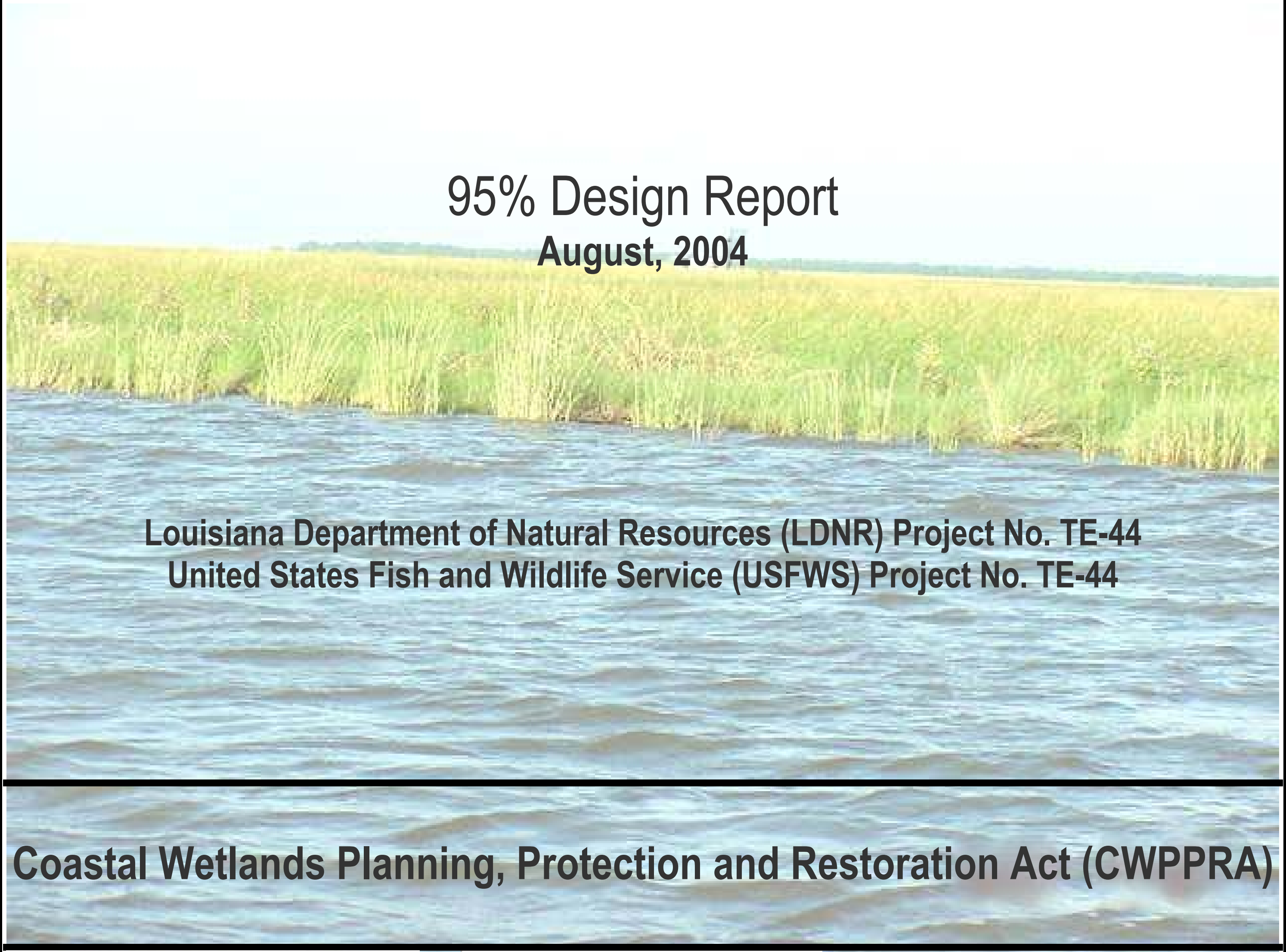
North Lake Mechant Landbridge Restoration

Terrebonne Parish, LA



95% Design Report

August, 2004

Louisiana Department of Natural Resources (LDNR) Project No. TE-44
United States Fish and Wildlife Service (USFWS) Project No. TE-44

Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA)

Table of Contents

1.0 INTRODUCTION

The Lake North Lake Mechant Landbridge Restoration Project (herein referred to as TE-44) is located in the Terrebonne Basin along the northern shore of Lake Mechant as shown in Figure 1. The Louisiana Coastal Wetlands Conservation and Restoration Task Force designated TE-44 as part of the 10th Priority Project List. The United States Fish and Wildlife Service (USFWS) was designated as the lead federal sponsor with funding approved on 1/10/01 through the Coastal Planning, Protection and Restoration Act of 1990 by the United States Congress and the Wetlands Conservation Trust Fund by the State of Louisiana. The Louisiana Department of Natural Resources (LDNR) is serving as the local sponsor and will also be performing engineering and design.

