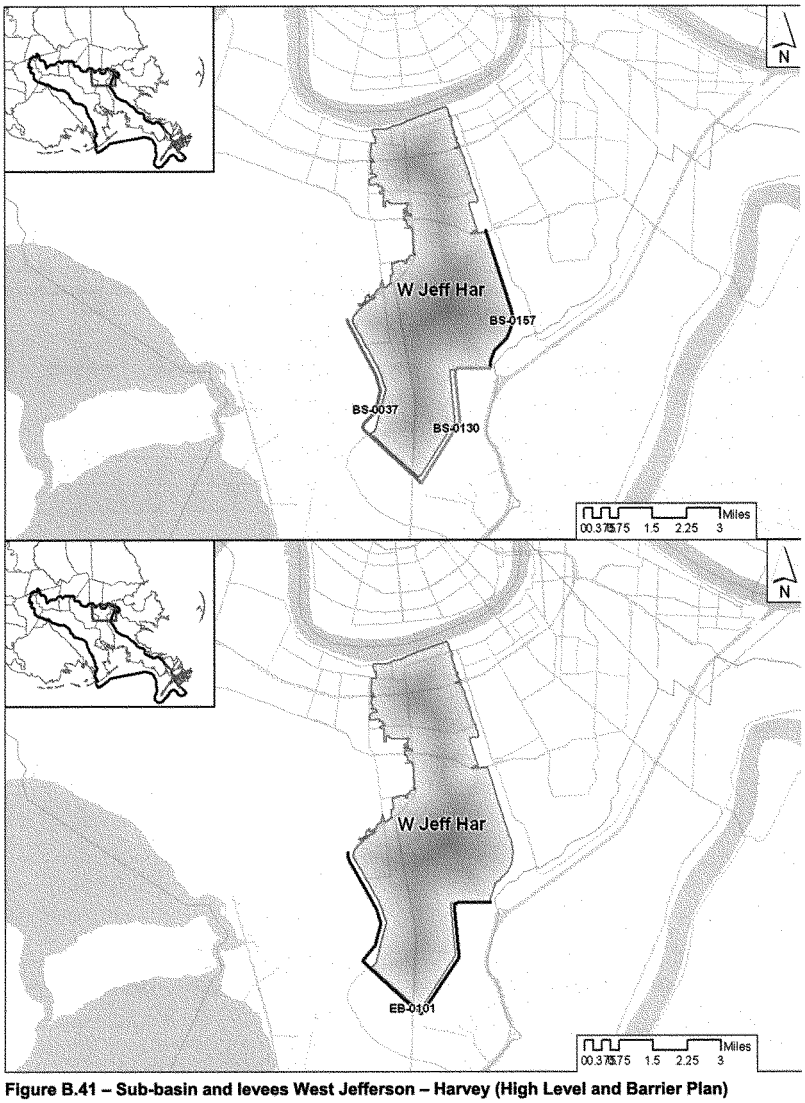


Figure B.41 – Sub-basin and levees West Jefferson – Harvey (High Level and Barrier Plan)

Table B-21 – Levee characteristics West Jefferson - Harvey

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0037	4.4	11	11	14	18	11	12.5	16
BS-0130	3.5	11	11	17	20	11	15	17
BS-0157	3.3	10	10	14	16.5	10	14	16

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	7.9	11	11	11	11	11	11	11

	Existing	Proposed	Notes
Pump Capacity (cfs)	7,930	7,930	

Storage Area	Type	Notes
West Jeff - Harvey	Linked area (West Jeff)	Connects to EoH (5') and Ames (5'). See description of linked system in chapter 5.

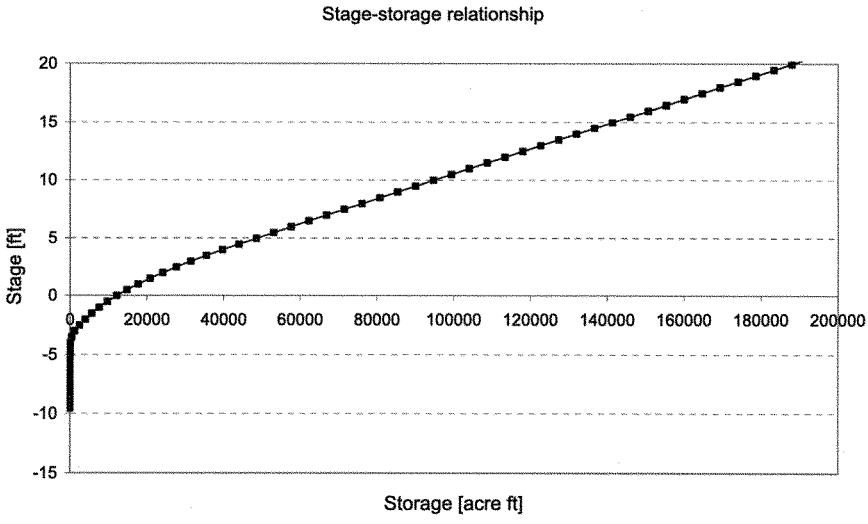


Figure B.42 – Stage storage relationship West Jefferson – Harvey

West Jefferson – Ames

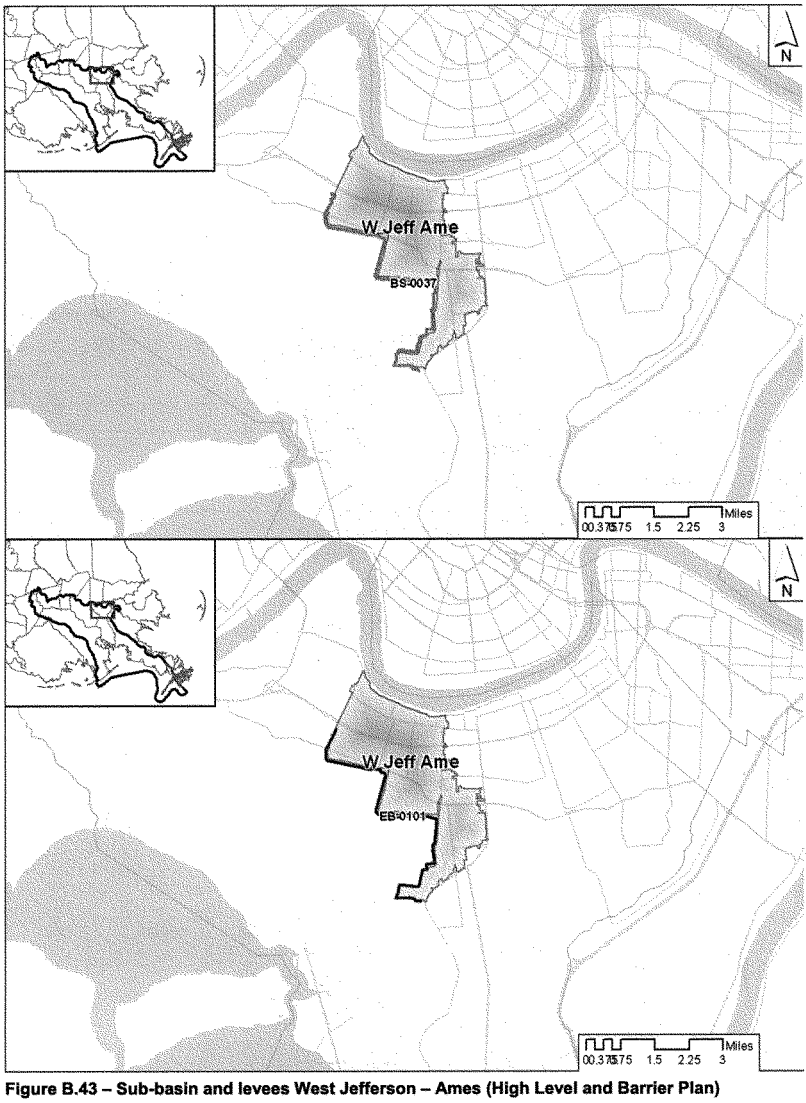


Figure B.43 – Sub-basin and levees West Jefferson – Ames (High Level and Barrier Plan)

Table B-22 – Levee characteristics West Jefferson – Ames

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0037	7.5	11	11	14	18	11	12.5	16

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	7.5	11	11	11	11	11	11	11

	Existing	Proposed	Notes
Pump Capacity (cfs)	4,760	4,760	

Storage Area	Type	Notes
West Jeff - Ames	Linked area (West Jeff)	Connects to Segnette (5') and fed from Harvey (5'). See description of linked system in chapter 5

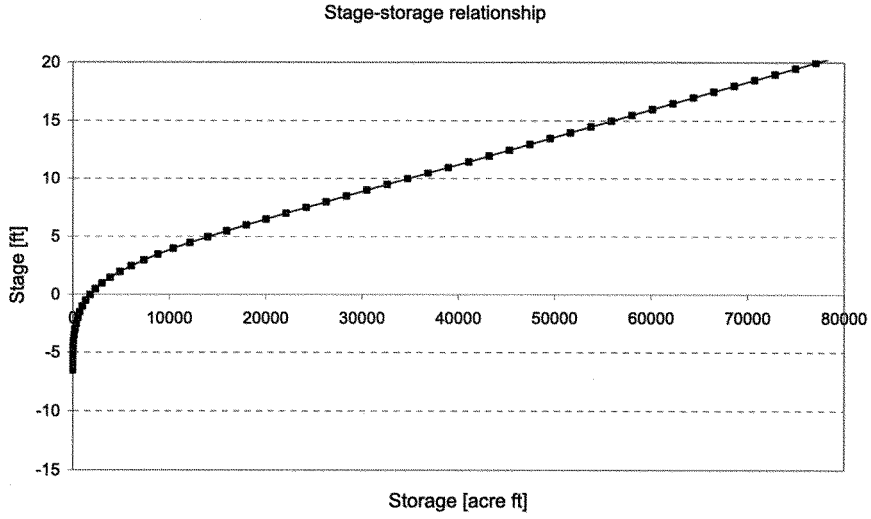


Figure B.44 – Stage storage relationship West Jefferson – Ames

West Jefferson – Segnette

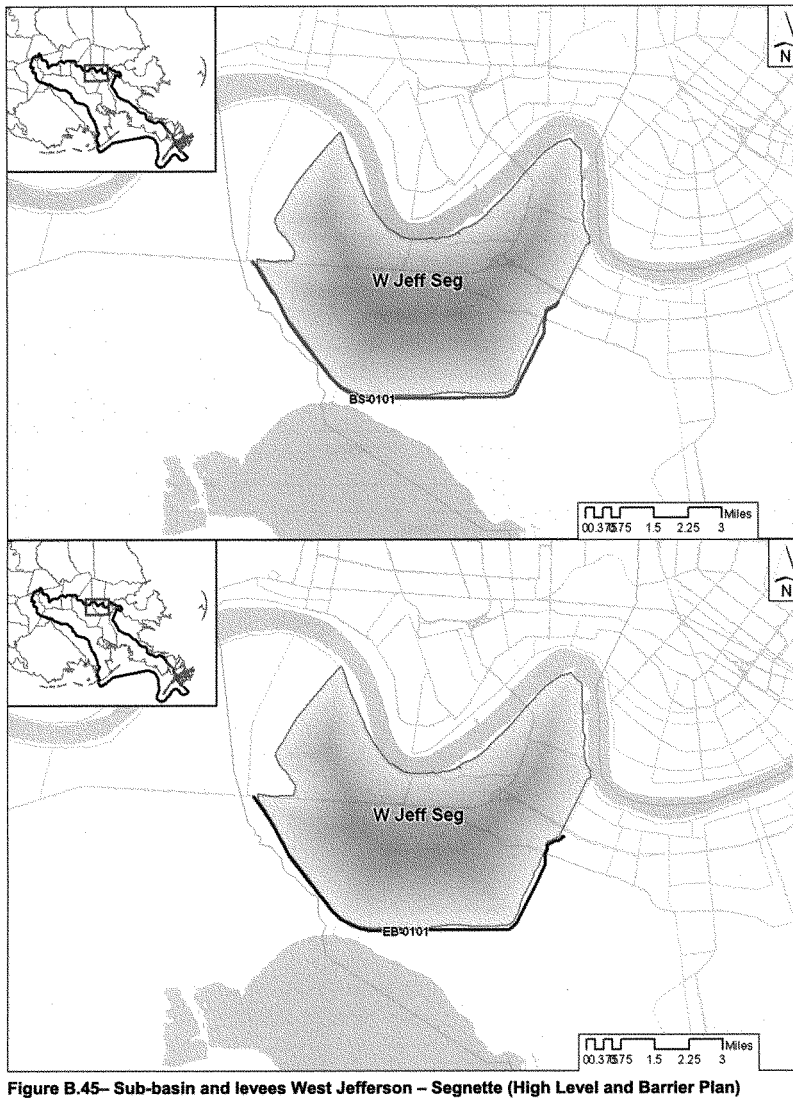


Figure B.45– Sub-basin and levees West Jefferson – Segnette (High Level and Barrier Plan)

Table B-23 – Levee characteristics West Jefferson – Segnette

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0101	9.6	11	11	14	16	11	14	16

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	9.6	11	11	11	11	11	11	11

	Existing	Proposed	Notes
Pump Capacity (cfs)	2,000	2,000	

Storage Area	Type	Notes
West Jeff - Segnette	Linked area (West Jeff)	Connects to Ames (5'). See description of linked system in chapter 5.

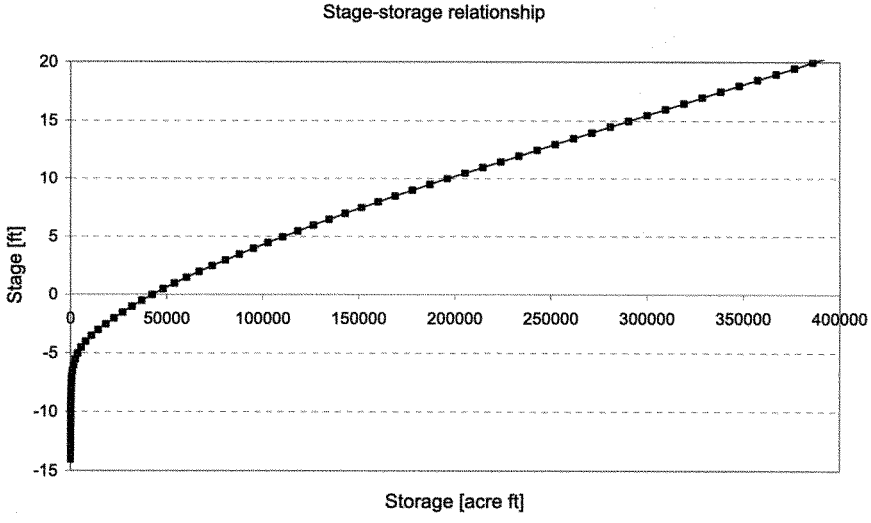


Figure B.46 – Stage storage relationship West Jefferson – Segnette

St Charles – Davis Pond

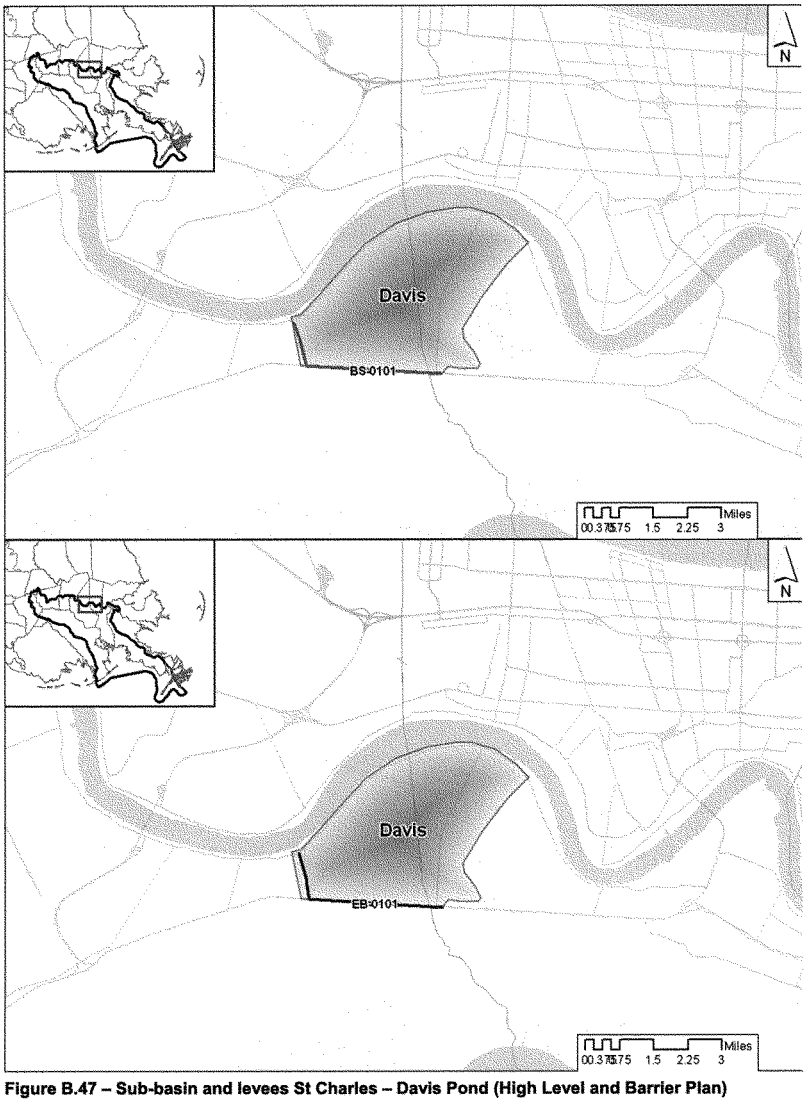


Figure B.47 – Sub-basin and levees St Charles – Davis Pond (High Level and Barrier Plan)

