Lake Hermitage
Marsh Creation Project
(BA-42)

Preliminary Design Report
August 2008

Coastal Wetlands Planning, Protection, and Restoration Act

Louisiana Department of Natural Resources
Coastal Engineering Division

United States Fish and Wildlife Service

LAKE HERMITAGE MARSH CREATION PROJECT BA-42

PLAQUEMINES PARISH, LA

PRELIMINARY DESIGN REPORT
AUGUST 2008

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1.0   INTRODUCTION

The Lake Hermitage Marsh Creation Project (herein referred to as BA-42) is located within the Barataria Hydrologic Basin in Plaquemines Parish, Louisiana, to the west of the community of Pointe a la Hache, and northwest of the community of Magnolia as shown in Figure 1. The Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) Task Force approved BA-42 for Phase I (engineering and design) as part of the 15th Priority Project List. The United States Fish and Wildlife Service (USFWS) was designated as the lead federal sponsor with funding approved 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-Coastal Engineering Division (LDNR-CED) is serving as the local sponsor and is also performing the engineering and design work.
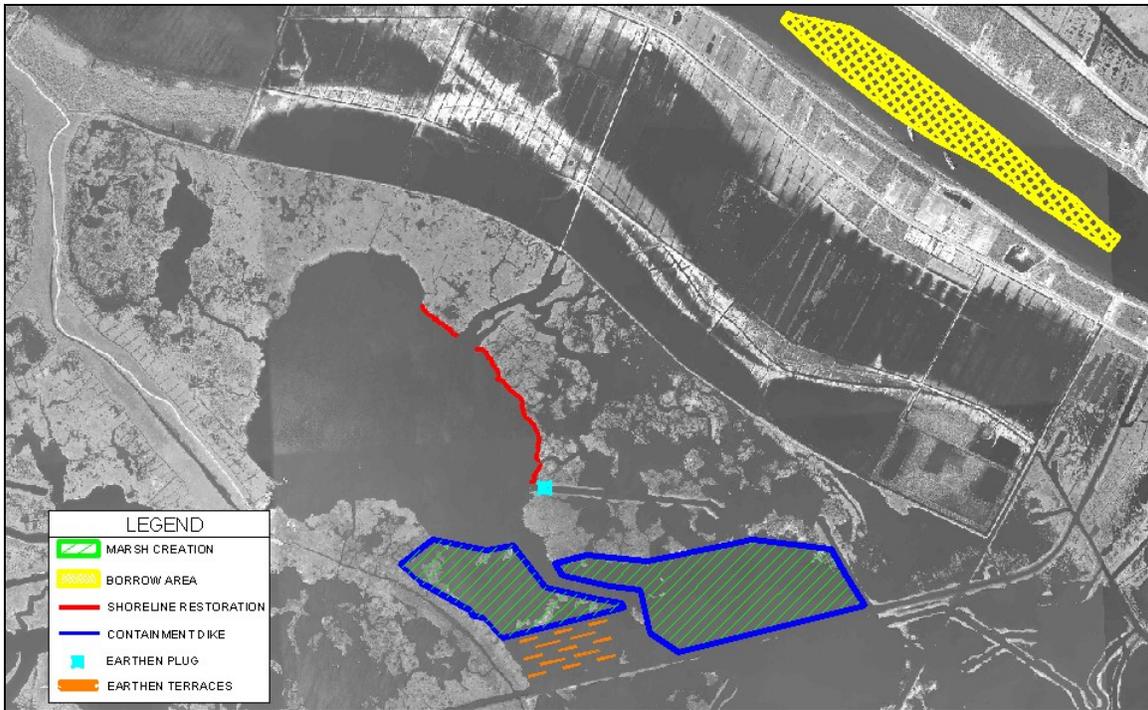
Figure 1 – Project Layout

The primary goal of BA-42 is to re-create marsh habitat in the open water areas along the southern rim of Lake Hermitage. This will maintain the lake-rim function along this section of shoreline, especially southeast of Lake Hermitage where very little land is left between the lake and the oil field canals. Interior ponding, subsidence, and shoreline erosion of the lake rim are the major causes of wetland loss in the project area. Although the shoreline erosion rates are relatively low, breaching and enlargement of tidal channels allow high tidal energy to intrude into the interior ponds of the project area and the interior marshes have experienced accelerated loss rates. Additionally, shoreline protection/restoration will reestablish the integrity of the eastern lake rim to prevent breaching into the interior marshes.