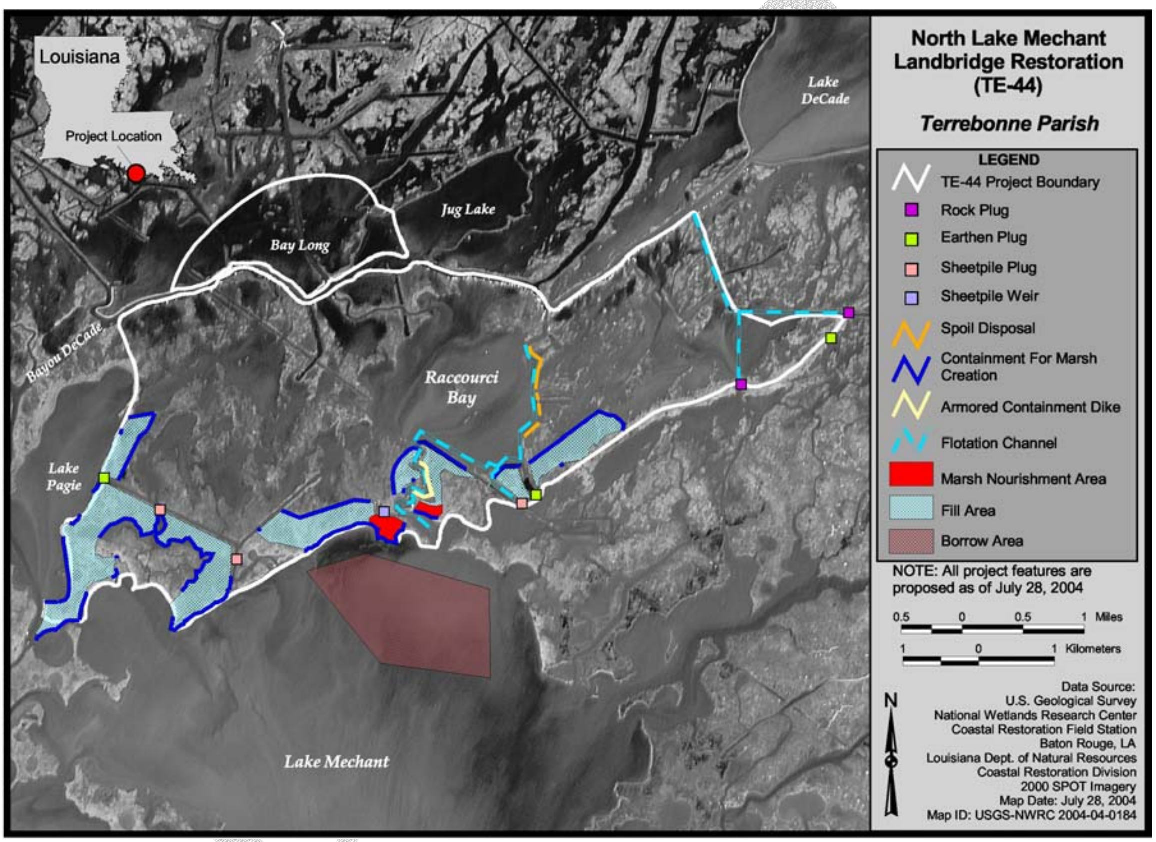


Figure 1 – Project Features

The North Lake Mechant Landbridge serves as a critical barrier between the easily erodible fresh marshes north of Bayou De Cade and the higher saline environment of Lake Mechant. At the present shoreline erosion rate of 7.5 feet/year,the north Lake Mechant shore will soon fail to act as a barrier, allowing the hydrologic connection between Lake Mechant and the fresher marshes to the north.

Additionally, erosion and deterioration along the banks of Raccourci Bayou are threatening to enlarge and straighten this winding tidal pass into a major conduit for water exchange. These changes will accelerate the loss of the remaining interior marshes, extend lake-like conditions, and increase salinities north to Bayou De Cade.

Should shoreline breaching and enlargement of tidal channels allow high tidal energy conditions to intrude into the project area, the organic interior marshes would likely experience increased loss rates.

Restoration strategies to be used for this project include marsh creation, breach repairs, and shoreline protection as shown in Figure 1. The proposed marsh creation will be achieved by mining sediment from northern Lake Mechant to fill open water and broken marsh areas along the northern lake rim. A series of plugs and weirs will be installed in an attempt to restore the integrity of the landbridge in locations that have been penetrated by erosion and canal dredging. Shoreline protection measures will include planting well-rooted, salt tolerant smooth cordgrass (Spartina alterniflora) plants along the shorelines of Lake Mechant, Goose Bay, and Lake Pagie and installing concrete mat systems along Bayou Racourcci. The plantings were installed in the summer of 2003 as construction unit #1. All other project features will be constructed as part of construction unit #2.

Engineering and design for construction unit #2 is being conducted by LDNR's Coastal Engineering Division (CED). Topographic, bathymetric, and magnetometer surveys, and a geotechnical investigation have been completed. Additionally, a wave analysis and a water level determination have been performed by CED. These efforts have been used to select material types and construction methods compiled in a set of construction plans and specifications.

In February 2001, the Louisiana Department of Wildlife and Fisheries (LDWF) established a public oyster seed ground in Lake Mechant. That seed ground and several private oyster leases may impact proposed construction activities. LDNR has conducted an oyster survey on the private leases which may serve as a baseline in negotiating compensation or relocation expenses. CED has also designated a 1500 foot buffer between the proposed sediment dredging area and the LDWF seed ground.

2.0 SURVEYS

In preparation for this project, CED contracted with survey firms to perform two topographic surveys and a magnetometer survey. This also involved establishing a survey monument to be included in LDNR's secondary GPS network.

2.1 Topographic

A topographic survey was completed on June 21, 2002 by ABMB Engineers, Inc. in order to facilitate the design of the marsh creation areas, plug features, and shoreline protection features. After further project development and the addition of new project features, CED decided to obtain another survey which was also completed by ABMB Engineers, Inc. on October 13,2003. Transects from these two surveys were combined and the layout is shown on sheet 3 of the project plans.

The transect intervals for the marsh creation fill areas were either 250' or 500', while borrow area transects taken in Lake Mechant were spaced at 1000' intervals. Lake Mechant bottom elevations were collected directly using a 4 meter antennae pole and a GPS Real-Time Kinematic (RTK) device. This method eliminated the need for any corrections due to water level or wave heights. Other survey transects were taken at irregular intervals specific to individual project features.

2.2 Secondary Monument

Installation of a permanent secondary monument, designated as TE-44-SM-A, was completed on June 20, 2002 by John Chance Land Surveys, Inc. This was not completed until after the first topographic survey by ABMB. ABMB did however install project benchmarks to base their first survey on which was then tied back to TE-44-SM-A. The monument has coordinates of 29°20'04.466514"N, 90°58'30.665643"W and an elevation of 0.84 ft NAVD88. It is also now part of the LDNR secondary GPS network.

2.3 Magnetometer

In order to locate pipelines and other potential obstructions to construction activities, CED contracted with Neel-Schaffer, Inc. to perform a magnetometer survey in the project area. The survey was completed on November 31, 2002. The data was collected using a G-881 Cesium marine magnetometer. Magnetometer lines were run within the dredging borrow area and in all other areas where dredging or equipment access is anticipated. In areas where magnetometer "hits" were interpreted as possible pipelines or major obstructions, a probe was used to identify the object and determine its depth.

3.0 GEOTECHNICAL

In order to determine the suitability of the soils in the TE-44 project area for the various proposed construction alternatives, a geotechnical investigation was performed by Soil Testing Engineers, Inc. (STE) and completed on October 31, 2002. STE was tasked to collect soil borings, perform laboratory tests to determine soil characteristics, calculate settlement of all structures including the dredge fill for different fill elevations, perform stability analyses on the plugs and shoreline protection features, and determine a cut to fill ratio for dredge and fill operations.

3.1 Soils Investigation

A total of seventeen subsurface borings were drilled in the project area from July 29, 2002 – August 7, 2002 by STE at locations shown in Figure 2. Fourteen borings were drilled to a depth of 25 ft and three borings were drilled to a depth of 60 ft. The soil samples were tested in the laboratory for classification, strength, and compressibility.