Table B-24 – Levee characteristics St Charles – Davis Pond

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0101	4.0	11	11	14	16	11	14	16

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0101	4.0	11	11	11	11	11	11	11

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	0	

Storage Area	Type	Notes
St Charles - Davis pond	Single area	Max stage is higher of surge (101) or authorized (11') or design height (101)

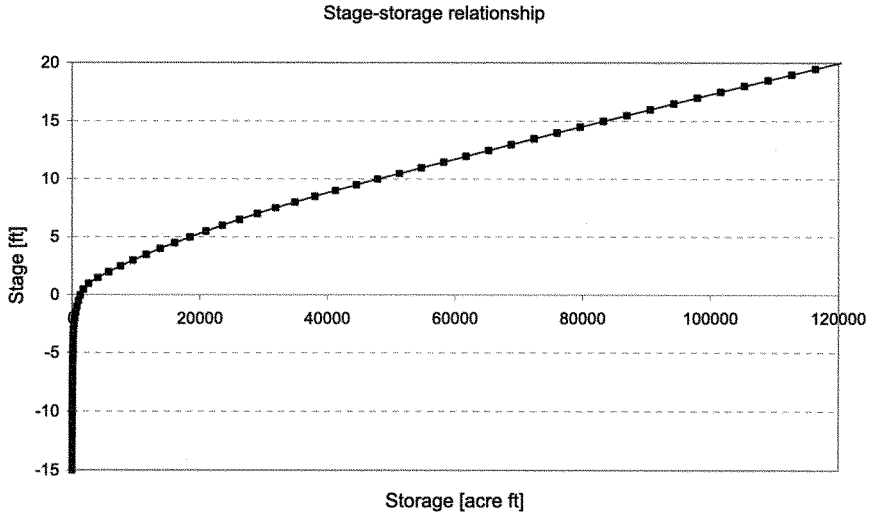


Figure B.48 – Stage storage relationship St Charles – Davis Pond

St Charles – Lulling

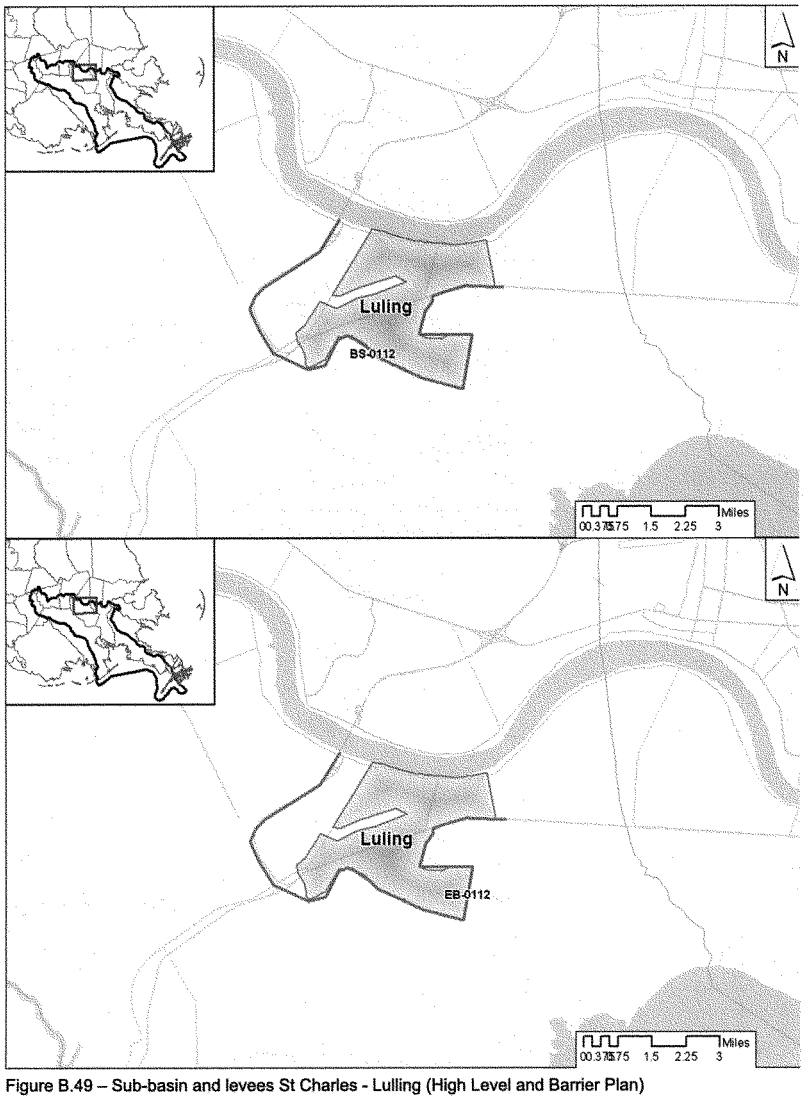


Figure B.49 – Sub-basin and levees St Charles - Lulling (High Level and Barrier Plan)

Table B-25 – Levee characteristics St Charles – Lulling

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0112	13.8	n.a.	8.5	12	13.5	8	11	13

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0112	13.8	n.a.	6.5	9	10.5	6	9	10

	Existing	Proposed	Notes
Pump Capacity (cfs)	3,150	3,150	

Storage Area	Type	Notes
St Charles - Luling	Single area	Max stage is higher of surge (112) or design height (112)

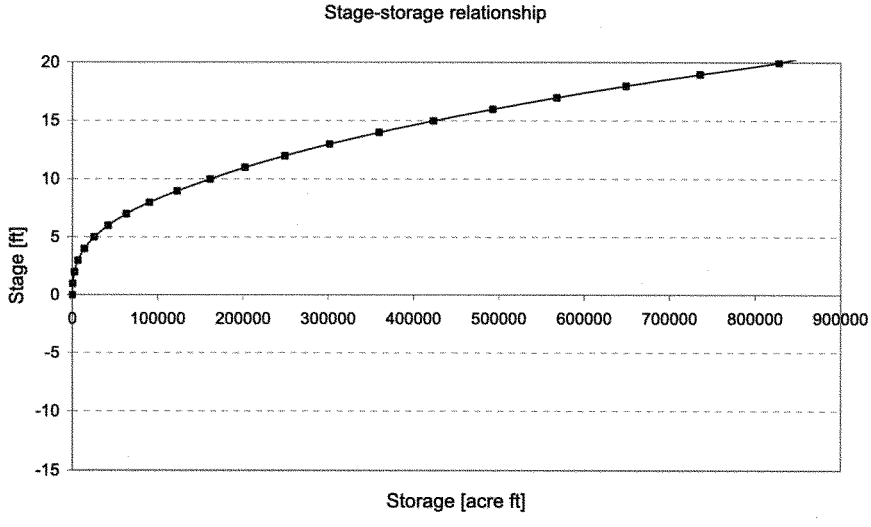


Figure B.50 – Stage storage relationship St Charles - Lulling

St Charles – Sunset

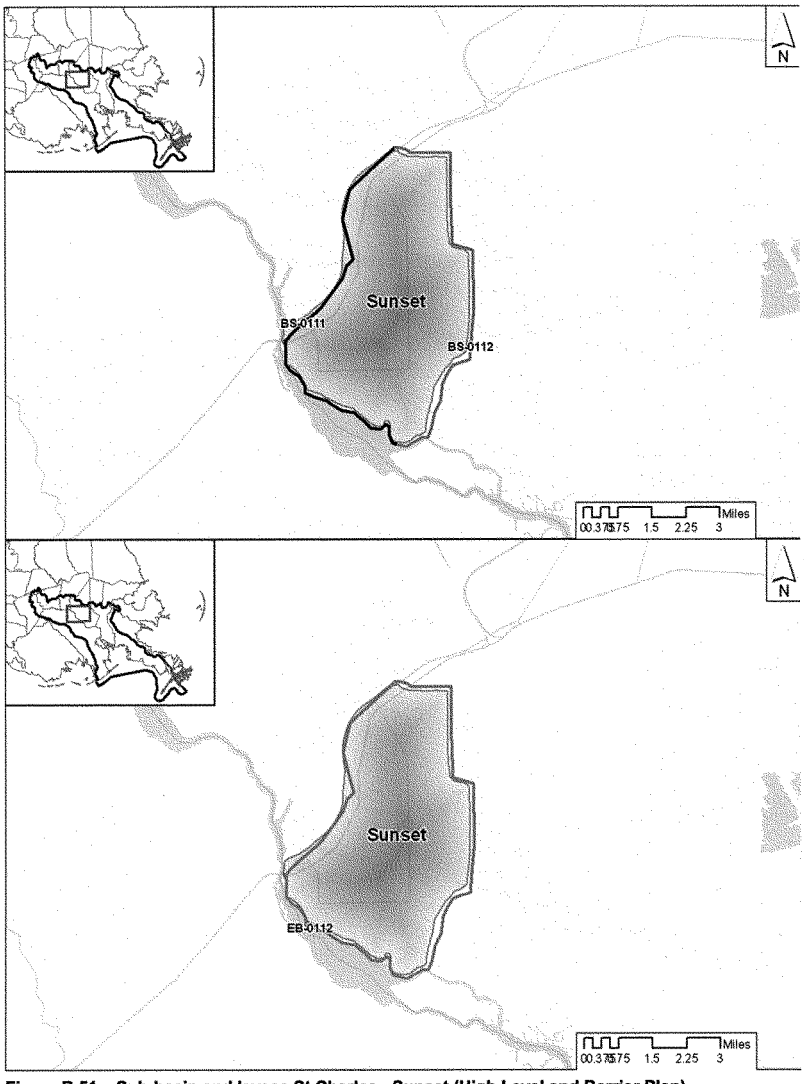


Figure B.51 – Sub-basin and levees St Charles - Sunset (High Level and Barrier Plan)

Table B-26 – Levee characteristics St Charles – Sunset

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0111	9.3	n.a.	8	11.5	13	8	11.5	13
BS-0112	8.9	n.a.	8.5	12	13.5	8	11	13

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0112	18.2	n.a.	6.5	9	10.5	6	9	10

	Existing	Proposed	Notes
Pump Capacity (cfs)	800	800	

Storage Area	Type	Notes
St Charles - Sunset	Single area	Max stage is higher of surge (112) or design height (112)

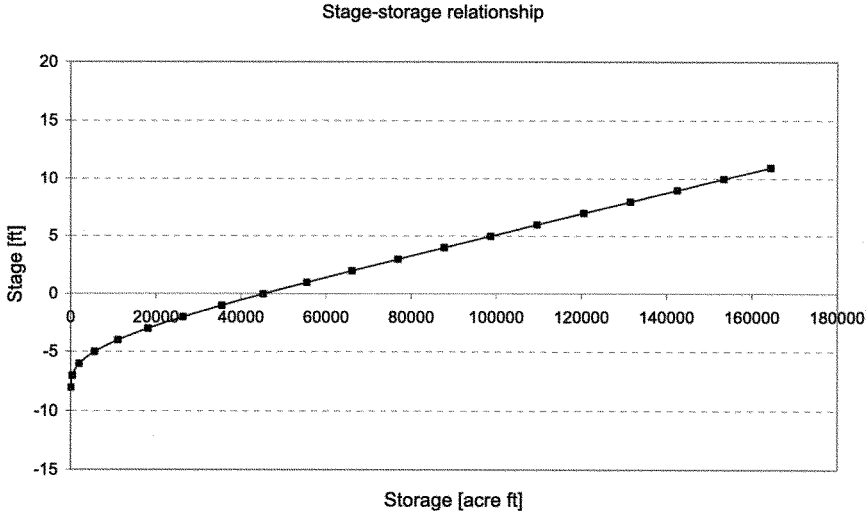


Figure B.52 – Stage storage relationship St Charles - Sunset

Lockport

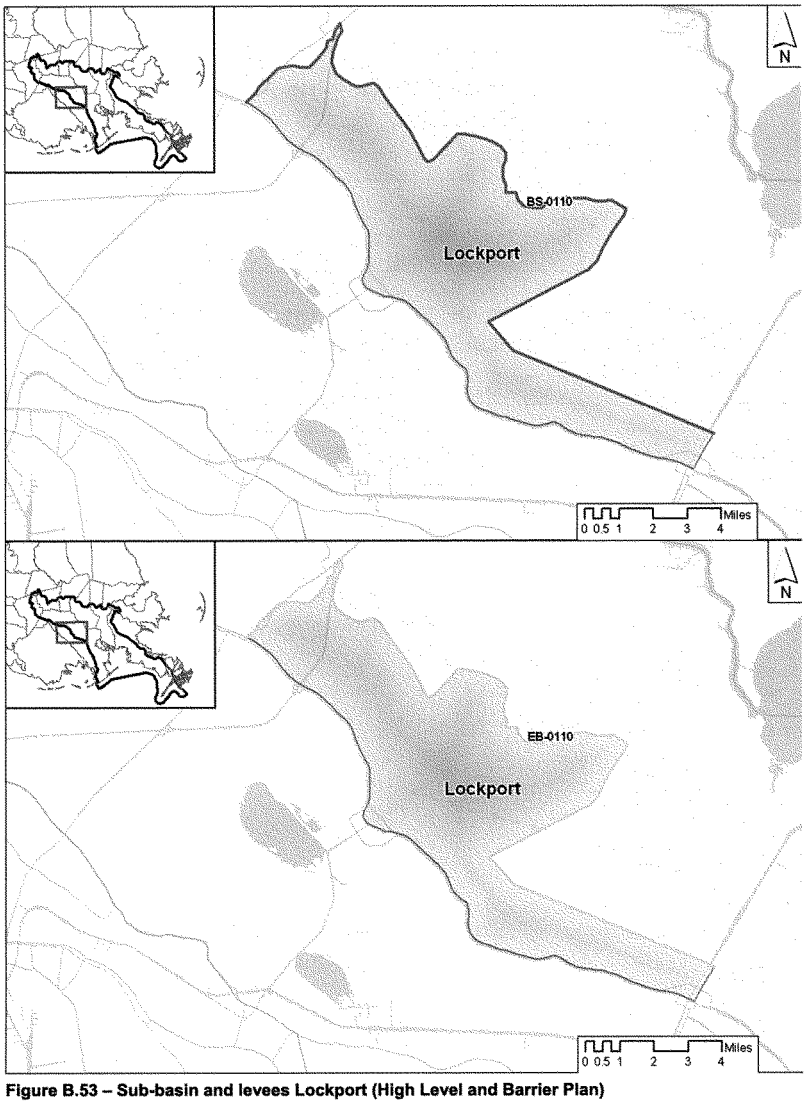


Figure B.53 – Sub-basin and levees Lockport (High Level and Barrier Plan)

Table B-27 – Levee characteristics Lockport

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0112	30.4	n.a.	8.5	12	13.5	8	11	13

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0500	30.4	n.a.	6	8.5	10	6	8	10

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	0	

Storage Area	Type	Notes
Lockport	Single area	Max stage is higher of surge (110BS or 101EB) or design height (110BS or 101EB)