Restoration strategies to be used for this project include marsh creation, shoreline restoration, and terraces as depicted in Figure 1. The construction of the marsh creation sites for BA-42 involves hydraulically dredging sediment from the Mississippi River to fill open water areas located within the marsh. Small earthen dikes or levees will be constructed around the designated fill sites to contain the dredged slurry. Sediment dredged from the Mississippi River will also be used to construct the shoreline protection/restoration and earthen plug features. Topographic/bathymetric surveys, magnetometer surveys, geophysical surveys, and a geotechnical investigation have been completed. Additionally, a tidal datum analysis has been performed by LDNR-CED to determine the mean water elevations in the fill sites. These efforts have been carried out in order to determine a suitable target fill elevation, site conditions, and quantity of fill material needed for the project features.

The project team, consisting of members of USFWS and LDNR, performed an on-site kick-off meeting on June 27, 2006. Based on that meeting, a work plan was developed to identify and address the project requirements. The engineering and design, environmental compliance, real estate negotiations, operation/maintenance planning, and cultural resources investigations have been executed to the 30% level of completion as required by the CWPPRA standard operating procedures.

2.0   SURVEYS

Topographic, bathymetric, and magnetometer survey data was collected within the project area in order to facilitate the design of the marsh creation cells, shoreline restoration feature, earthen plug, and earthen terraces.   Additionally, a geophysical, bathymetric, and magnetometer survey was performed in the Mississippi River to delineate a sediment borrow site. The majority of the design survey effort was performed from February 2, 2007 to April 12, 2007 by Sigma Consultant Group, Inc. All horizontal coordinates are referenced to Louisiana State Plane Coordinate System, North American Datum of 1983 (NAD 83).   All elevations are referenced to North American Vertical Datum of 1988 (NAVD 88).

2.1   Horizontal and Vertical Control

Two permanent secondary monuments exist in the vicinity of Lake Hermitage. "BA04c-SM-01" is located on the bank of a pipeline canal south of the project area, at coordinates 29°32'00.97" N, 89°49'07.87" W. "876 1602 C TIDAL" is located in the vicinity of the Hermitage community, near the fire station, at coordinates 29°33'33.83" N, 89°53'05.03" W. These two secondary monuments were used as horizontal and vertical control for the fill site survey and the borrow site survey. The data sheets for these monuments are located in Appendix A.

2.2   Fill Site Surveys

A 23,000 foot survey baseline was established along the remnants of an oil field canal spoil bank south of the project area. This baseline is identified as "Oil Field Canal Baseline" in the Marsh Creation Plan View drawing included in Appendix B.   The baseline was staked with 10 foot lengths of 1 inch PVC pipe at the beginning and ending stations, and at 3,000 foot intervals. The baseline begins at the western end of the canal and proceeds northeastward to the bend in the canal. A second survey baseline was established along the eastern lake shoreline of Lake Hermitage.   This baseline is identified as "Eastern Lake Shoreline Baseline" in the Marsh Creation Plan View drawing included in Appendix B.   The baseline begins near an unnamed canal and proceeds northward for 7,000 feet along the eastern shore of Lake Hermitage.

Survey transects were taken along the two established baselines at 500 foot increments. The transects were taken in open water, across open marsh areas and extended 25 feet past existing marsh into open water along the perimeter of the fill sites. Position, elevation, and water depth were recorded every 25 feet along each transect or where elevation changes of greater than 0.5 feet occur. Transects extended beyond the spoil banks of the southern oilfield canal and across the marsh creation and terrace fill sites. Transects "Line 2" and "Line 3" were taken perpendicular to the survey transects that pass through Fill Site A and Fill Site B.   Transect "Line 1" was taken across Lake Hermitage.   Additional cross sections were taken near the bridge at the community of Hermitage and also at the designated earthen plug location. The bridge at the community of Hermitage was surveyed to determine the low chord elevation.