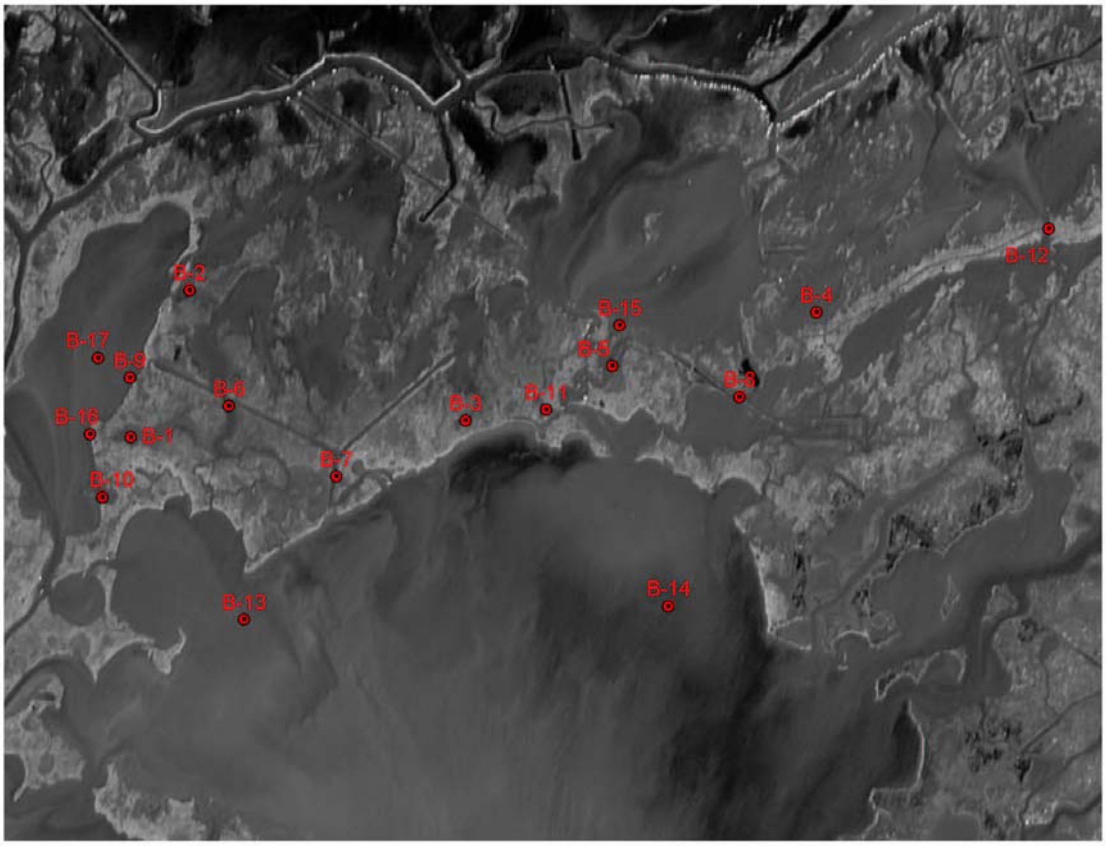


Figure 2 – Soil Borings Locations

3.2 General Evaluation

From a geological standpoint, the site is generally underlain by weak and highly compressible Delta Plain, Marsh deposits of Holocene Age (normally consolidated) to about Elev. -500 feet (+/-) NAVD 88. More competent Pleistocene materials begin below this depth. Most of the borings contain peat and/or organic clay in the top layers. On average the soils become somewhat more desirable with depth. The bottom layers contain less organics and stronger clays and silts with the occasional lens of silty sand.

3.3 Settlement Analyses

Settlement analyses were performed using VSTRESS originally developed by the Corps of Engineers and SETOFF as developed by Ensoft, Inc. One-dimensional settlement was calculated based on Boussinesq stress distributions. For the soil types that required consolidation tests, actual consolidation curves were used in the calculations. Published correlations were also used to obtain consolidation indices using Atterberg Limits and moisture content values.

Settlement analyses were run for shoreline protection, marsh creation fill, earthen containment dikes, earthen plugs, and rock plugs. The results of these analyses are based on the assumption that the features are placed on the existing ground surface. Settlement values for specific project features are discussed in each individual design section of the STE report.

STE recommended staged construction for all features due to the extremely weak foundation soils encountered. Actual settlements are predicted to increase as earthen material or rock is added. The structures should be initially constructed to target elevations and, upon completion of the majority of the anticipated settlement, built up in sequential stages to the final design elevations.

3.4 Slope Stability Analyses

Project features evaluated for slope stability include shoreline protection, containment dikes, rock plugs, and earthen plugs. Slope stability analyses were performed using XSTABL marketed by Interactive Software Designs. The Bishop method of analysis was used to find the accepted measure of a slope's stability or its factor of safety. This is defined as the ratio of resisting forces to the driving forces. Several trial failure surfaces were evaluated to determine the failure surface yielding the minimum factor of safety.

Due to the extremely soft soils at this site, it is very difficult to achieve the normally accepted safety factor without the use of geotextile reinforcement. Therefore, a minimum safety factor of 1.3 was used for structure stability assuming a layer of high strength woven geotextile fabric is placed beneath the earthen plugs, shoreline protection, and rock plugs. A maximum woven geotextile strength of 4200 lbs/ft is recommended for design.

3.5 Sheet Pile Analyses

Upon review of the settlement analyses at potential plug locations, borings B-6, B-7, and B-8, STE concluded that the amount of geotextile reinforcement and rip-rap material needed for rock plugs would most likely preclude this from consideration. Hence, STE proceeded with the evaluation and design of cantilevered steel sheet pile walls to serve as plugs in these locations. The computer program CWALSHT developed by the USACE Waterways Experiment Station was employed for the analyses. The design criteria is further discussed in Section 7.1.

4.0 HYDRAULICS

Hydraulic calculations performed during the design of this project included historical water level and design wave height determinations. These values were then used in the design of all TE-44 project features.

4.1 Historic Water Levels

USGS stage recorder #0738165067 was selected to determine historical water levels due to its close proximity to the project area and database availability. It is located on Bayou Raccourci at 29°20'18"N, 90°57'08"W near the center of the TE-44 project area where Bayou Raccourci meets Lake Mechant. Approximately 2½ years of hourly water level data was recorded from 8/27/99 - 3/19/02. This data was used to determine mean high water (MHW) and mean low water (MLW) values.