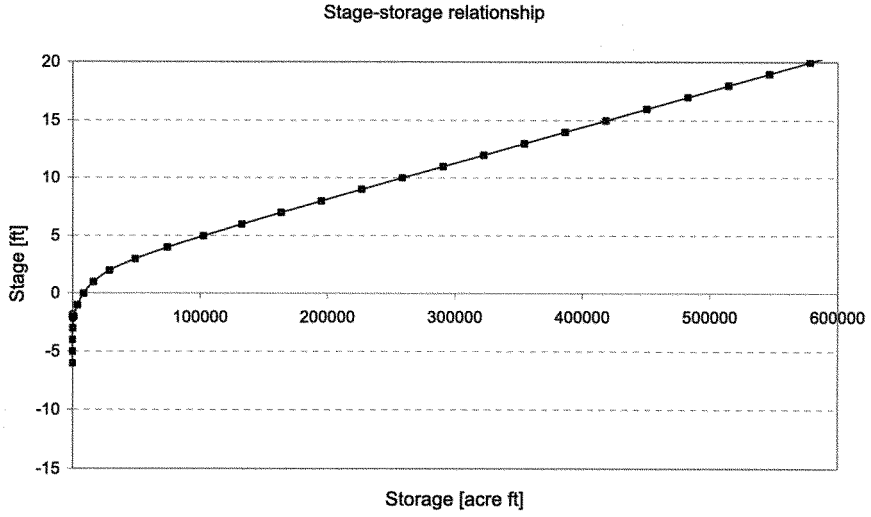


Figure B.54 – Stage storage relationship Lockport

Larose to Golden Meadow

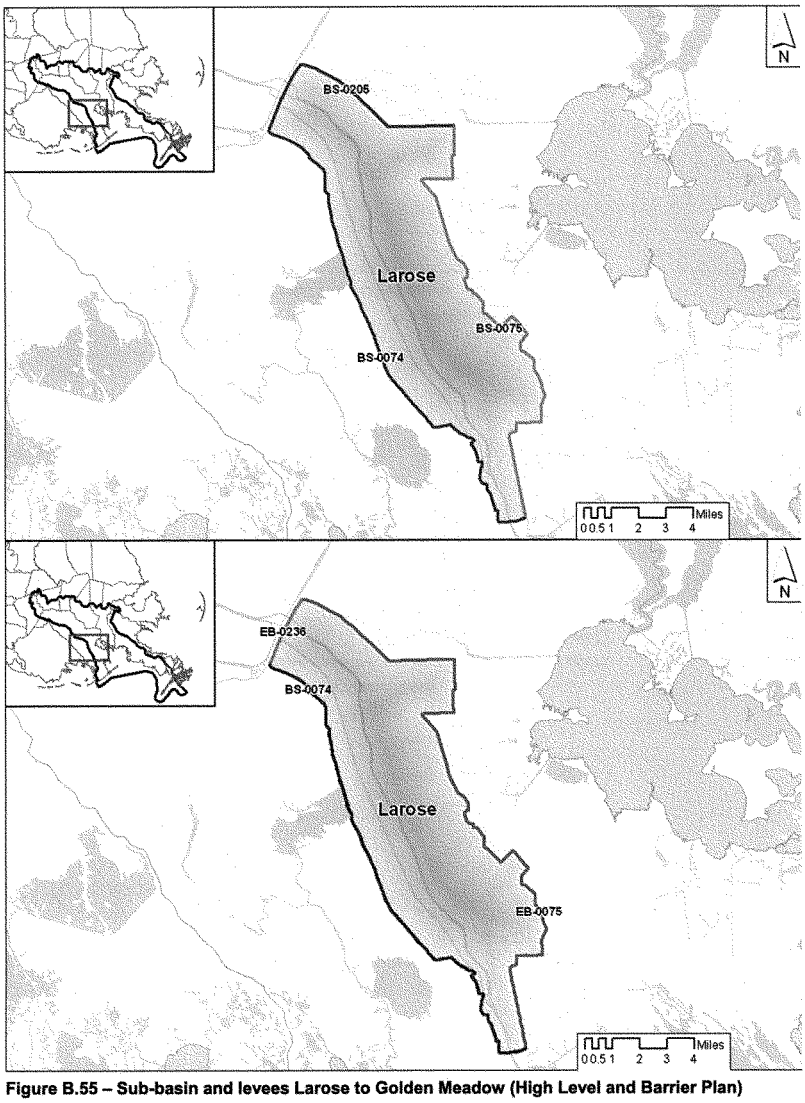


Figure B.55 – Sub-basin and levees Larose to Golden Meadow (High Level and Barrier Plan)

Table B-28 – Levee characteristics Larose to Golden Meadow

High Level Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
BS-0074	19.2	12	18	25	28.5	15.5	21.5	25
BS-0075	19.6	12	17	21.5	23.5	16	20.5	22.5
BS-0205	9.2	12	12	14	16.5	11	14	16.5

Barrier Plan			Design Height (waves modeled without friction)			Design Height (waves modeled with friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year	100 year	400 year	1000 year
[-]	[miles]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]
EB-0074	19.8	12	18.5	26.5	30.5	15.5	22	25.5
EB-0075	26.1	12	17.5	23	26	16.5	22	24.5
EB-0236	2.7	12	17.5	23	26	16.5	22	24.5

	Existing	Proposed	Notes
Pump Capacity (cfs)	0	0	

Storage Area	Type	Notes
Larose	Single area	This is a ring levee system with a varying level of levee because of a rapidly changing elevation of surge. Max surge is the higher of surge (75), authorized (13.5') or design height (75), but capped at 15' as the damages don't increase beyond this point. (see additional note below)

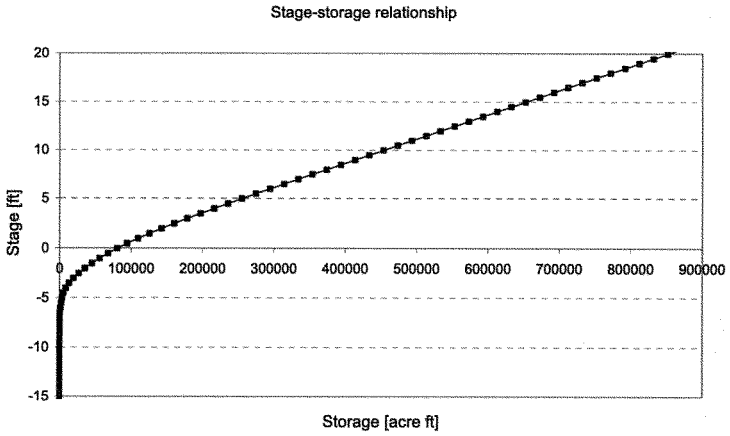


Figure B.56 – Stage storage relationship Larose to Golden Meadow

Additional Notes

Two authorized heights have been used, 13.5' for the southern end of the ring and 10' for the northern end. A review of the stage damage curves has suggested that damages tail off dramatically at 15'.

The Larose to Golden Meadow area is considered as a single interior stage storage area with the potential for overtopping from both the east and west. The western side falls within Planning Unit 3a, which has not been fully investigated yet. If the results from the EB model grid are used for the west it results in higher design heights than with the base case. This is because of the effects of the closure across the Morganza region in the EB model. As the alternatives for this area have yet to be evaluated it has been decided to use the values from the base grid for both the without and with GIWW weir alternatives so the rates of overtopping are constant into the Larose areas from the west. (i.e. BS-0074 is to be used for all alternatives in Planning Unit 2.

As there will be no 100 year level of protection levee system in place before 2010 the base case internal stage frequency should be based on the existing authorized heights rather than the 100 year design heights. For this study three sections are used, BS-0074, BS-0075 and BS-0205. Using the original study reports the authorized heights vary from 13.5' in the south to 10' in the north. For analysis the 13.5' levels have been used for points BS-0074 and BS-0075 whilst 10' has been used for BS-0205.

Planning Unit 3a

Polder Ring: Morganza_no_ret_ring

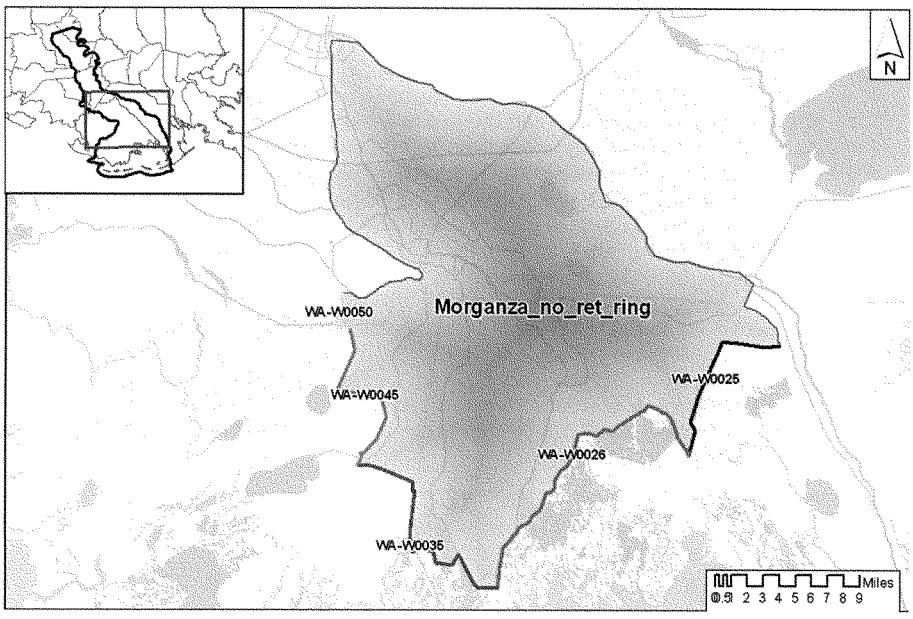


Figure B.57 – Sub-basin and levees Morganza_no_ret_ring

Table B-29 – Levee characteristics Morganza_no_ret_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0050	17395	n.a.	12	19.5	23.5
WA-W0045	63223	n.a.	15	24	29
WA-W0035	83975	n.a.	23	31	35.5
WA-W0026	110857	n.a.	28	36.5	41
WA-W0025	57759	n.a.	22	30.5	35.5

Storage Area	pumping rate [cfs]	Type	Notes
Morganza_no_ret_ring	36581	Single area	

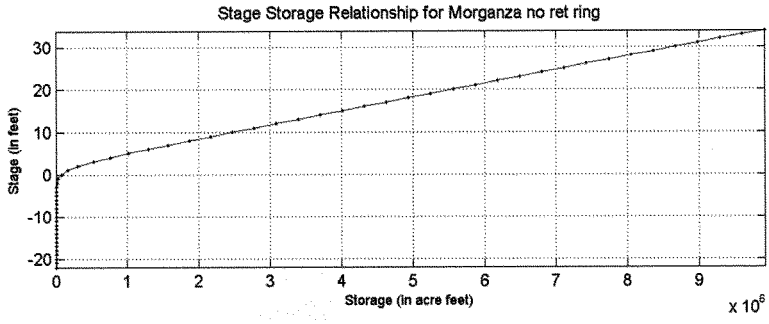


Figure B.58 – Stage storage relationship for Morganza no ret ring

Polder Ring: East_of_Morgan_City_ring

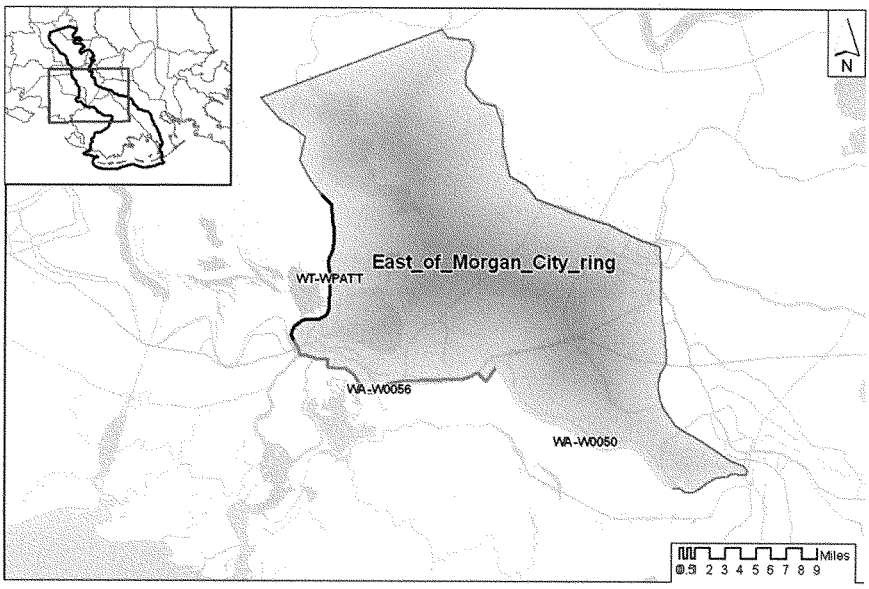


Figure B.59 – Sub-basin and levees East_of_Morgan_City_ring

Table B-30 – Levee characteristics East of Morgan_City_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0056	86276	n.a.	16.5	23	27
WA-W0050	78062	n.a.	12	19.5	23.5
WT-WPATT		n.a.	13.5	20	24

Storage Area	**pumping rate [cfs]**	**Type**	**Notes**
East_of_Morgan_City_ring	400	Single area	

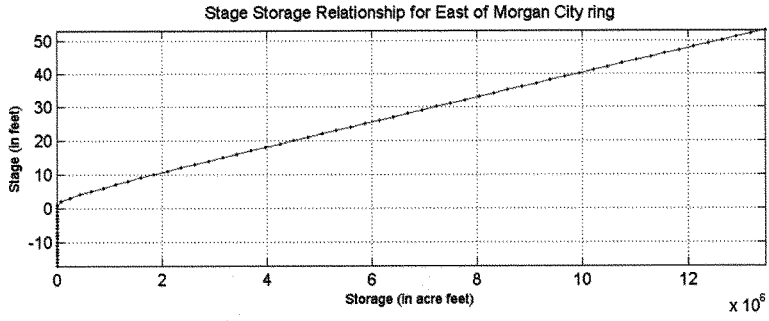


Figure B.60 – Stage storage relationship for East_of_Morgan_City_ring

Polder Ring: Morganza_with_ret_ring

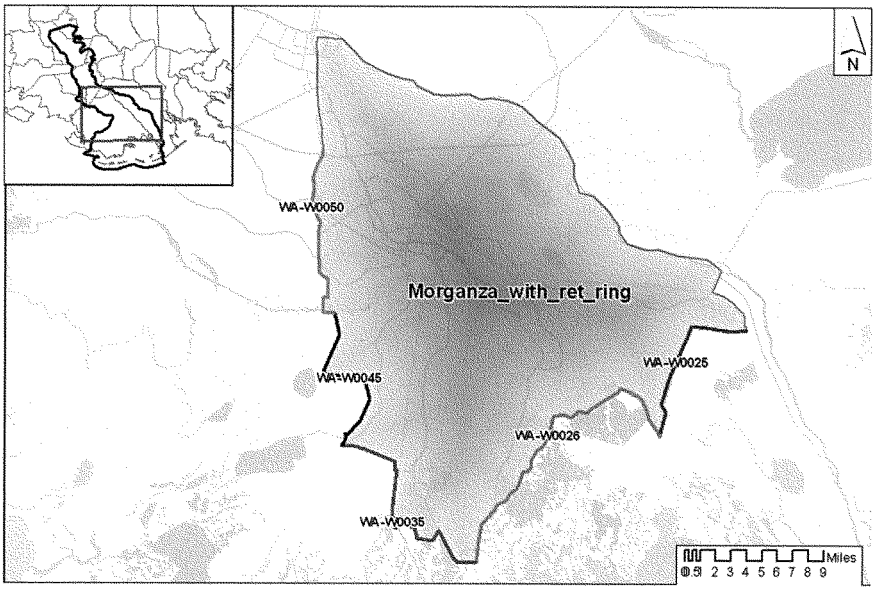


Figure B.61 – Sub-basin and levees Morganza_with_ret_ring

Table B-31 – Levee characteristics Morganza_with_ret_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0050	43663	n.a.	12	19.5	23.5
WA-W0050	21006	n.a.	12	19.5	23.5
WA-W0045	63223	n.a.	15	24	29
WA-W0035	83975	n.a.	23	31	35.5
WA-W0026	110857	n.a.	28	36.5	41
WA-W0025	57759	n.a.	22	30.5	35.5

Storage Area	**Pumping rate [cfs]**	**Type**	**Notes**
Morganza_with_ret_ring	55151	Single area	