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2.3 Staff Gages

Two staff gages were set at the project site to monitor water levels during the topographic survey. Staff Gage 1 is located in Lake Hermitage at 29° 34" 03.85" N latitude and 89° 51" 41.81" W longitude. Staff Gage 2 is located at the end of a pipeline canal bordering marsh creation cell 1 at 29° 33' 24.72" N latitude and 89° 50' 46.35" W longitude. These staff gages were used to monitor water levels during the survey and to help verify the calculated tidal datum.

2.4 Marsh Elevation Survey

Average Marsh Elevation Surveys were conducted at three sites that were predetermined to have apparent healthy marsh. These surveys consisted of a minimum of twenty spot elevations at each location utilizing the same equipment used to acquire the elevations in the marsh creation cells. The survey shot was taken such that the tip of the rod was resting on the vegetation root. Average marsh elevations for each location were derived by using the following procedure: (sum of elevations at location # divided by total number of elevations at same location # = Average Marsh Elevation). Table 1 shows the results from data acquired from the three average marsh elevation surveys. All elevations shown are referenced to the North American Vertical Datum of 1988 (NAVD 88). Locations of the marsh elevation surveys are shown in the Marsh Creation Plan View drawing included in Appendix B.

SPOT ELEVATION	SITE NO. 1 N=393,831 E=3,748,880	SITE NO. 2 N=387,809 E=3,749,381	SITE NO. 3 N=387,518 E=3,745,997
1	1.15	0.92	1.18
2	1.10	1.00	1.21
3	1.31	1.34	1.18
4	1.19	1.13	1.23
5	1.33	1.03	1.17
6	1.35	0.95	1.13
7	1.17	1.13	1.23
8	1.08	1.13	1.36
9	1.18	1.31	1.18
10	1.13	1.14	1.31
11	1.11	1.10	1.21
12	1.07	0.95	1.23
13	1.07	1.00	1.22
14	1.09	1.08	1.15
15	1.13	1.39	1.15
16	1.07	1.06	1.15
17	1.04	1.10	1.11
18	1.17	1.34	1.12
19	1.17	1.25	1.22
20	1.18	1.26	1.12
21	1.10	0.99	1.15
TOTAL	24.21	23.63	24.99
AVERAGE	1.15	1.13	1.19

CUMULATIVE AVERAGE= 1.16

Table 1-Average Marsh Elevation Survey Results

2.5 Mississippi River Borrow Site Survey

A bathymetric survey was performed in the Mississippi River from River Mile 49.0 to 52.0 at 800 foot increments. Sections were extended to the protected side of the westbank levee every 1600 feet. Bathymetric data was surveyed using an Odom Echotrac 3200 model. GPS positioning was collected using Leica System 530 GPS receivers in RTK mode. Overbank portions of the transects were collected using the Trimble System 5700 GPS receivers in RTK mode. Drawings of the transect locations and cross section views are shown on the drawing titled "Mississippi River Sections" located in Appendix B. The mud line elevation data obtained from this survey was used for determining borrow/dredge quantities.

2.6 Highway Cross Sections Survey

Cross sections were taken across LA Highway 23 at two proposed dredge discharge pipeline crossings near the intersection with the southern remnant oil field canal (Jefferson Canal). These transect locations are shown in the drawing titled "Highway 23 Transects" located in Appendix B. The highway cross sections are necessary to create a pipe jacking plan for the installation of the dredge discharge pipe under the highway. Due to existing underground utilities in the vicinity of these two site, a new crossing has been proposed (see Section 10.0). The LDNR-CED survey staff will be performing additional surveys along the newly proposed pipeline highway crossing alignment in order to finalize the crossing details.

2.7 Magnetometer Survey

A magnetometer survey of the Mississippi River, fill sites, earthen terrace field and proposed access routes was conducted in attempts to identify potential pipelines and other metallic obstructions in the project area. For the marsh creation area, a GEOMETRIC-858 Cesium magnetometer was used to identify ferrous anomalies. For the Mississippi River segment, a Marine Magnetometer Model G 882 was used to identify ferrous anomalies. Horizontal positions for both locations were determined using the NAVCON GPS system.

The magnetometer survey verified the location of an 18 inch pipeline that runs along the eastern portion of the project area. Additionally, this survey revealed that a smaller line (with a diameter less than 6 inches) may exist in the Jefferson Canal. Other magnetic anomalies were detected. Since these anomalies are located in portions of the project area where no dredging would occur, further investigation was not initiated. A map showing the magnetometer survey lines and the location of the anomalies is located in Appendix B.