A normal tidal epoch lasts approximately 19 years. In order to accurately estimate MHW and MLW, a data set which has less than 19 years of data should be correlated to a gauge which has a data set of 19 years. CED used NOAA station #8761724 located at Grand Isle near Barataria Pass at 29°15'48"N, 89°57'24"W as a control station for making this correlation. The period of record used for the 19 year tidal epoch was from January 1, 1984 to December 31, 2002. The method used by CED to make this correlation is summarized below in Table 1.

TABLE 1 – Summary of Water Level Determination

KNOWN VARIABLES	ELEV. FT NAVD 88
MHWc = 19 YEAR MEAN HIGH WATER AT CONTROL STATION	1.35
MTLc= 19 YEAR MEAN TIDE LEVEL AT CONTROL STATION	0.83
MLWc = 19 YEAR MEAN LOW WATER AT CONTROL STATION	0.30
MRc = 19 YEAR MEAN TIDE RANGE AT CONTROL STATION	1.05
TLc = MEAN TIDE LEVEL FOR THE OBSERVATION PERIOD AT CONTROL STATION	0.87
Rc = MEAN TIDE RANGE FOR THE OBSERVATION PERIOD AT CONTROL STATION	1.00
TLs = MEAN TIDE LEVEL FOR THE OBSERVATION PERIOD AT SUBORDINATE STATION	0.90
Rs = MEAN TIDE RANGE FOR THE OBSERVATION PERIOD AT SUBORDINATE STATION	1.12
CALCULATED VARIABLES	ELEV. FT NAVD 88
MHWs = 19 YEAR MEAN HIGH WATER AT SUBORDINATE STATION (MHWs=MTLs+MRs/2)	1.45
MTLs= 19 YEAR MEAN TIDE LEVEL AT SUBORDINATE STATION (MTLs=TLs+MTLc-TLc)	0.86
MLWs = 19 YEAR MEAN LOW WATER AT SUBORDINATE STATION (MLWs=MTLs-MRs/2)	0.27
MRs = 19 YEAR MEAN TIDE RANGE AT SUBORDINATE STATION MRs=(MRc*Rs)/Rc)	1.18

REFERENCE: Cole, George M. Water Boundaries. New York, NY: John Wiley & Sons, Inc., 1997. pp. 24-27.

4.2 Design Wave

In order to design shoreline protection features proposed for this project, a design wave must first be determined. CED followed guidelines set forth in a technical memorandum (TM) issued jointly in January of 2000 by DNR and the Natural Resources Conservation Service (NRCS) entitled “Design Guidelines for CWPPRA Shoreline Protection Structures”. This TM is intended for use in conjunction with the U.S. Army Corps of Engineers (USACE) Shoreline Protection Manual (SPM). The edition of the SPM used in these calculations was the 1984 edition, second printing.

Objective 2 of the TM is to determine design approaches for wave heights of different structural measures. Step 1 of this objective is to determine normal conditions. Given the data available, option 1 of 2 was chosen to determine mean MHW and MLW. Option 1 recommends using spring and summer water elevations in the MHW determination. CED used the entire year due to the lack of riverine influence in the area. MHW (1.45 NAVD 88) plus 1 foot of storm setup was used as the still water elevation (SWE = 2.45 NAVD 88) in the wave height calculations.

Step 2 is to determine wave heights. The first criteria is to set the minimum design height of the wave at 2.0 feet. The maximum wave design height is set at 0.78 times the water depth. For a water depth of 2.45 feet (bottom elev. = 0, SWE = 1.45) the maximum design height came out to be 1.91 feet. This was then compared against a wind generated wave for comparison. The wind speed chosen was 30 mph with the largest fetch in the project area being 7,000 ft in Raccourci Bay. Using these criteria and Figure 3-27 in the SPM for shallow water waves in depths of less than 5 ft, the wind generated wave height was 0.8 ft. Finally, these wave heights were compared to those generated by boat waves. Shrimp boats and small crew boats which produce wave heights ranging from 1.5 – 2.5 ft were considered. To be conservative CED chose to use the upper end of this range being 2.5 ft for design.

5.0 MARSH CREATION DESIGN

This project proposes to create marsh by pumping sediment from Lake Mechant into areas designated in Figure 1 using hydraulic dredging techniques. The three components which must be evaluated in this process are the dredge borrow area, the dredge fill areas, and the containment dikes.

5.1 Borrow Area

The first step in designing a borrow area is determining its location. Two major considerations affecting this determination were oyster leases and proximity to fill areas. Figure 3 shows the locations of oyster leases in the project vicinity as well as the Louisiana Department of Wildlife and Fisheries (LDWF) oyster seed grounds.