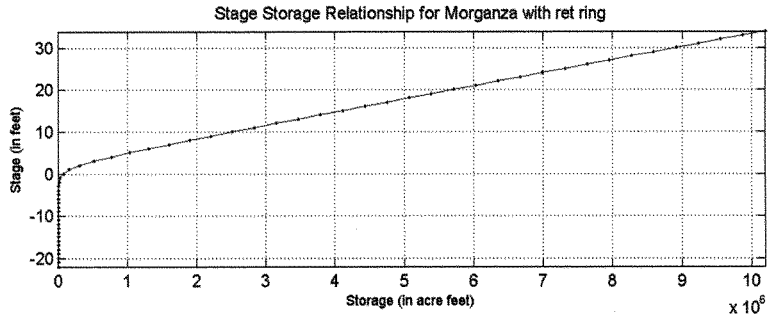


Figure B.62 – Stage storage relationship for Morganza with ret ring

Polder Ring: Morganza_with_ret_ring_m_only

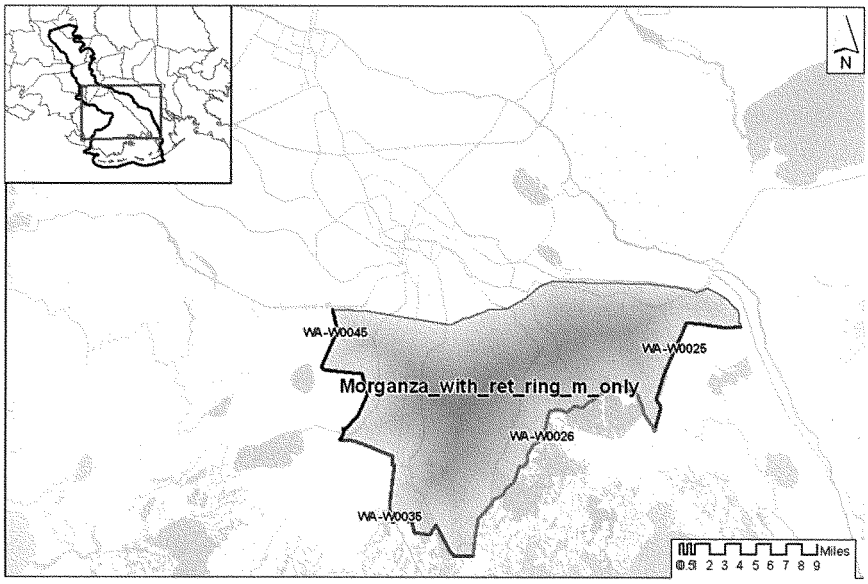


Figure B.63 – Sub-basin and levees Morganza_with_ret_ring_m_only

Table B-32 – Levee characteristics Morganza_with_ret_ring_m_only

			Design Height (waves modeled without friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WA-W0045	63223	n.a.	15	24	29
WA-W0035	83975	n.a.	23	31	35.5
WA-W0026	110857	n.a.	28	36.5	41
WA-W0025	57759	n.a.	22	30.5	35.5

Storage Area	Pumping rate [cfs]	Type	Notes
Morganza_with_ret_ring_m_only	2042	Single area	

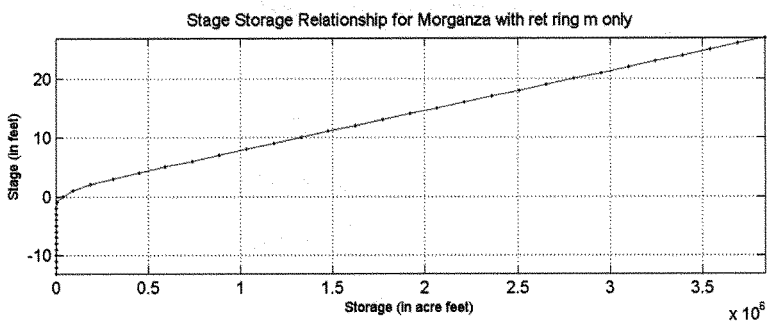


Figure B.64 – Stage storage relationship for Morganza_with_ret_ring_m_only

Polder Ring: Morganza_back_levee

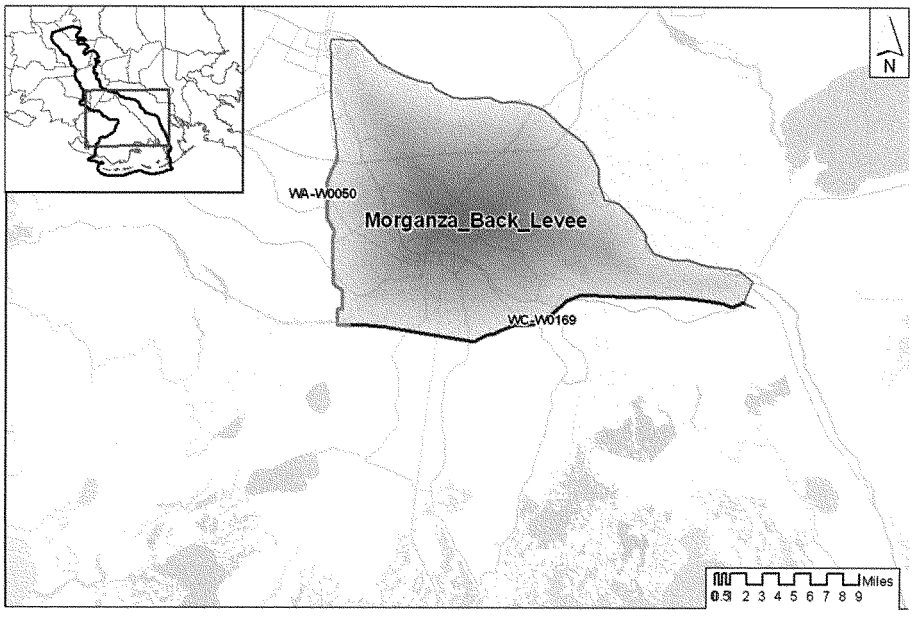


Figure B.65 – Sub-basin and levees Morganza_back_levee

Table B-33 – Levee characteristics Morganza_back_levee

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0050	43663	n.a.	12	19.5	23.5
WA-W0050	21006	n.a.	12	19.5	23.5
WC-W0169	154153	n.a.	-	9.5	24

Storage Area	**Pumping rate [cfs]**	**Type**	**Notes**
Morganza_back_levee	53109	Single area	**Stages behind the secondary defense (Morganza back levee) are calculated based upon overtopping from the flooded polder Morganza. An exception is made for the 2000 year event where stages are equal to the base surge conditions**

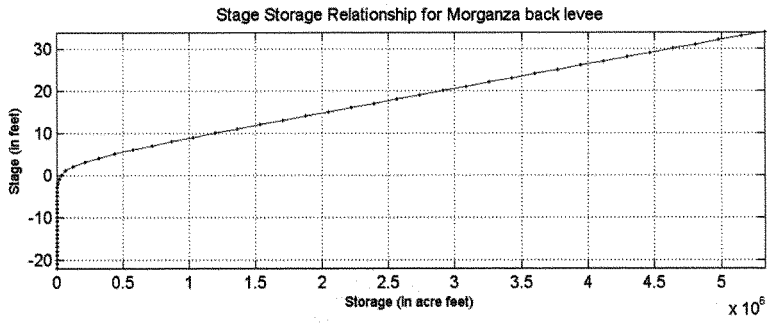


Figure B.66 – Stage storage relationship for Morganza_back_levee

Morgan_City

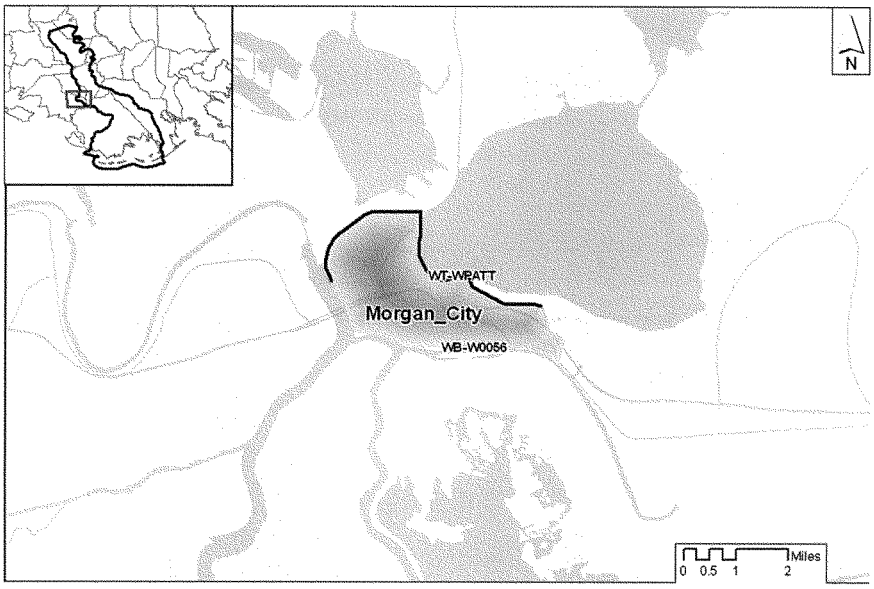


Figure B.67 – Sub-basin and levees Morgan_City_ring

Table B-34 – Levee characteristics Morgan_City_ring

			Design Height (waves modeled without friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WB-W0056	33243	n.a.	18	25	28.5
WT-WPATT		n.a.	13.5	20	24

Storage Area	Pumping Rate [cfs]	Type	Notes
Morgan_City_ring	400	Single area	For small ring levees overtopping is assumed only to occur from the sea side levee. In this case that is levee 56

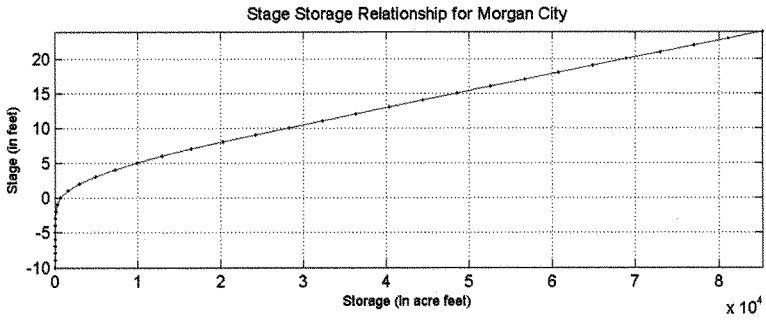


Figure B.68 – Stage storage relationship for Morgan City

Planning Unit 3b

Polder Ring: South_of_Franklin_ring

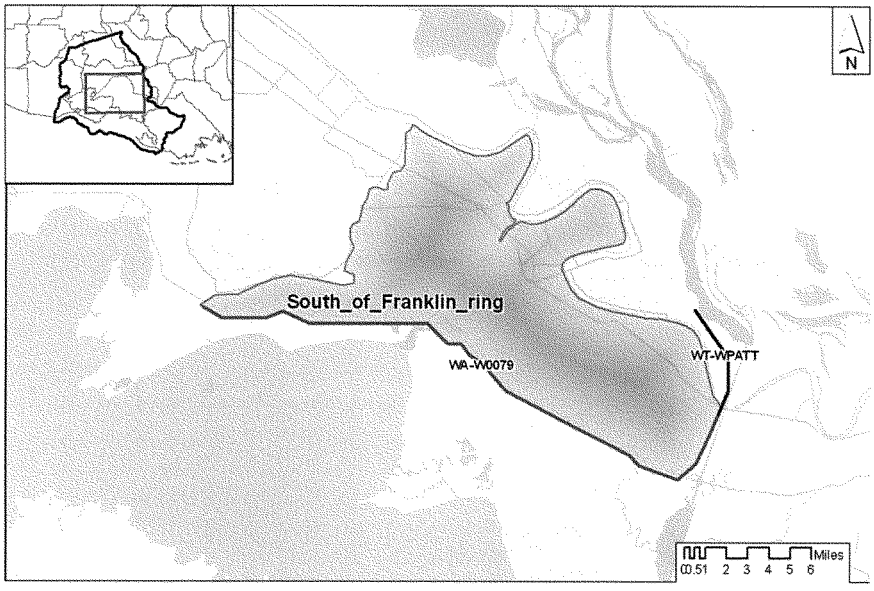


Figure B.69 – Sub-basin and levees South_of_Franklin_ring

Table B-35 – Levee characteristics South_of_Franklin_ring

			Design Height (waves modeled without friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WA-W0079	151872	n.a.	20.5	28	32.5
WT-WPATT		n.a.	13.5	20	24

Storage Area	Pumping Rate [cfs]	Type	Notes
South_of_Franklin_ring	311	Single area	

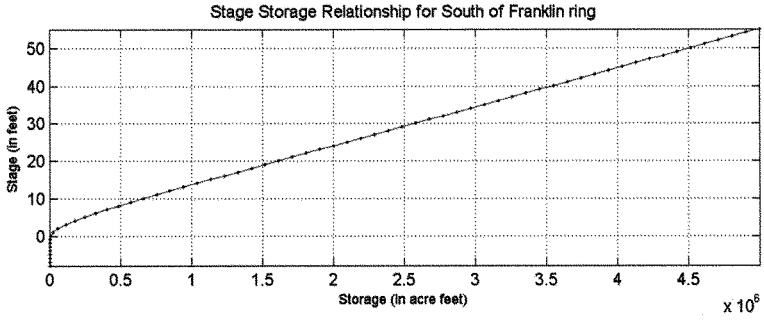


Figure B.70 – Stage storage relationship for South_of_Franklin_ring

Polder Ring: GIWW_PU3b_ring

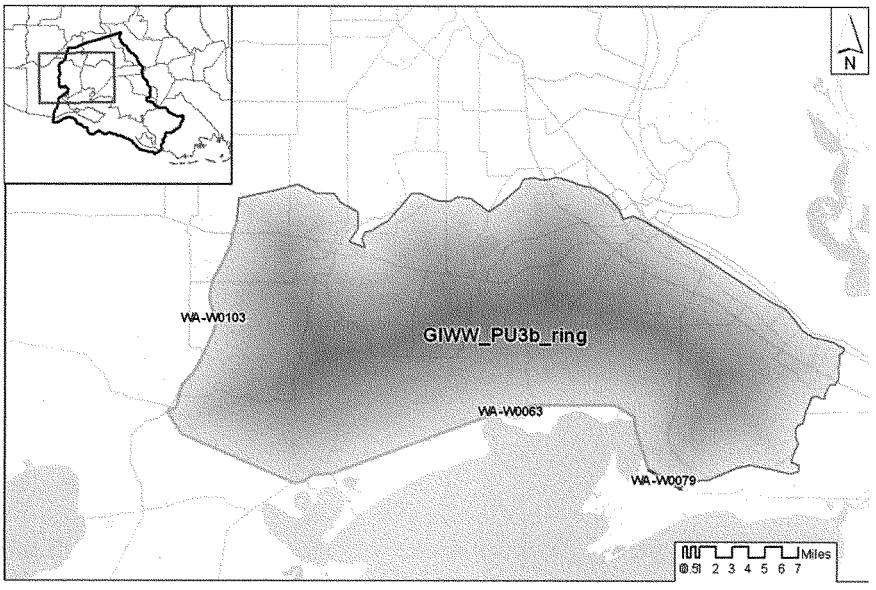


Figure B.71 – Sub-basin and levees GIWW_PU3b_ring

Table B-36 – Levee characteristics GIWW_PU3b_ring

			Design Height (waves modeled without friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WA-W0103	48316	n.a.	21.5	28	31.5
WA-W0063	131984	n.a.	20	27.5	32.5
WA-W0079	12102	n.a.	20.5	28	32.5

Storage Area	Pumping Rate [cfs]	Type	Notes
GIWW_PU3b_ring	19046	Single area	

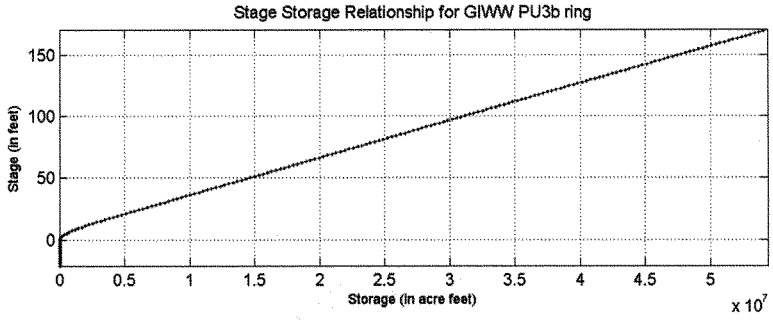


Figure B.72 – Stage storage relationship for GIWW_PU3b_ring

Polder Ring: Patterson

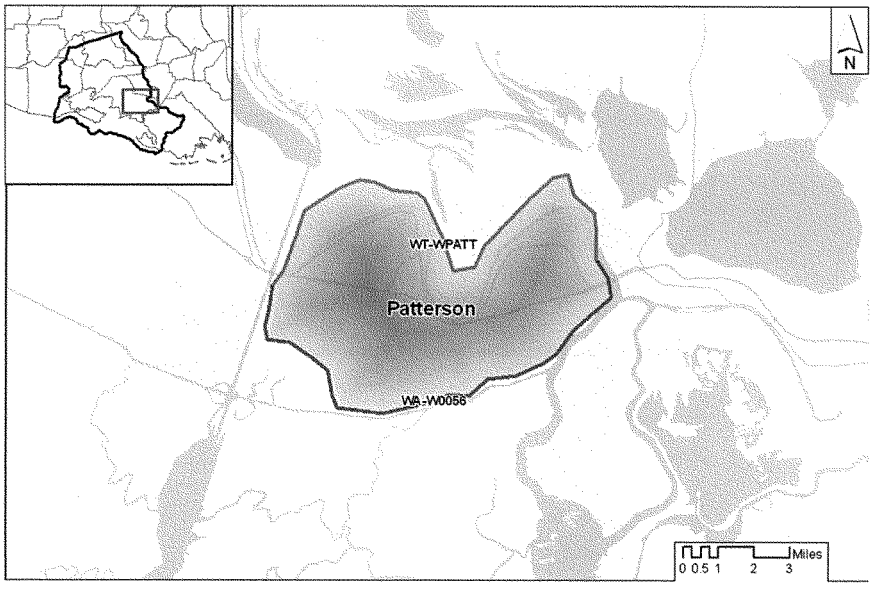


Figure B.73 – Sub-basin and levees Patterson

Table B-37 – Levee characteristics Patterson

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WB-W0056	77285	n.a.	18	25	28.5
WT-WPATT		n.a.	13.5	20	24

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Patterson	29853	Single area	For the Patterson ring levee overtopping is assumed to only occur over the levee facing the sea.