2.8 Geophysical Survey

A geophysical survey was performed in the borrow site to compliment the magnetometer survey results, obtain information regarding river bottom material and morphological

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features, and to detect any large underwater obstructions that may exist in the borrow site. This survey was accomplished using the R/V Coastal Profiler (see Figure 2), which is owned and operated by the Louisiana State University Coastal Studies Institute, and consisted of a magnetometer survey, a side-scan sonar survey, and a full spectrum sub-bottom profile survey.

Figure 2 – R/V Coastal Profiler

A Geometrics Model G882 marine cesium magnetometer was used for the magnetometer survey. The magnetometer sensor and associated electronics are housed in a waterproof tow body and is pulled behind the vessel with a tow cable. The system is equipped with Maglog software which allows the operator to receive, display, and manage data from the tow body on a personal computer. The interpretation of the magnetometer data identified eighteen magnetic anomalies within the borrow site. Of these anomalies, three were associated with the pipeline crossings located on the southeastern portion of the borrow site. The remaining anomalies were interpreted to be the result of passing ships or small pieces of metallic debris located on the river bottom.

Side-scan sonar data was acquired using a Klein 2260NV digital dual frequency tow fish with a swath range of 200 meters. The main purpose of this survey was to efficiently map the water bottom to detect any features that may obstruct dredging operations. This is accomplished by measuring the reflection amplitudes from the tow fish to the water bottom. No major underwater objects were detected. This survey did detect a pattern in the river bed known as sand waving, which was more prevalent on the southern portion of the borrow site.

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The sub-bottom profile was obtained using a high frequency chirp system. The R/V Coastal Profiler is equipped with an EdgeTech SB512i tow fish and Model FS 5B Signal Processor. This system sends an acoustic signal towards the river bottom. Since different sediment types reflect the acoustic signal with different strengths, the bottom "hardness" can be interpreted from the amplitude of the signal. The sub-bottom profile data revealed numerous sand waves and poorly defined sub-bottom reflectors. This is typical for water bottom conditions with thick sand layers.

3.0 GEOTECHNICAL ANALYSIS

In order to determine the suitability and physical characteristics of the soils in the BA-42 project area for the proposed project features, a geotechnical investigation and analysis was performed by Eustis Engineering Services, L.L.C. (Eustis) and completed on October 1, 2007. Eustis was tasked to collect soil borings, perform laboratory tests to determine soil characteristics, perform stability analyses on the proposed containment levees, earthen terraces and shoreline protection/restoration features, calculate the settlement of the proposed containment dikes, earthen terraces, and marsh fill sites for different fill elevations, and determine an adequate cut to fill ratio for dredge and fill operations. A detailed summary of the geotechnical investigation and analysis is presented in the Geotechnical Investigation report prepared by Eustis. This document can be made available upon request.