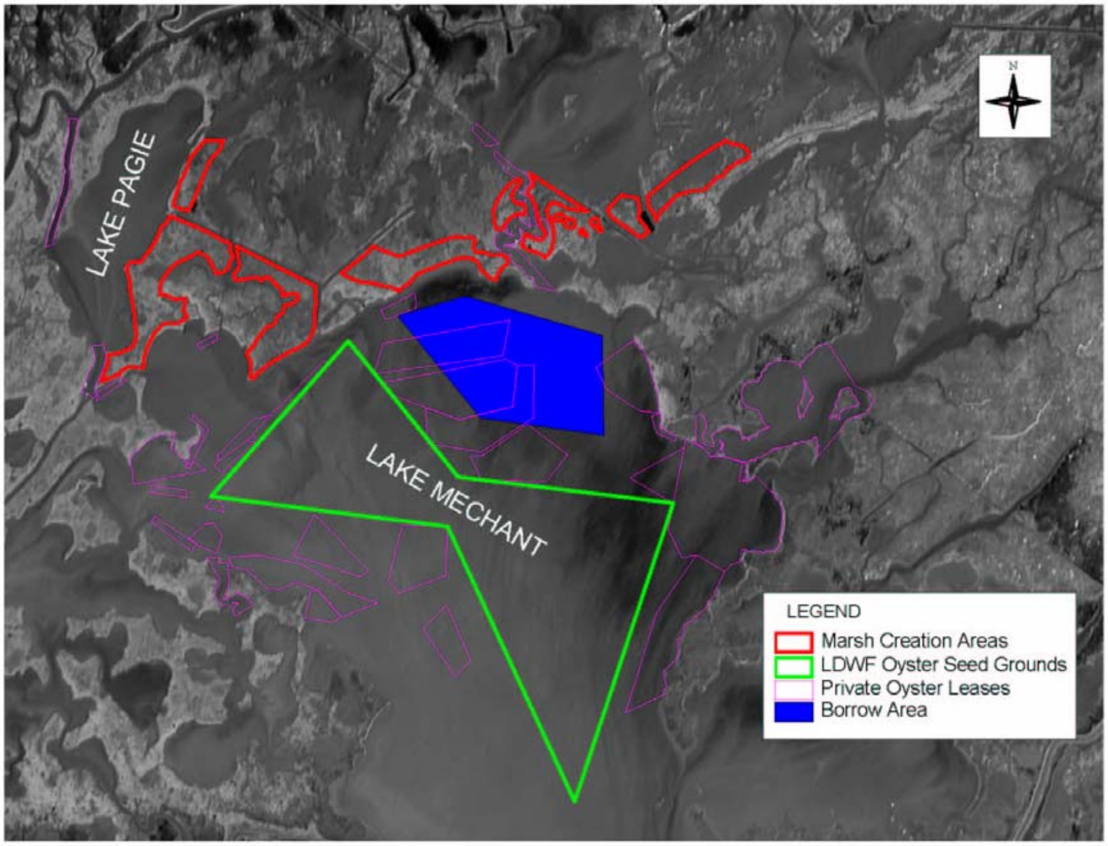


FIGURE 3 – Oyster Map

In an attempt to avoid existing oyster leases, Lake Pagie was considered as source of borrow material. Although Lake Pagie does not contain any oyster leases, it is a shallow lake with a large population of submerged aquatic vegetation (SAV). Deepening Lake Pagie would destroy this SAV habitat and possibly create anoxic zones. This is not as likely to occur in Lake Mechant because it is much larger in area and is less likely to stagnate. Another reason Lake Pagie is not favorable is that the pumping distance to the eastern most fill site is just over four miles. This would make the need of booster pumps more likely than the more centralized borrow area proposed in Lake Mechant.

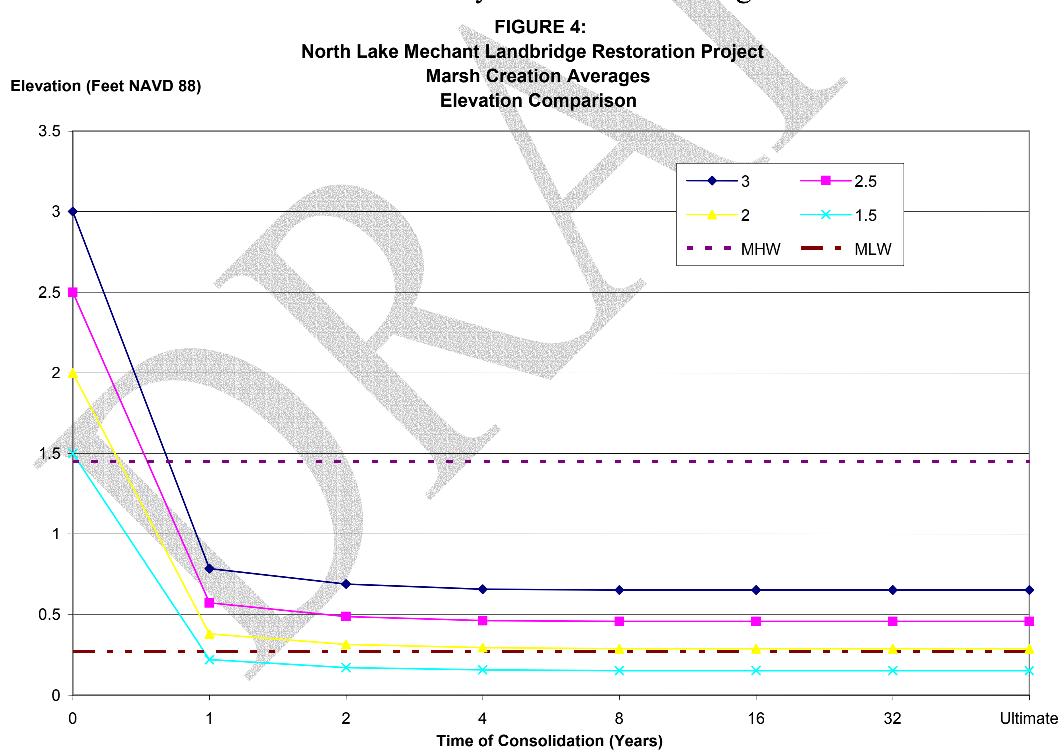
For these reasons, Lake Mechant was chosen as the borrow area. Criteria used to determine the borrow area size include fill volumes needed, depth of cut, cut to fill ratios, and an oyster seed ground buffer zone of 1500 ft set by LDWF. It was decided to negotiate with the private oyster lease holders through the LDNR's Oyster Lease Acquisition Program.

Due to void spaces filled by water in the undisturbed soil of the Lake Mechant borrow area, the volume of material dredged will be greater than that of the material placed. Water and material will be lost during operations and volume will be decreased during soil consolidation and compaction. Because of this phenomenon, a cut to fill ratio must be estimated to determine dredging quantities. STE used soil conditions present in borings B-13, B-14, B-16, and B-17 to estimate a cut to fill ratio of 2 to 2½. To be conservative, STE recommended using 2½ for estimating purposes.