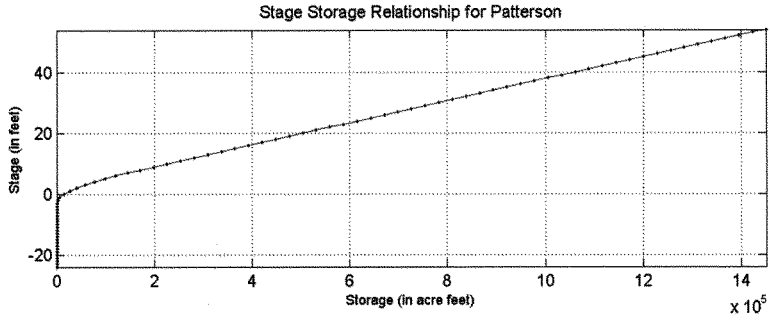


Figure B.74 – Stage storage relationship for Patterson

Polder Ring: Abbeville_to_Delcambre_ring

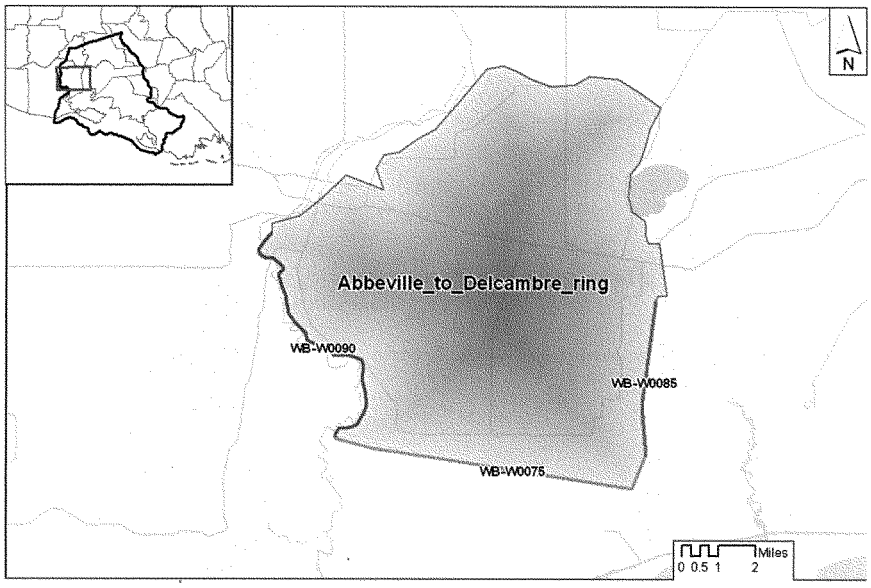


Figure B.75 – Sub-basin and levees Abbeville_to_Delcambre_ring

Table B-38 – Levee characteristics Abbeville_to_Delcambre_ring

			Design Height (waves modeled without friction)		
Reach ID	Length	Authorized Height	100 year	400 year	1000 year
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WB-W0090	42402	n.a.	18	24.5	28.5
WB-W0075	48469	n.a.	20	27	31
WB-W0085	22333	n.a.	17	26	31.5

Storage Area	Pumping Rate [cfs]	Type	Notes
Abbeville_to_Delcambre_ring	27617	Single area	

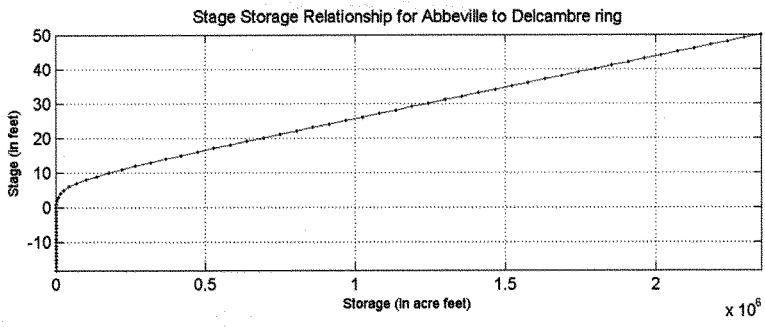


Figure B.76 – Stage storage relationship for Abeville_to_Delcambre_ring

Polder Ring: New_Iberia_ring

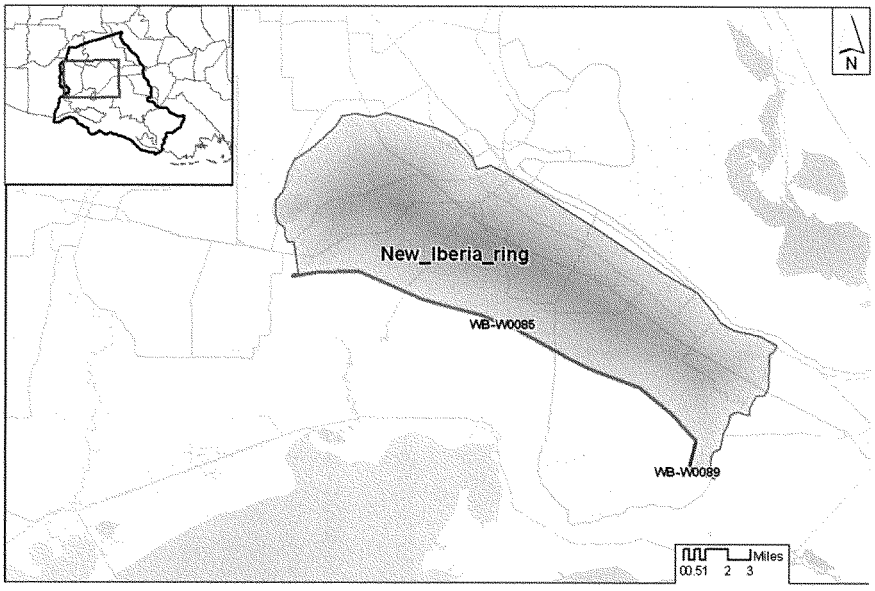


Figure B.77 – Sub-basin and levees New_Iberia_ring

Table B-39 – Levee characteristics New_Iberia_rign

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WB-W0085	106523	n.a.	17	26	31.5
WB-W0089	13927	n.a.	19	26.5	30.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
New_Iberia_ring	0	Single area	For large storage areas pumping capacities are set to zero as it is assumed that enough "natural storage" is available inside these drainage basins.

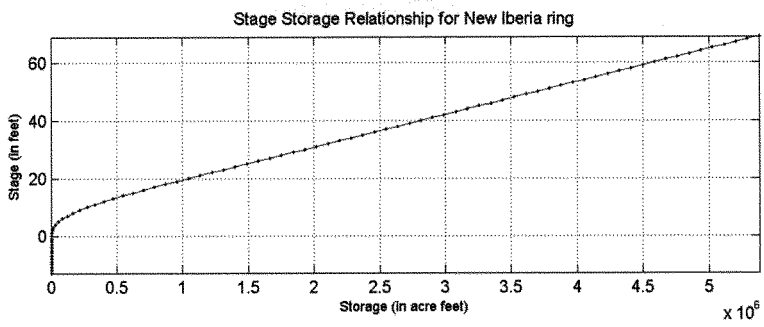


Figure B.78 – Stage storage relationship for New_Iberia_ring

Polder Ring: Charenton_ring

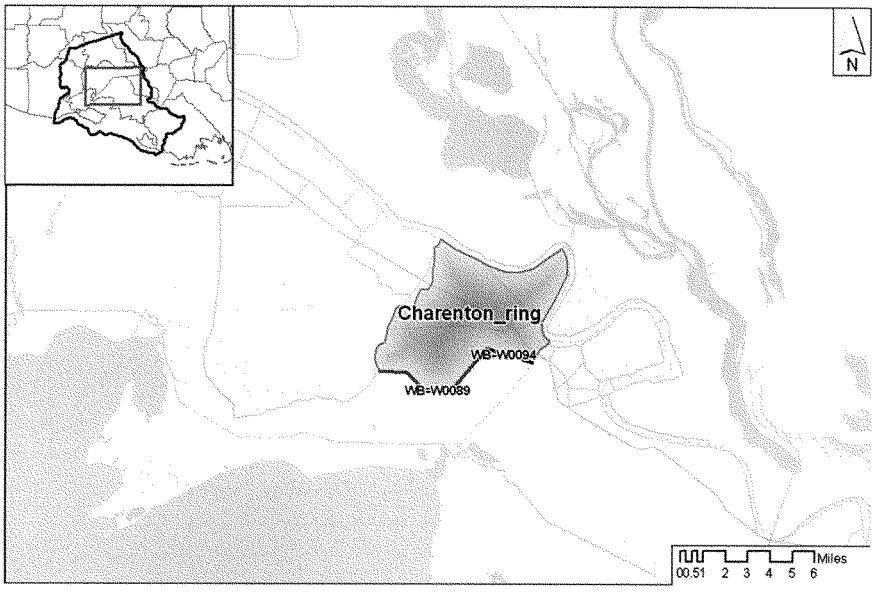


Figure B.79 – Sub-basin and levees Charenton_ring

Table B-40 – Levee characteristics Chareton_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WB-W0089	31581	n.a.	19	26.5	30.5
WB-W0094	11259	n.a.	16.5	25	30

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Charenton_ring	10472	Single area	

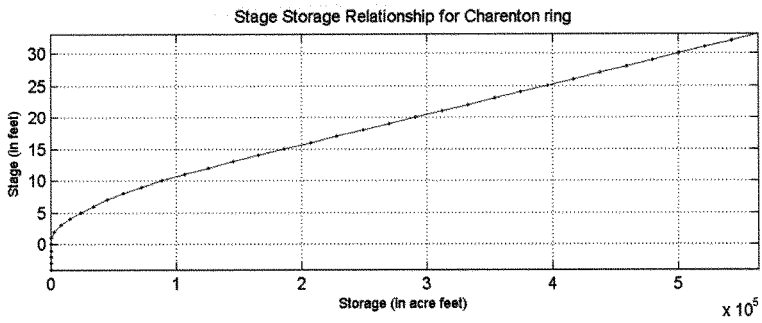


Figure B.80 – Stage storage relationship for Charenton_ring

Polder Ring: Abbeville

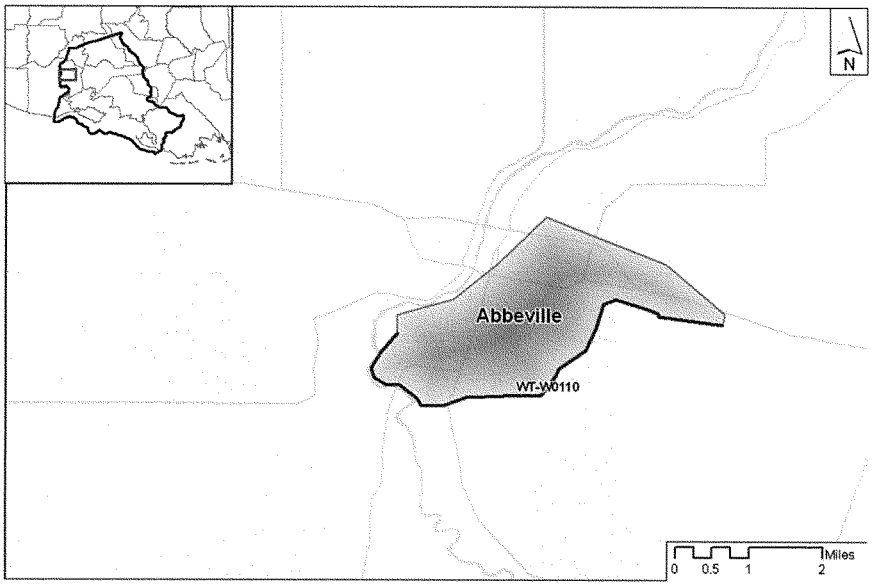


Figure B.81 – Sub-basin and levees Abbeville

Table B-41 – Levee characteristics Abbeville

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0110	33797	n.a.	12.5	20.5	25

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Abbeville	1836	Single area	

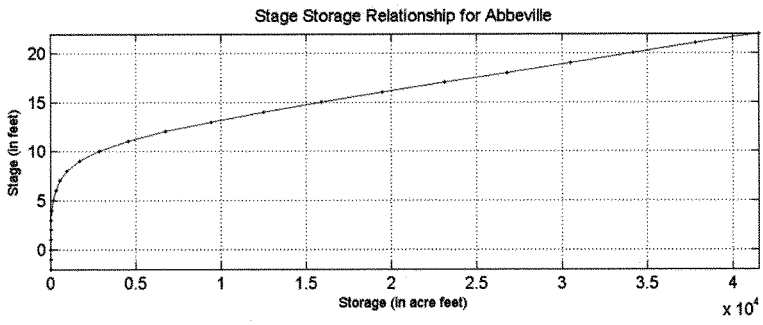


Figure B.82 – Stage storage relationship for Abbeville

Polder Ring: Erath

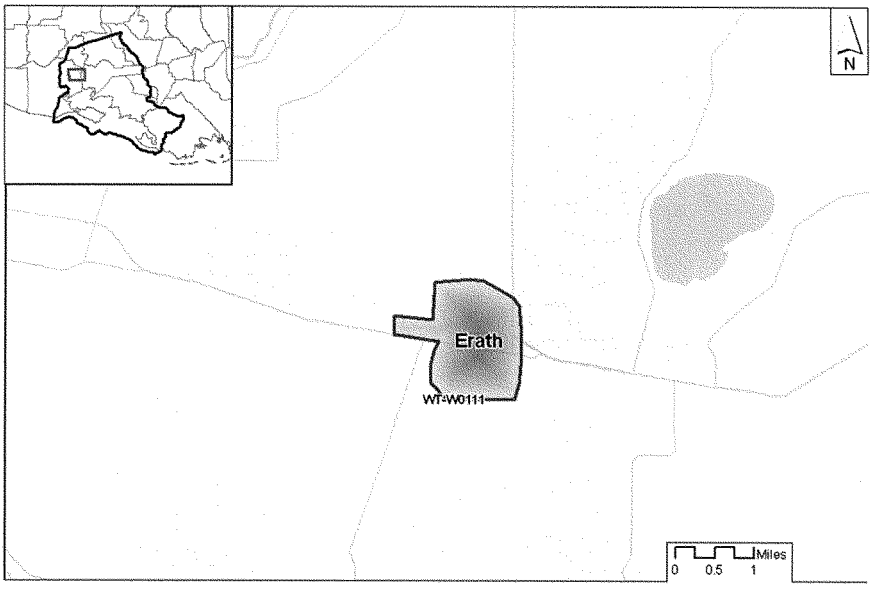


Figure B.83 –Sub-basin and levees Erath

Table B-42 – Levee characteristics Erath

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0111	19667	n.a.	17.5	25	29

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Erath	553	Single area	Erath is amongst the ring levees for which overtopping is only included for the seaward side of the levee

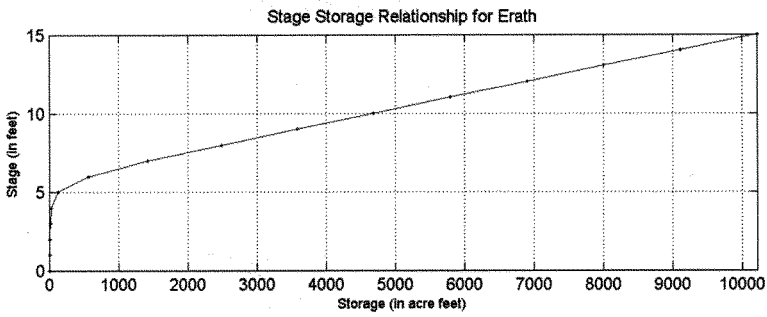


Figure B.84 – Stage storage relationship for Erath

Polder Ring: Delcambre

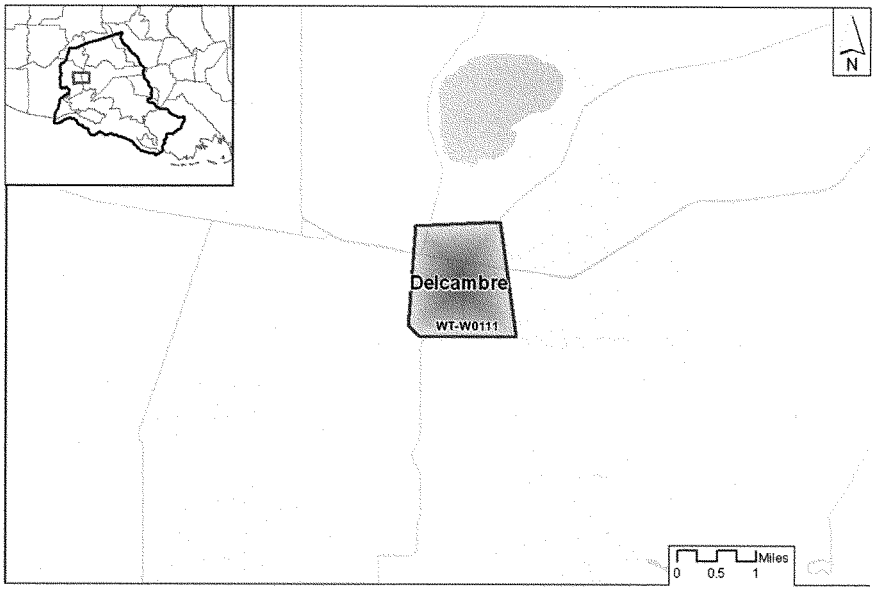


Figure B.85 – Sub-basin and levees Delcambre

Table B-43 – Levee characteristics Delcambre

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	[ft]	[ft]	[ft]	[ft]	[ft]
WT-W0111	21979	n.a.	17.5	25	29

Storage Area	Pumping Rate [cfs]	Type	Notes
Delcambre	555	Single area	

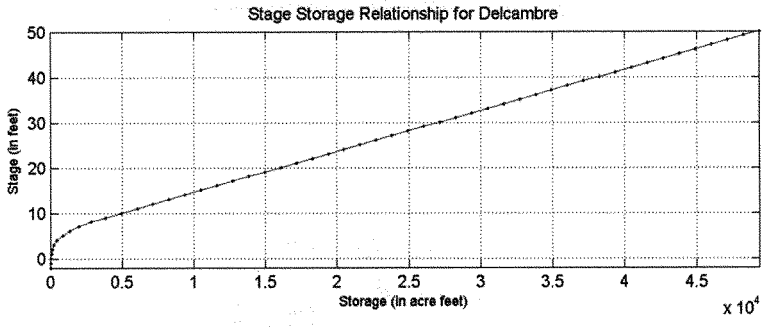


Figure B.86 – Stage storage relationship for Delcambre

Polder Ring: New_Iberia

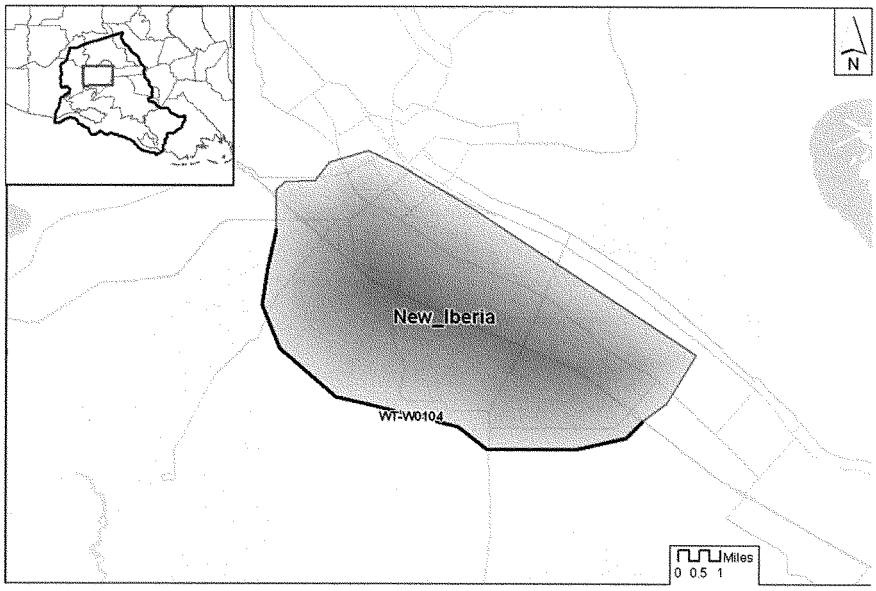


Figure B.87 – Sub-basin and levees New_Iberia

Table B-44 – Levee characteristics New_Iberia

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0104	66021	n.a.	14.5	22	26.5

Storage Area	Pumping Rate [cfs]	Type	Notes
New_Iberia	0	Single area	

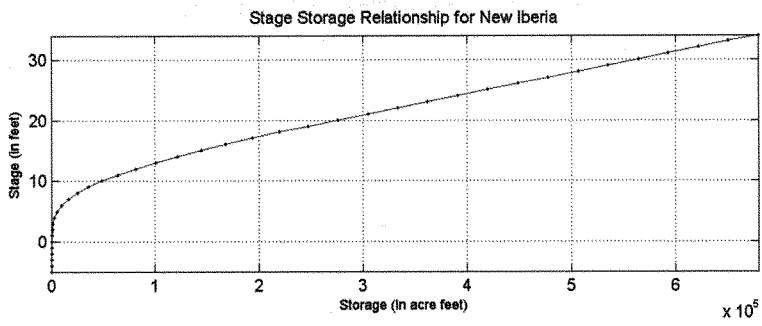


Figure B.88 – Stage storage relationship for New_Iberia

Polder Ring: Baldwin

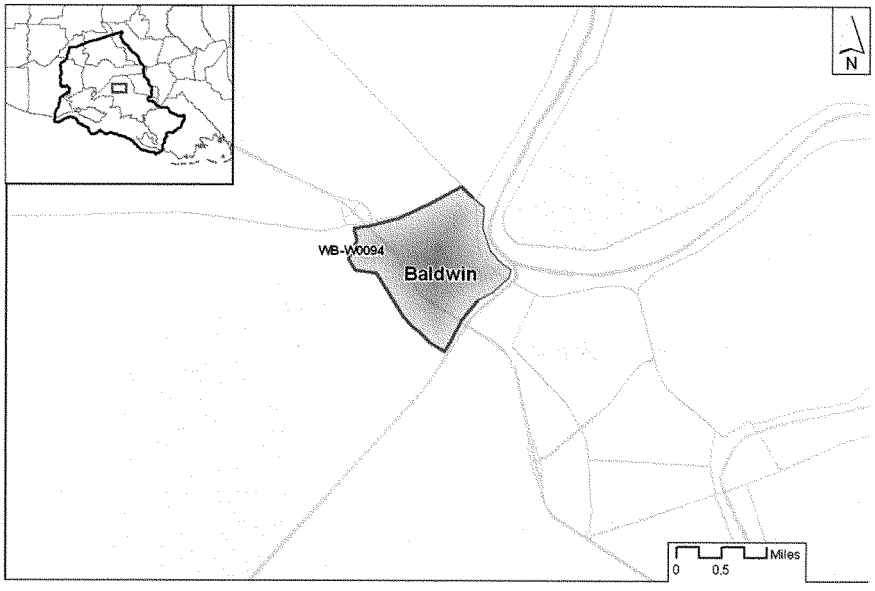


Figure B.89 – Sub-basin and levees Baldwin

Table B-45 – Levee characteristics Baldwin

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WB-W0094	17831	n.a.	16.5	25	30

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Baldwin	554	Single area	

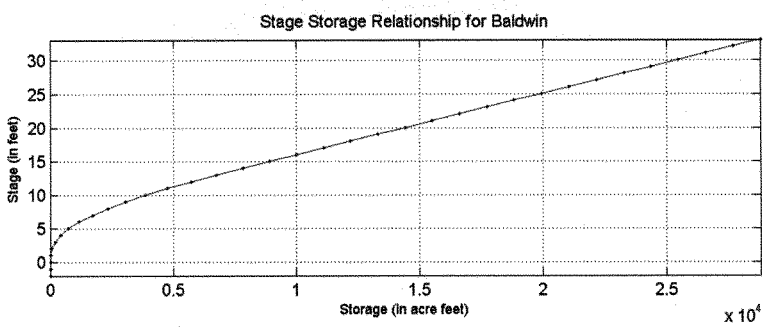


Figure B.90 – Stage storage relationship for Baldwin

Polder Ring: Franklin

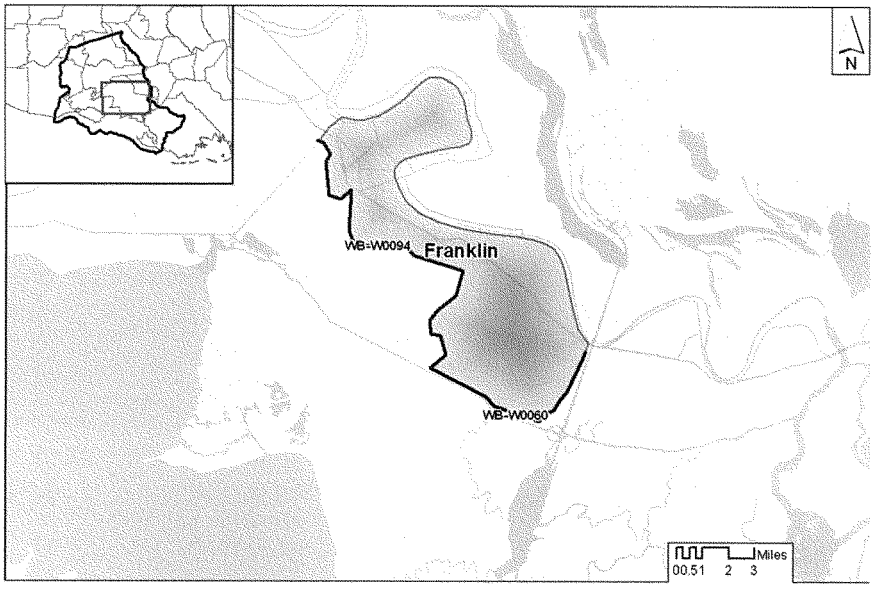


Figure B.91 – Sub-basin and levees Franklin

Table B-46 – Levee characteristics Franklin

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WB-W0094	78444	n.a.	16.5	25	30
WB-W0060	42760	n.a.	16.5	23.5	27.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Franklin	17125	Single area	

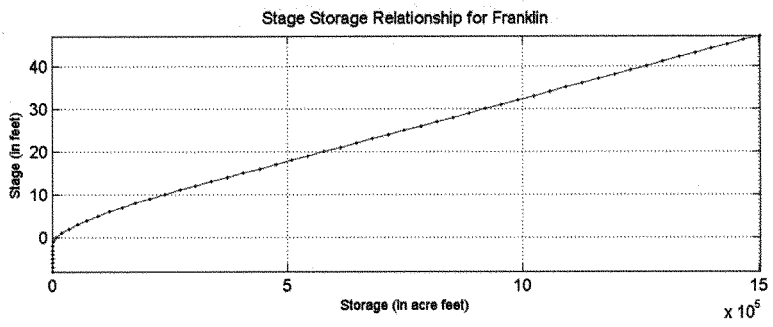


Figure B.92 – Stage storage relationship for Franklin

Planning Unit 4

Polder Ring: Central_PU4_ring

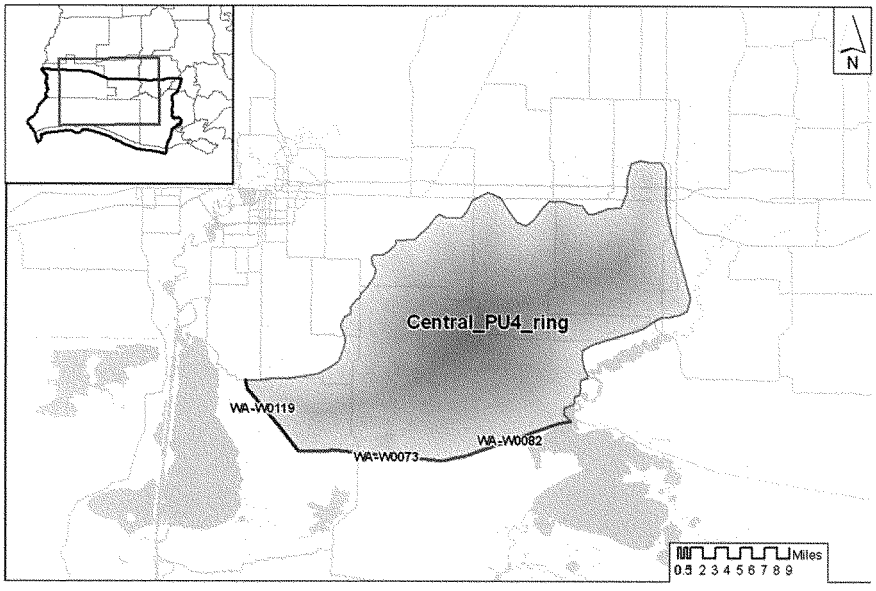


Figure B.93 – Sub-basin and levees Central_PU4_ring

Table B-47 – Levee characteristics Central_PU4_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0119	38332	n.a.	13.5	21.5	26
WA-W0073	62206	n.a.	16	24	28.5
WA-W0082	57523	n.a.	14	20.5	24.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Central_PU4_ring	0	Single area	

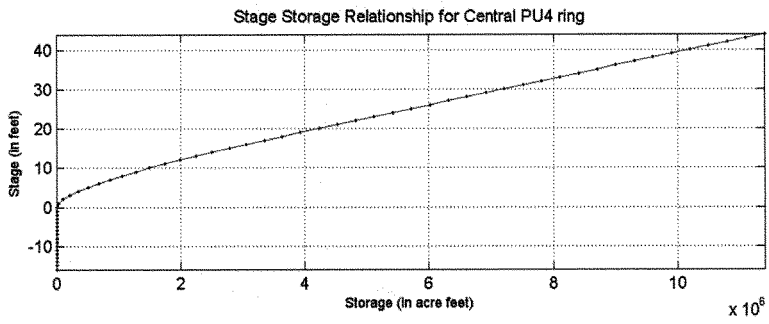


Figure B.94 – Stage storage relationship for Central_PU4_ring

Polder Ring: GIWW_to_Veterans_ring

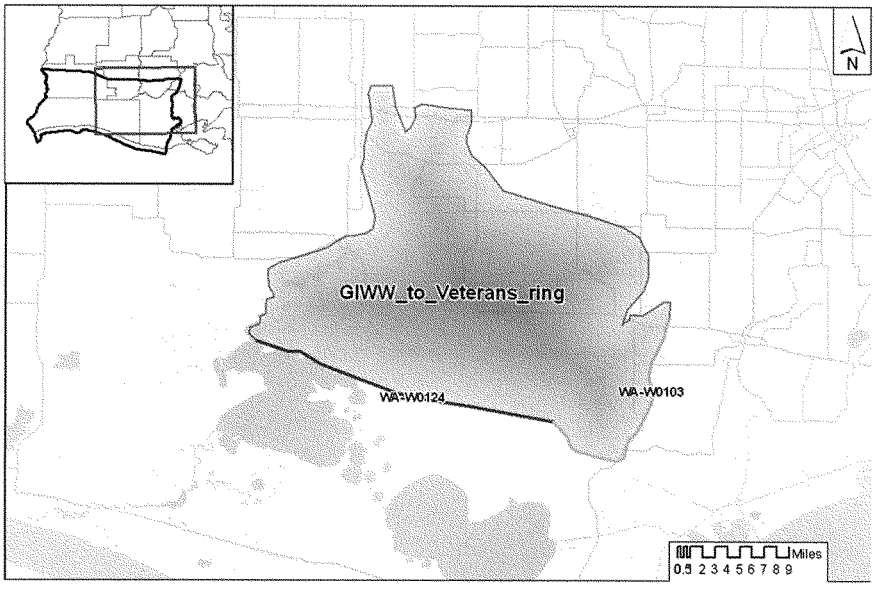


Figure B.95 – Sub-basin and levees GIWW_to_Veterans_ring

Table B-48 – Levee characteristics GIWW_to_Veterans_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0124	132503	n.a.	12	19.5	24
WA-W0103	33981	n.a.	21.5	28	31.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
GIWW_to_Veterans_ring	366	Single area	