5.2 Fill Area

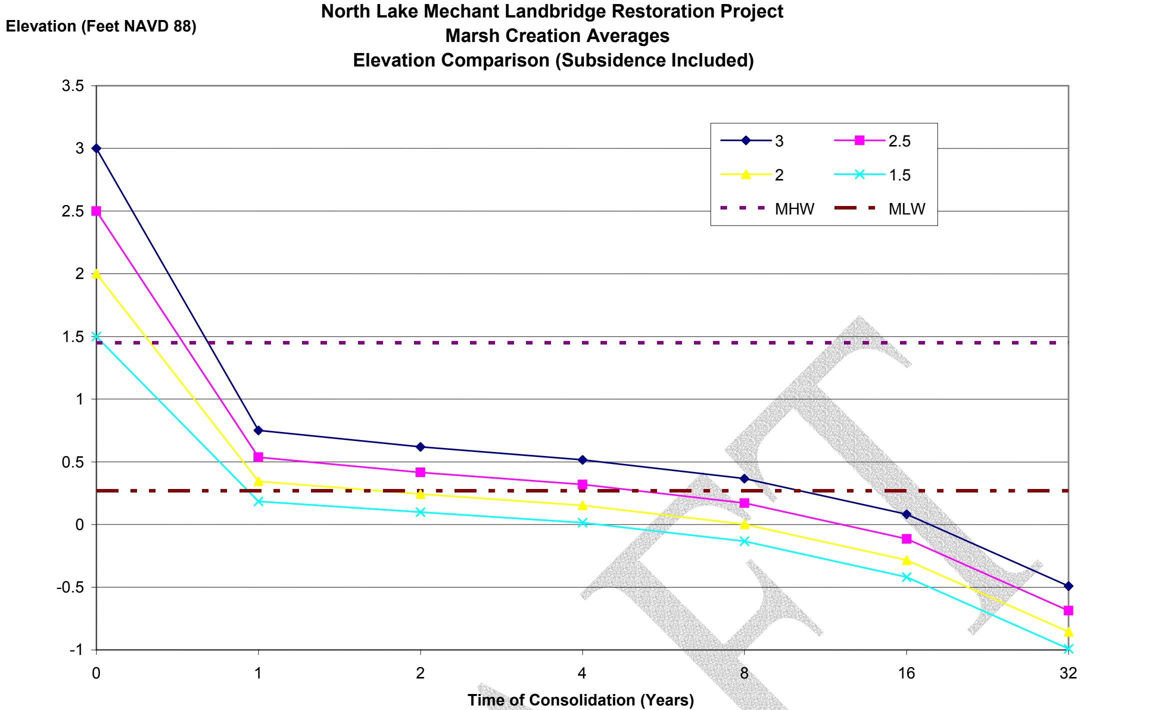
In order to evaluate the performance of the created marsh over the 20 year life of the project (standard for CWPPRA), the project team decided that the criteria would be marsh elevation. Ideally, biologists from both USFWS would like the created marsh to be inter-tidal. This means that the final marsh elevation (after initial consolidation and settlement) would fall between mean high water (MHW) and mean low water (MLW). To achieve this, the marsh platform will initially have to be pumped higher than MHW and settle into the inter-tidal zone over time. A third factor, subsidence, also plays a role in decreasing elevation of the marsh. Subsidence will eventually cause the marsh to sink below MLW.

STE ran consolidation settlement calculations for boring locations B-1, B-2, B-3, and B-4 which are located in marsh fill areas. The calculations were performed for potential top of fill elevations of 3.0, 2.5, 2.0, and 1.5 feet NAVD 88. The consolidation settlement of the marsh fill was evaluated out to 32 years as shown in Figure 4.



An additional factor effecting marsh elevation in this area is subsidence. The subsidence rate in this area is .03576 ft/yr (1.09 cm/yr) (Penland and Ramsey, 1990). This rate was added to STE's settlement calculations and the results are shown in Figure 4.

FIGURE 5
North Lake Mechant Landbridge Restoration Project
Marsh Creation Averages
Elevation Comparison (Subsidence Included)



The goal of the marsh creation is to build marsh which has an elevation within the intertidal zone for the longest period of time possible. Intertidal marsh lies between MHW and MLW and is considered the most desirable in terms of environment benefits. Based on the elevation curves in Figure 5, an initial construction elevation of +3.0 ft NAVD 88 will result in intertidal marsh for the longest period of time. Consequently, elevation +3.0 ft was chosen as the target elevation for construction. Table 2 summarizes length of time for which the constructed marsh would remain intertidal for different construction elevations.

TABLE 2 – Intertidal Durations for Various Construction Elevations

CONSTRUCTION ELEV. (FEET NAVD 88)	BEGINNING AND ENDING INTERTIDAL YEARS	TOTAL TIME WITHIN INTERTIDAL ZONE (YEARS)
3.0	0.75-10	10.25
2.5	0.6-5	4.4
2.0	0.5-2	1.5
1.5	0-0.9	0.9

The settlement curves shown in Figures 4 and 5 were based on pumping to the listed elevations all at once. As recommended by STE, CED plans to reach the target construction elevation of +3.0 ft in two lifts. The first lift will be pumped to an elevation of +1.5 ft and allowed to dewater and consolidate for at least 30 days. This will make the construction elevation more achievable and should prolong the time for which marsh elevation remains within the intertidal zone. Following the first lift, the marsh fill will be placed to elevation +3.0 ft. Another factor not taken into account in the settlement curves is biomass accumulation. Decomposed biological matter should also contribute to marsh elevation over the life of the project.

5.3 Containment Dikes

STE also evaluated the structural stability of earthen levees or dikes which will be used to contain dredge fill material. STE used a maximum crown elevation of +4.0 feet NAVD 88 in their settlement analysis. This value would provide for 1 foot of freeboard if the containment areas are pumped to an elevation of 3 feet NAVD 88. This produced a range of settlement values of 1.5 to 2.5 feet depending on the thickness of underlying peat layers. In order to minimize settlement, STE recommended using staged construction, using initial design elevations as a target for the first stage. Additionally, STE recommended minimum side slopes of 4 to 1 horizontal to vertical and a minimum distance of 25 feet between the toe of the dike and the borrow area to maintain dike stability. A detailed cross section of the containment dikes is shown on sheet 14 of the project plans.

6.0 SHORELINE PROTECTION DESIGN

Originally, rock shoreline protection features were proposed at the four locations in the project area shown on Figure 6. After investigating the logistics of rock dike construction in the four proposed areas, this alternative became less attractive. The main issues facing designers were access to remote locations, proximity to pipelines and power lines, stability of the structure, and containment of marsh fill.