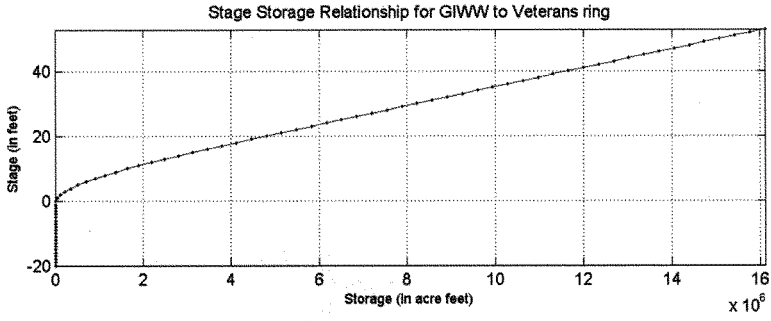


Figure B.96 – Stage storage relationship for GIWW_to_Veterans_ring

Polder Ring: South_of_Lake_Charles_ring

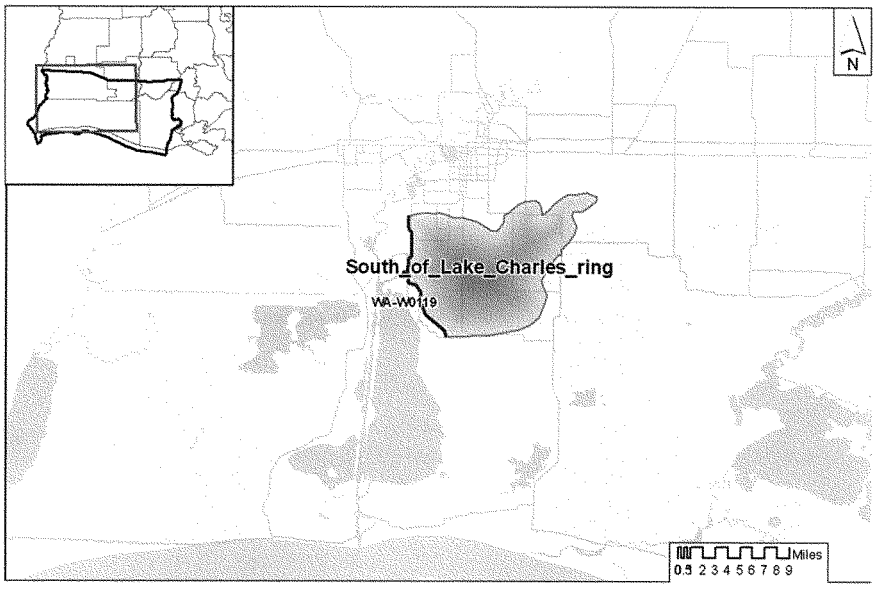


Figure B.97 – Sub-basin and levees South_of_Lake_Charles_ring

Table B-49 – Levee characteristics South_of_Lake_Charles_ring

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0119	62456	n.a.	13.5	21.5	26

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
South_of_Lake_Charles_ring	29761	Single area	

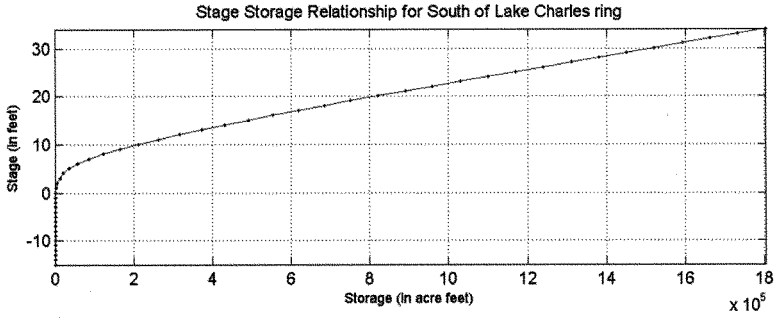


Figure B.98 – Stage storage relationship for South_of_Lake_Charles_ring

Polder Ring: West_Lake_Charles

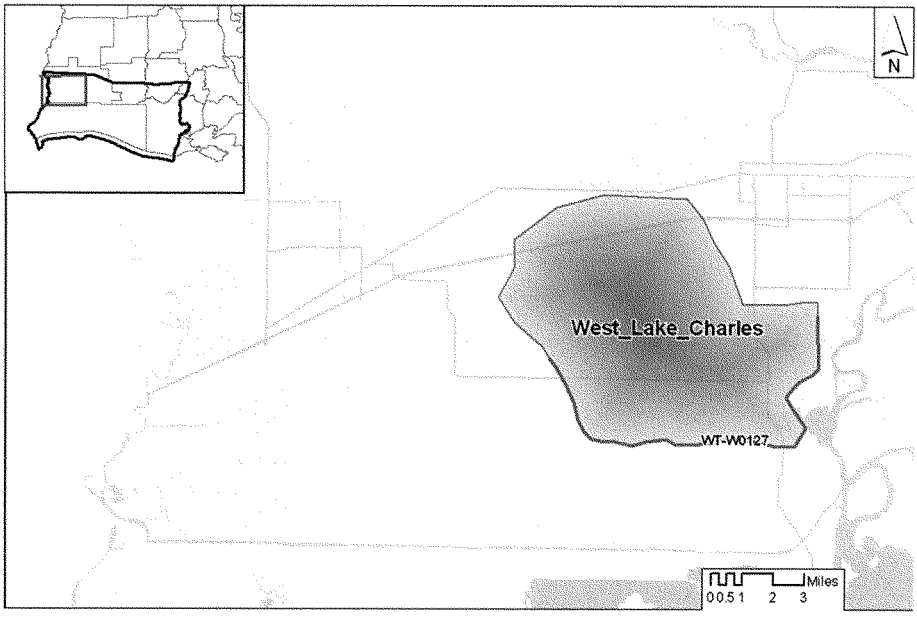


Figure B.99 – Sub-basin and levees West_Lake_Charles

Table B-50 – Levee characteristics West_Lake_Charles

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0127	77095	n.a.	11	15.5	17.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
West_Lake_Charles	29761	Single area	

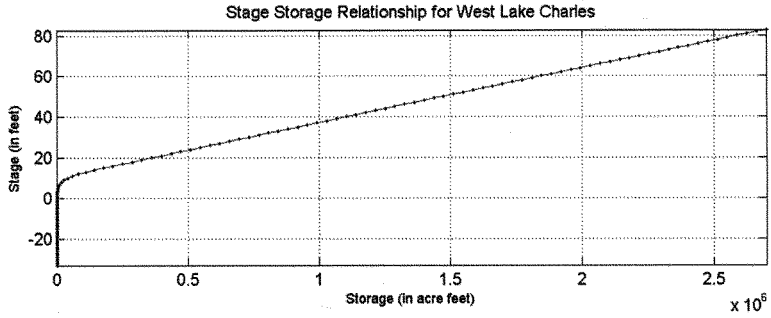


Figure B.100 – Stage storage relationship for West_Lake_Charles

Polder Ring: Prien

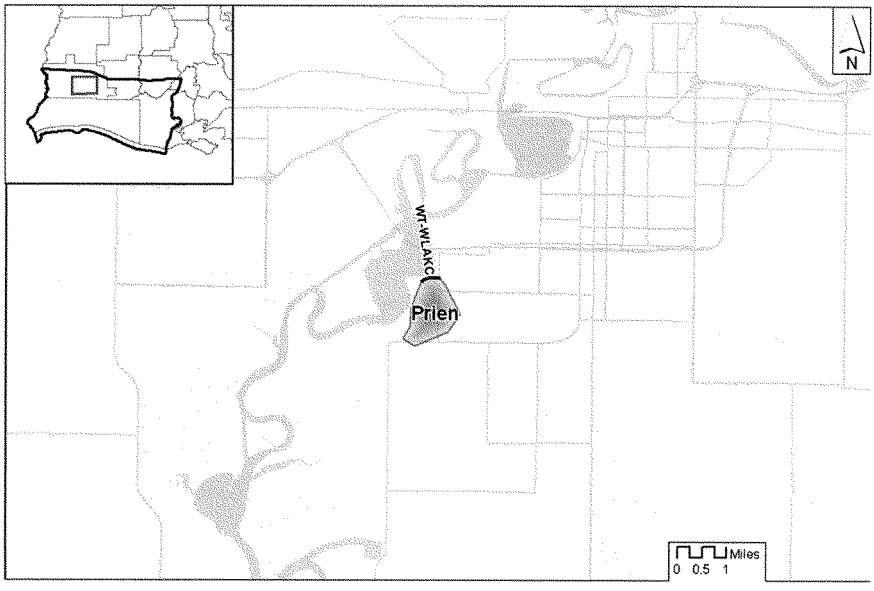


Figure B.101 – Sub-basin and levees Prien

Table B-51 – Levee characteristics Prien

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0999	1918	n.a.	13.5	17	19.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Prien	13927	Single area	The Prien area is more prone to flooding from the river side instead of the sea side while sea surge penetrate up the river. Due to lack of wave data a 3ft wave is applied to come to overtopping rates for this levee.

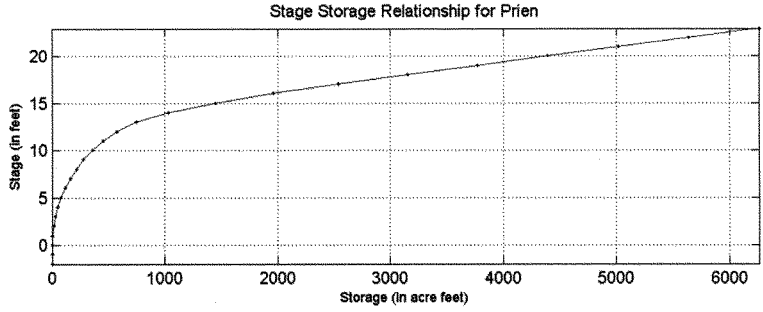


Figure B.102 – Stage storage relationship for Prien

Polder Ring: Inner_Lake_Charles

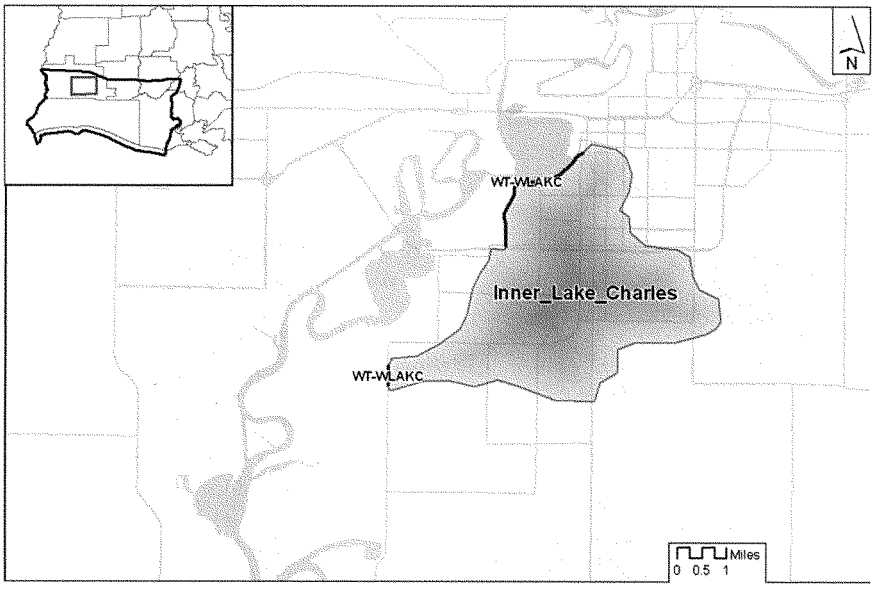


Figure B.103 – Sub-basin and levees Inner_Lake_Charles

Table B-52 – Levee characteristics Inner_Lake_Charles

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0999	2103	n.a.	13.5	17	19.5
WT-W0999	15698	n.a.	13.5	17	19.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Inner_Lake_Charles	13927	Single area	Equally to the Prien area this area is more prone to flooding from the river side instead of the sea side while sea surge penetrates up the river. Due to lack of wave data a 3ft wave is applied to come to overtopping rates for this levee.

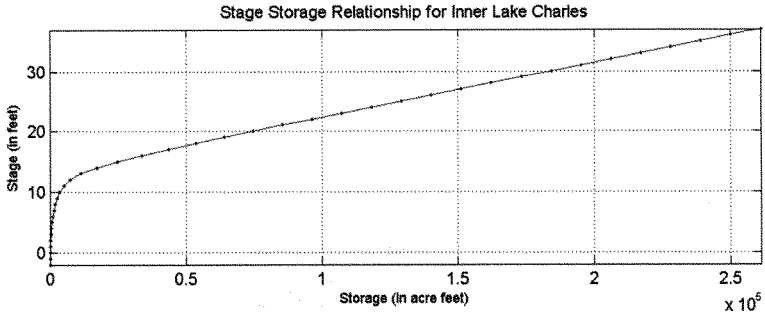


Figure B.104 – Stage storage relationship for Inner_Lake_Charles

Polder Ring: South_of_Lake_Charles_ring_12

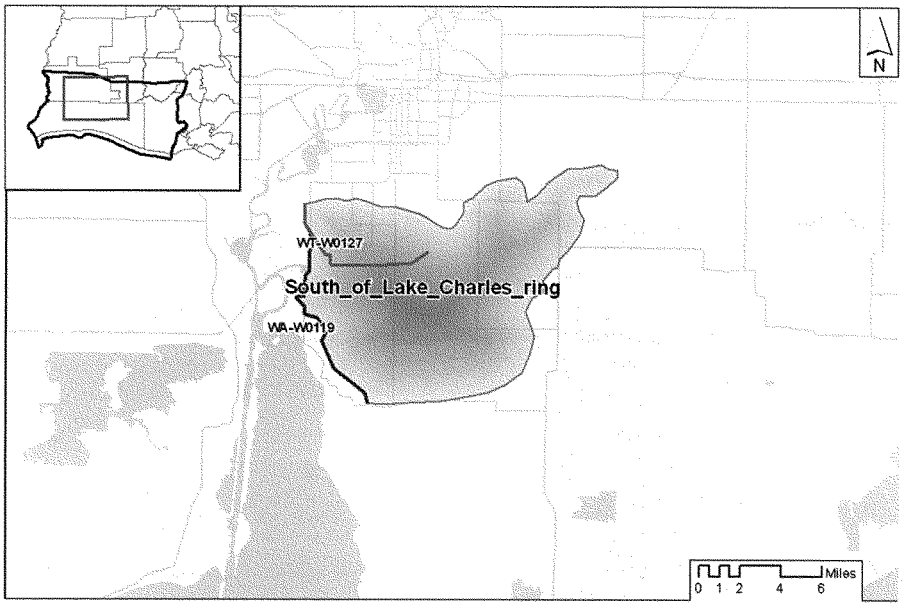


Figure B.105 – Sub-basin and levees South_of_Lake_Charles_ring_12

Table B-53 – Levee characteristics South_of_Lake_Charles_ring_12

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0119	62456	n.a.	12	12	12
WT-W0127	44374	n.a.	-	10.7	12.3

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
South_of_Lake_Charles_ring_12	311	Single area	