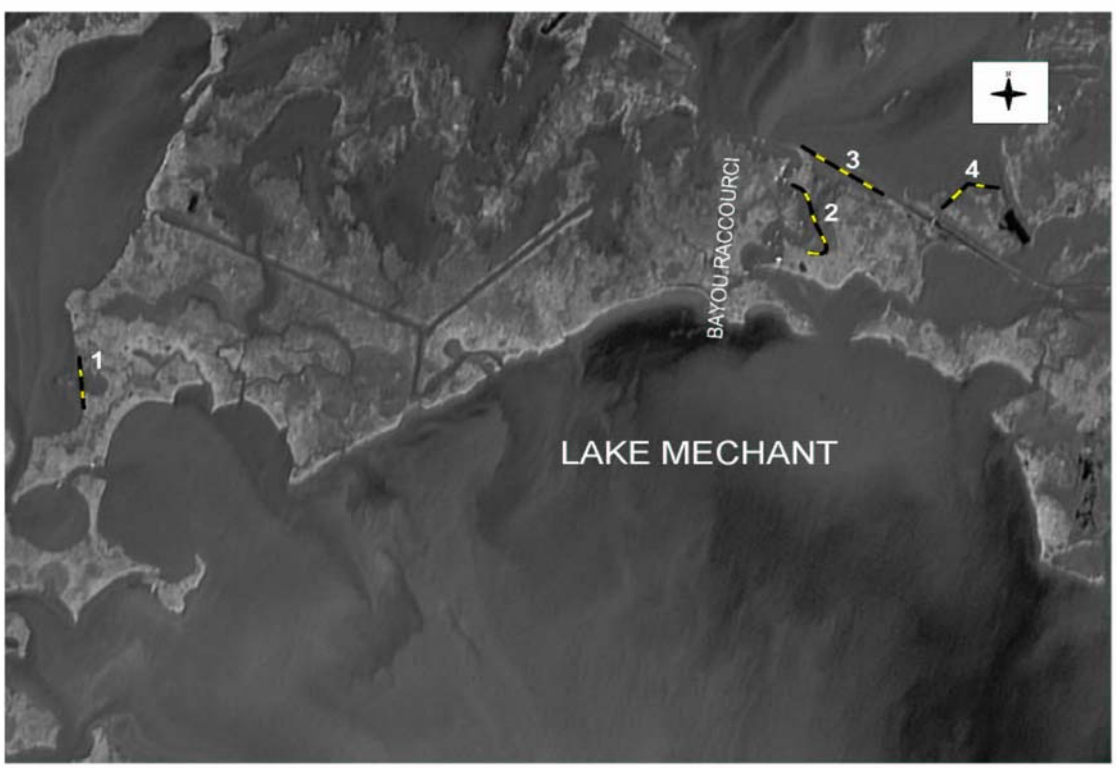


FIGURE 6 – Originally Proposed Shoreline Protection Locations

To assess the need for shoreline protection in these four areas, design waves as calculated in Section 4.2 were considered. Due to boat traffic along Bayou Raccourci, a design wave height of 2.5 ft (as determined in section 4.2) was used for location #2. Wave heights of this magnitude could cause considerable erosion along the banks of Bayou Raccourci. The other three locations also border marsh creation areas and will have containment dikes built to the dimensions specified in Section 5.3. These containment dikes will help prevent erosion from the smaller predicted waves heights. It is recommended that containment dikes at locations #1, #3, and #4 not be degraded at any time during the project life for fish access.

Because location #2 is located along Bayou Raccourci, which is over 20 feet deep in some locations, CED designers had to carefully consider the alignment of the shoreline protection feature due to bank stability. A structure too close to the bayou edge could result in slope failure. The likelihood of slope failure would be increased by dredging for equipment or rock barge access. Power lines and poles running adjacent to the bayou also present access problems for rock placing equipment. Together, bank stability and utility alignments lead CED to consider an alternative shoreline protection method to protect the banks of Bayou Raccourci from erosion.

The alternative chosen is an armored containment dike. This was chosen because the alignment borders marsh creation area six and will have to function both as containment and shoreline protection. The dimensions of the dike will be the same as those which are unarmored. The armor material used will be a concrete mat system previously used by DNR and USFWS on the TE-41 Mandalay Bank Shoreline Protection Demonstration CWPPRA project. To account for the extra weight added to the dike by the mat, the dike will be built on a geogrid material. The dike will be inspected and approved for height after initial settlement. Once the dike section is approved, a geotextile fabric will be placed on top of the earthen material before the concrete mat is placed. A typical cross section for this structure is shown on sheet 13 of the project plans.

7.0 PLUG DESIGN

Three types of plugs were designed as part of this project to help restore the land bridge integrity. Various construction materials considered include steel sheet pile, rock, and earthen material. Material types were chosen based on width and depth of the cut or breach being plugged and soil properties.

7.1 Steel Sheet Pile Plugs

The steel sheet pile plugs were designed by STE as part of their geotechnical investigation scope of work. STE used the computer program CWALSHT developed by the USACE Waterways Experiment Station for the design. Two load cases were considered in the analyses. The first case considered a typical non-breaking wave force of 1,000 lbs/ft, applied at an elevation of +2.34 NAVD 88, which includes 1-foot above

MHW. This non-breaking wave force was calculated taking the mean water depth for the sheet pile structures. The second load case considered the complete siltation of one side of the sheet pile plug to an elevation of +2.0 NAVD 88. A safety factor of 1.5 was applied to the passive pressures to determine the penetration requirements for the cantilevered sheet pile plug structures. The use of a safety factor of 1.5 will produce an unrealistic moment distribution along the wall resulting in an unknown safety factor. Therefore, the maximum design moments were determined using service loads (factor of safety = 1.0). The penetration requirements were determined using a safety factor of 1.5 for the passive resistances; the maximum design moments were determined using service loads. The results of the analysis indicates that a minimum steel sheet pile section equivalent to a PDA-27, Grade 42 be utilized for all of the sections except for sheet pile plug 3, which requires a PDA-27, Grade 50 section at the deepest point of the crossing.

The top elevation of the sheet pile plugs is +4.0 NAVD 88. The design also includes sheet pile caps, exterior epoxy coating, and earthen wing wall tie-ins which will be built to an elevation of +5.0 NAVD 88 and armored with concrete mats. Details and sections for the steel sheet pile plugs are shown on sheets 20 – 23 of the project plans.

<u>7.2 Weir Removal and Replacement</u>

An existing weir is located within the project area at N29º20’19”, W90º57’19”. This weir is in poor shape and is no longer functioning properly. Originally CED and USFWS considered repairing the weir. Ultimately, CED could not locate enough information on the original structural design to determine if a repair would be sufficient. Therefore, CED decided to remove the weir entirely and replace it with either a steel sheet pile plug or steel sheet pile weir. NOAA Fisheries requested that fish access be maintained in this canal. CED and USFWS agreed to this request after determining that project goals would not be compromised.

CED designed the steel sheet pile weir after the completion of the geotechnical investigation. To be consistent, CED also used the CWALSHT software in designing the weir. CED used the same design criteria listed in section 7.1 that was used for the sheet pile plugs as well a head differential of 1.0 ft to account for rising or falling tides across the weir. Soil properties were obtained from boring B-11, which was taken near the weir. Details and sections of the existing weir to be removed and the replacement steel sheet pile weir are shown on sheets 24 -26 of the project plans.

<u>7.3 Rock Plugs</u>

Originally the TE-44 project plan called for one rock plug at the Little Deuce cut in the Bayou La Pointe ridge located at N29º21’10.9”, W90º54’24.6”. As the project developed, a second rock plug was added further north in an old access canal cutting through the ridge at N29º21’40.9”, W90º53’28.92”. This second plug was originally part of the TE-39 South Lake Decade Freshwater Introduction CWPPRA project.

Because the second rock plug was added to the project after the geotechnical investigation, settlement and stability analyses were performed only for rock plug #1 at the Little Deuce. The results showed a maximum settlement of approximately 5.5 feet. This large number is due to the depth of the structure. The bottom elevation of the cut to be plugged with rock is approximately -12.0' NAVD 88 at the deepest point. With a top elevation of +4.0' NAVD 88, this results in almost 16 feet of rock before settlement is factored in. The rock will be placed in a staged fashion until all initial settlement has taken place and the crown elevation is achieved.

The bottom elevation of the channel at the proposed location of rock plug #2 is -6.0 NAVD 88 at its deepest point. This results in approximately 10 feet of without factoring in settlement at this locationDue to the lack of geotechnical information at this location, CED compared rock volumes between rock plug #1 and #2 and estimated 3 feet of settlement for rock plug #2.

Rock plug dimensions include a +4.0' NAVD 88 crown elevation, a 10' crown width and 3 to 1 side slopes. These dimensions were determined through a slope stability analysis done by STE using a safety factor of 1.9. The rock will be placed over a woven geotextile fabric which help distribute loads and separate the rock from the underlying soils. Details and sections of the two rock plugs are shown on sheets 15 and 16 of the project plans.

7.4 Earthen Plugs

The TE-44 project plan calls for three earthen plugs at locations shown in Figure 1. Earthen material was chosen for these plugs because the openings being closed are smaller in volume and receive less flow than the sheet pile and rock plugs. Earthen plugs #1 and #2 close breaches which have deteriorated over time, and earthen plug #3 is a repair of an existing access canal plug.

All three earthen plugs are designed with a top elevation of +4.0 NAVD 88 and 4 to 1 side slopes. Earthen plugs #1 and #2 will be placed over woven geotextile to help aide in stability and #3 will be place directly onto existing spoil.

Borrow areas for the earthen plugs were designed using a cut to fill ratio of 1.3 to 1. This resulted in borrow depths at elevation -20' NAVD 88. To maintain stability, the borrow areas will be at least 40 feet from the toe of the plug. Details and sections of the earthen plugs are shown on sheets 17 – 19 of the project plans.

8.0 COST ESTIMATE

CONSTRUCTION COST ESTIMATE

Item	Description	Unit	Number of Units	Unit Cost	Total
1	Mob/Demob	ls	1	$ 1,000,000	$ 1,000,000
2	Riprap (rock)	tons	11,000	$ 45.00	$ 495,000
3	Armor Mats	sq yd	15,500	$ 60.00	$ 930,000
4	Geotextile Fabric	sq yd	23,500	$ 3.00	$ 70,500
5	Geogrid	sq yd	13,500	$ 8.00	$ 108,000
6	Settlement Plates	ea	10	$ 1,000	$ 10,000
7	Warning Signs	ea	10	$ 2,000	$ 20,000
8	Hydraulic Dredging*	cy	3,360,000	$ 5.00	$ 16,800,000
9	Containment Dikes	lf	57,000	$ 9.00	$ 513,000
10	Steel Sheetpile	sq ft	21,675	$ 25.00	$ 541,875
11	Earthen Plugs	cy	30,000	$ 2.00	$ 60,000
12	Weir Removal	ls	1	$ 50,000	$ 50,000
	Subtotal				$ 20,598,375
Contingency (15% x total)					$ 3,089,756
GRAND TOTAL					**$ 23,688,131**

*Note: Based on fill in place.

9.0 MODIFICATIONS TO APPROVED PHASE 0 PROJECT

Phase 0 components were conceptual and it was the intent of the 30% design, and the 95% design to assist in the possible project component revisions. The original Phase 0 features were; 1) Earthen containment dikes constructed on the southern boundary of project area where existing marsh or spoil banks do not provide adequate elevations to contain the dredged material; 2) Dredged material for borrow sites in Lake Mechant, Goose Lake and Lake Pagie; 3) Armoring "weak spots" along Lake Pagie; 4) Construct two earthen plugs; 5) Construct four sheet pile plugs; 6) Construct three rock dikes were wave erosion could be a factor; 7) Construct two rock plugs; 8) Repair of existing weir.

Changes from Phase 0 to final design are; 1) Armoring weak spots along Lake Pagie was deleted because of cost and earthen containment dikes would achieve the goal of protecting the marsh. This was a change of approximately 4.8 acres reduction in benefits; 2) Weir repair became weir replacement because of cost; 3) Rock dikes along Raccourci Bay and Raccourci Bayou were deleted or replaced with armored mats because of additional wave analysis and pipeline issues; 4) A new earthen plug was added in existing land bridge ridge because that feature was not included in CU1 of the South Lake Decade project and it was deemed necessary for the integrity of this land bridge project; 5)

Additional containment dikes will be added along the southern boundary project area; 6) Fill sites 7,8,9 were eliminated because of the small amount of acres (total for all three sites was 8.6 acres) benefited and the cost because of size and accessibility; 7) The remaining fill sites were adjusted as per aerial photography and to account for actual containment dike alignment; these acreages were approximately the same as the acreages stated in Phase 0.

All changes were deemed to be less than a 25% change in benefits, so therefore no new WVA analysis was conducted.

10.0 30% DESIGN MEETING COMMENTS

Issues Resolved from the TE-44 30% Design Meeting:

1) Paying on the Fill Volume - It was concluded that the construction of "training dikes" will be added at the contractors discretion in order for fill volumes to be met and maintained across the project area. This would result in more than one lift cycle on each fill area. One month was decided as the time frame after an area has been filled, then it will be surveyed to determine if the desired height was reached and the volume on which to pay the contractor.

2) Permit Application and Drawings – (done) Permits applied for and acquired.

3) Borrow sites – The borrow sites in Lake Pagie and Goose Lake were eliminated because of pipeline crossings, geo-technical analysis, and surveys. One borrow site in Lake Mechant was delineated as the sole site for dredging. This site is at a minimum of 1500 feet away for the State seed grounds for oysters.

4) Fill height – Fill height was determined, as per geo-technical report, to be approximately to an elevation of +3.0 feet NAVD 88.

Thirty percent concurrence was given in a letter written May 8, 2003, which stated "Based on our review of the technical information compiled to date, the ecological review, the preliminary land ownership investigation, and the preliminary designs, we, as local sponsor, concur to proceeding with the design of the project. Since oyster leases will be affected by this project, the assessment of potential impacts will be conducted in accordance with the CWPPRA Oyster Lease Acquisition Program adopted by the CWPPRA Task Force at their April 16th, 2003 meeting."

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