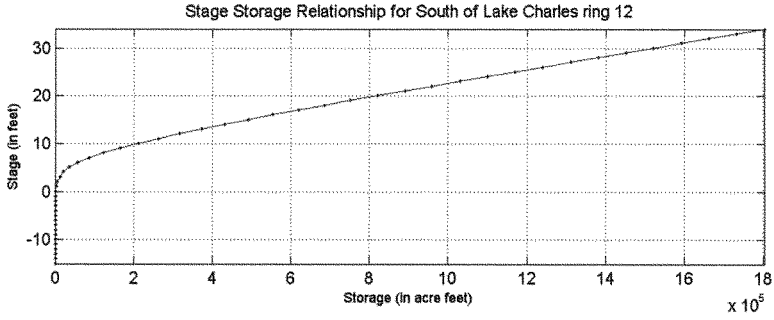


Figure B.106 – Stage storage relationship for South_of_Lake_Charles_ring_12

Polder Ring: Central_PU4_ring_large_12

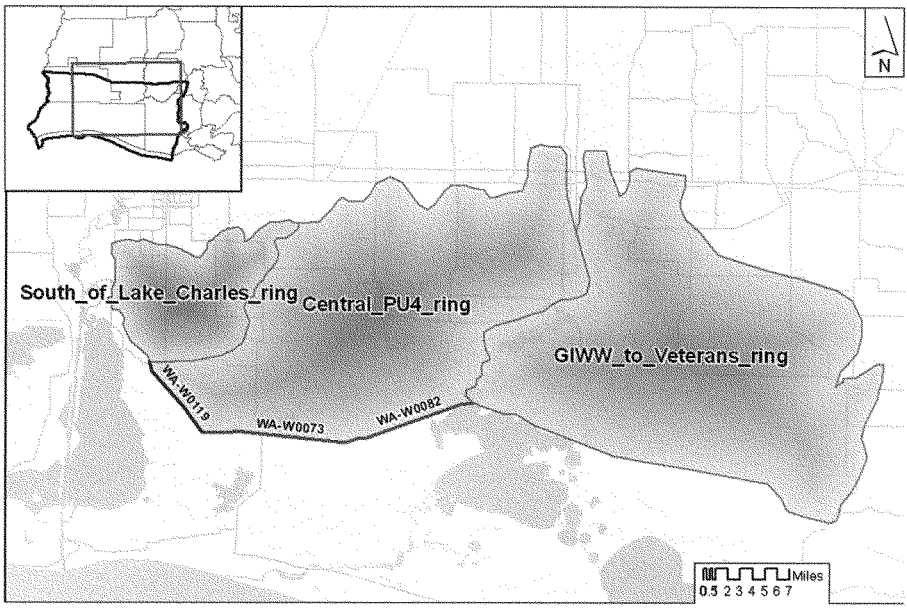


Figure B.107 – Sub-basin and levees Central_PU4_ring_large_12

Table B-54 – Levee characteristics Central_PU4_ring_large_12

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0119	38332	n.a.	12	12	12
WA-W0073	62206	n.a.	12	12	12
WA-W0082	57523	n.a.	12	12	12

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Central_PU4_ring_large_12	29761	multiple areas	For the 12' levee design to be effective areas are combined to let the flooding to spread out over several areas.

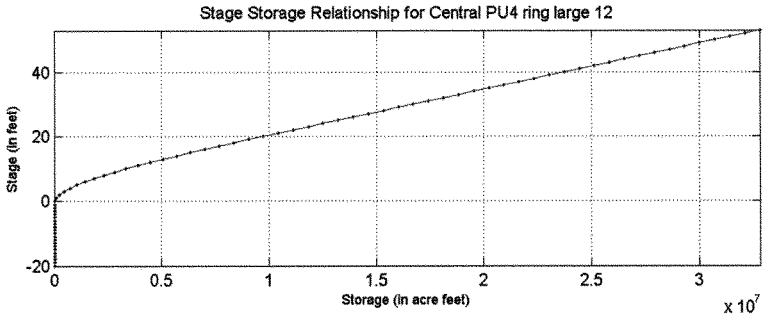


Figure B.108 – Stage storage relationship for Central_PU4_ring_large_12

Polder Ring: GIWW_to_Veterans_ring_large_12

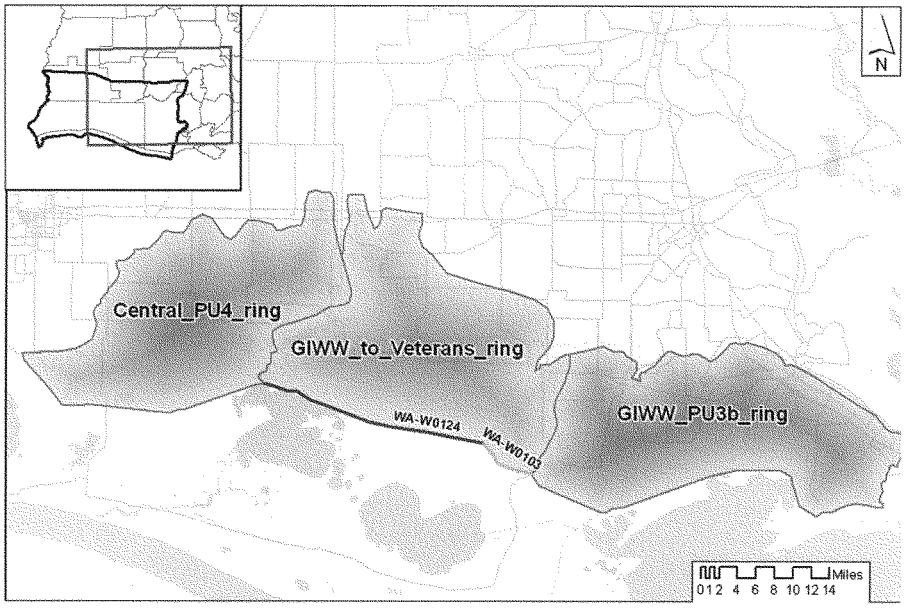


Figure B.109 – Sub-basin and levees GIWW_to_Veterans_ring_large_12

Table B-55 – Levee characteristics GIWW_to_Veterans_ring_large_12

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WA-W0124	132503	n.a.	12	12	12
WA-W0103	33981	n.a.	12	12	12

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
GIWW_to_Veterans_ring_large_12	366	multiple areas	For the 12' levee design to be effective areas are combined to let the flooding to spread out over several areas.

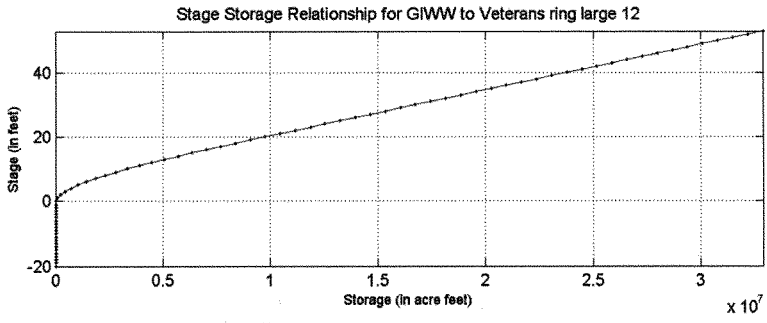


Figure B.110 – Stage storage relationship for GIWW_to_Vetterans_ring_large_12

Polder Ring: East_Lake_Charles

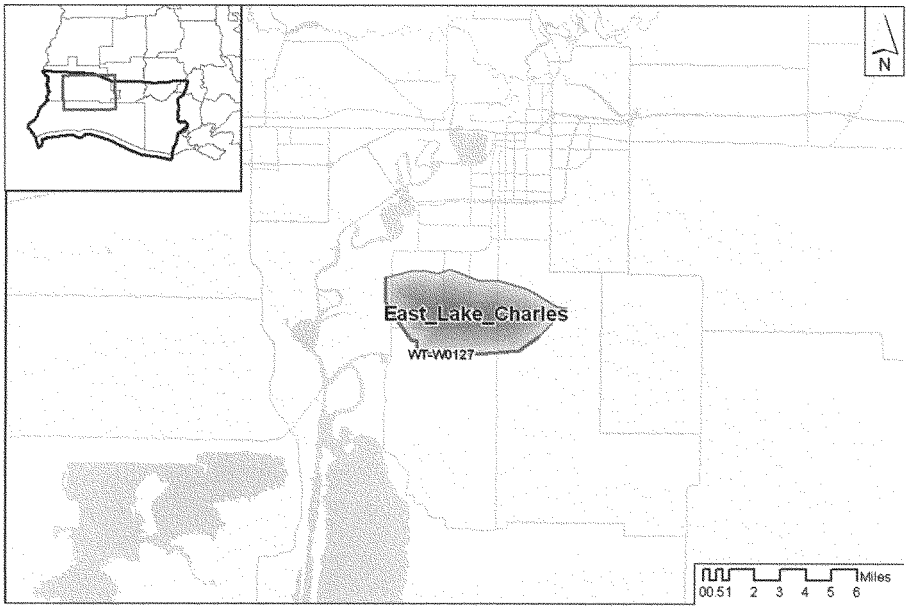


Figure B.111 – Sub-basin and levees East_Lake_Charles

Table B-56 – Levee characteristics East_Lake_Charles

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0127	77095	n.a.	11	15.5	17.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
East_Lake_Charles	5396	single area	

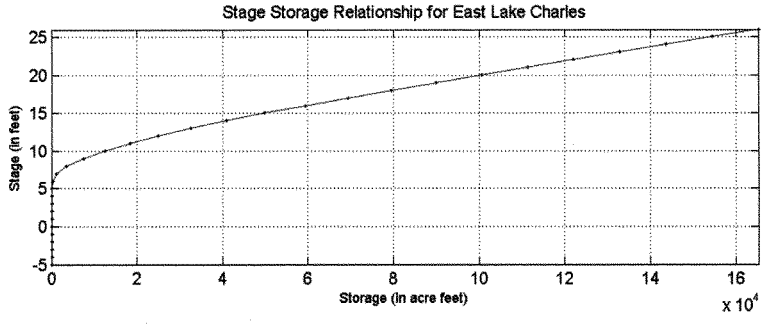


Figure B.112 – Stage storage relationship for East_Lake_Charles

Polder Ring: Gueydan

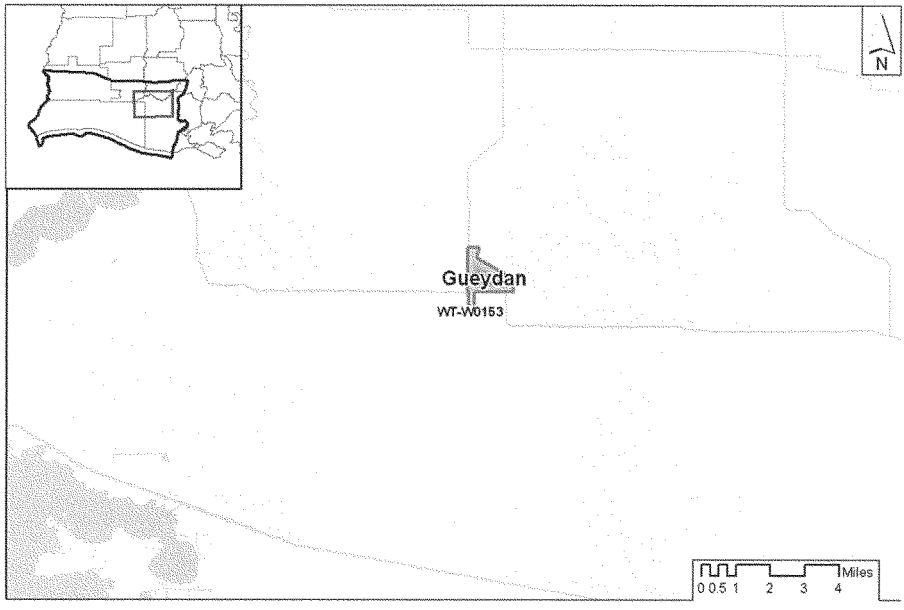


Figure B.113 – Sub-basin and levees Gueydan

Table B-57 – Levee characteristics Gueydan

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0153	21674	n.a.	9	15.5	18.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Gueydan	366	single area	Equally to other smaller ring levees Gueydan is only overtopped from the seawards side

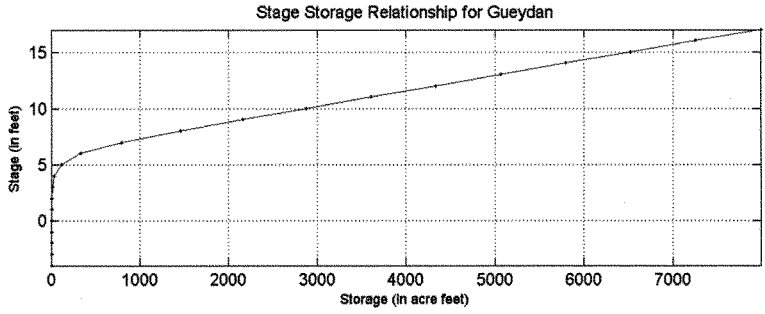


Figure B.114 – Stage storage relationship for Gueydan

Polder Ring: Kaplan

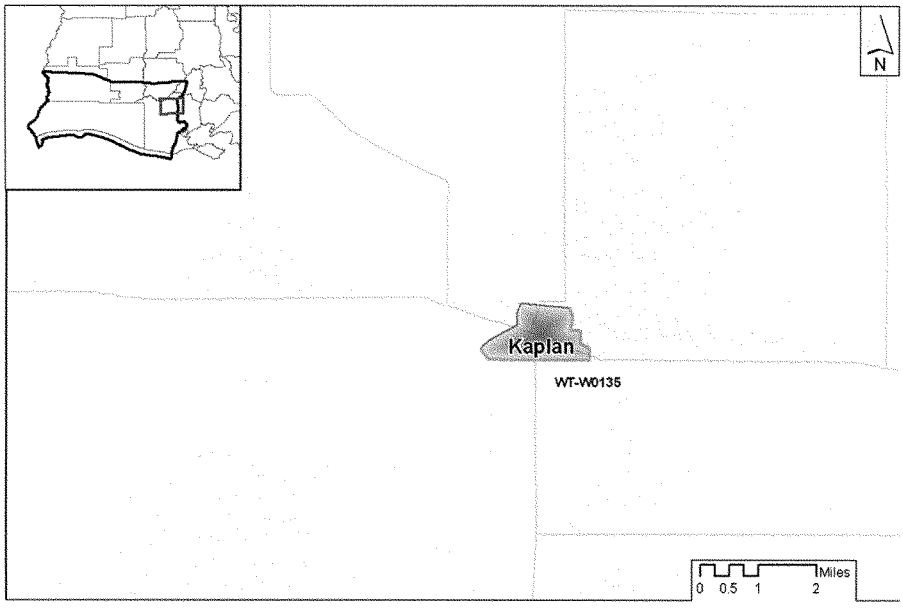


Figure B.115 – Sub-basin and levees Kaplan

Table B-58 – Levee characteristics Kaplan

			Design Height (waves modeled without friction)		
Reach ID	**Length**	**Authorized Height**	**100 year**	**400 year**	**1000 year**
[-]	**[ft]**	**[ft]**	**[ft]**	**[ft]**	**[ft]**
WT-W0135	16630	n.a.	9	15.5	18.5

Storage Area	**Pumping Rate [cfs]**	**Type**	**Notes**
Kaplan	5496	single area	Equally to other smaller ring levees Kaplan is only overtopped from the seawards side

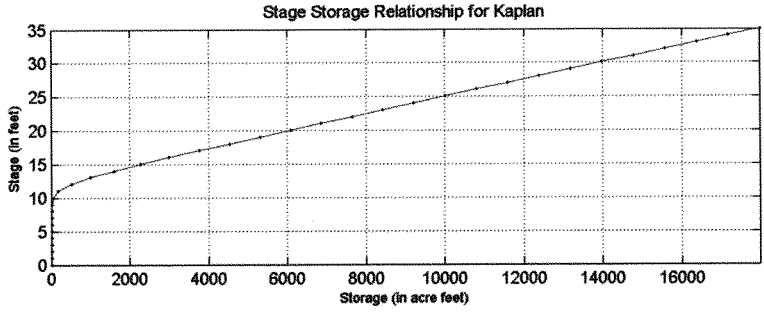


Figure B.116 – Stage storage relationship for Kaplan

FOR CONTINUATION OF HOUSE DOCUMENT 111-129
LOUISIANA COASTAL PROTECTION AND RESTORATION (LACPR) REPORT
SEE PART 3