3.1 Field Investigation

A total of thirteen subsurface borings were drilled in the project area during the period of April 12, 2007 through April 30, 2007 at locations shown in Figure 3. Three borings were drilled in the Mississippi River to a depth of 40 feet, six borings were drilled in the interior marsh areas to depths of 40 feet, two borings were drilled in the interior marsh areas to a depth of 60 feet, one boring was drilled in the interior marsh areas to a depth of 100 feet, and one boring was drilled on land (near the proposed dredge pipeline crossing) to a depth of 60 feet. Undisturbed soil samples were obtained with rotary type drilling equipment. For the borings located in the Mississippi River, the drilling rig was mounted on a barge and positioned using anchors and wenches. The borings taken in the interior marsh areas were mounted on a marsh buggy. Soil samples were laboratory tested for classification, strength, and compressibility.
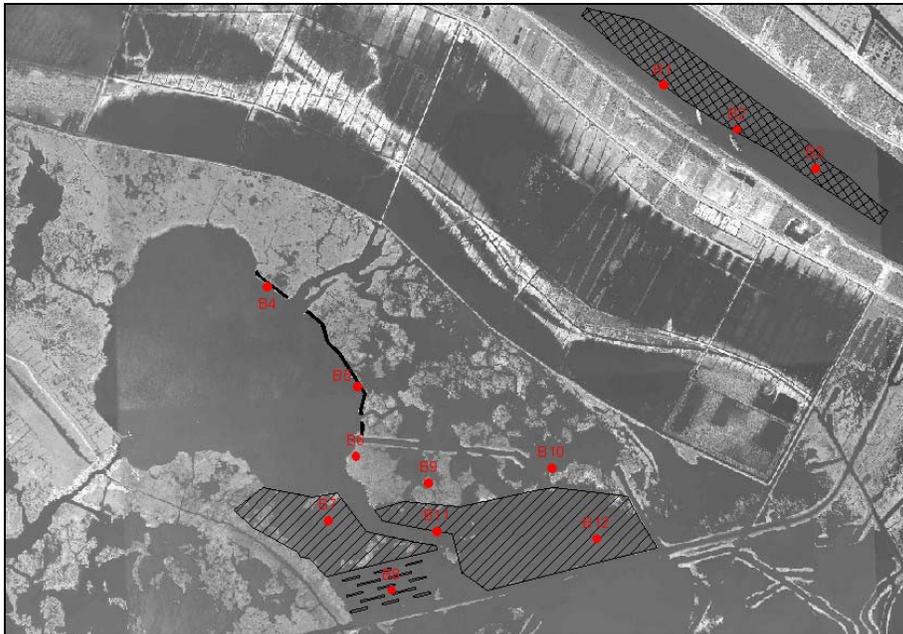

Figure 3 – Soil Borings Locations

3.2   General Subsurface Evaluation

The samples extracted from the Mississippi River borings were relatively consistent, revealing poorly graded sands that varied in density from loose/medium near the surface to dense at approximately 30 feet below the mudline. The samples extracted from borings in and around Lake Hermitage revealed very soft, plastic clays throughout the full depth of the borings. The boring logs for BA-42 can be found in Appendix C.

3.3   Slope Stability Analysis

Slope stability analyses were performed for the earthen containment dikes and the three proposed shoreline protection/restoration alternatives (an offshore rock dike, an onshore rock dike, and an onshore sand fill). The slope stability of any embankment or dike has two types of driving forces: (1) the forces induced by the soil weight, and (2) any seepage forces which tend to cause the soil to slide. In response to these driving forces, the subsurface soils have a resistant force in the form of shear strength, which attempts to keep the slope from sliding. Both the driving forces and the resisting forces are dependant on the geometry of the situation: the "Failure Surface". Eustis utilized the software package PCSTABL to perform this analysis. PCSTABL is two-dimensional limit equilibrium model that utilizes Spencer's Method, which isolates individual blocks of soil and computes the ratio of resisting forces to driving forces. This ratio is known as the global slope stability safety factor.

Eustis performed this analysis for the earthen containment dikes using composite data from the marsh fill soil borings. Containment dike heights of 5.0 feet, 5.5 feet, 6.0 feet, and 6.5 feet with 5.0 feet crown widths were analyzed. It was also assumed that the borrow sites for the containment dikes are located on the interior of the marsh fill cell. Eustis recommended that the toe of the containment dikes be built no closer than 25.0 feet from the edge of the borrow site. A similar slope stability analysis was performed for the terraces. Terrace heights of 4.0 feet, 5.0 feet, and 6.0 feet were analyzed. A crown width of 10.0 feet was used for this analysis. Eustis also recommended that the toe of the terraces be built no closer than 25.0 feet from the borrow site.

Stability analyses and estimation of the failure surfaces were also performed on three shoreline protection/restoration alternatives: an offshore rock dike, an onshore rock dike, and an onshore sand fill feature. The computer program Slope/W, developed by Geoslope International, Ltd., and Spencer's Method of Slices were utilized for these analyses.   Since a flotation channel would be necessary to mobilize barges of rip rap, Eustis recommended that both rock dike features be constructed no closer than 25.0 feet from this channel.

3.4   Settlement Analysis

Settlement analyses for BA-42 were performed using two computer programs: a software package CSETT, developed by the Corps of Engineers, and the program SD3 developed by the University of Texas-Austin. Actual consolidation curves were used in the

calculations for the soil types that required consolidation tests. Both programs implement the Boussinesq stress distribution theory. Published correlations for pre-consolidation pressure, coefficient of consolidation, and compression/re-compression indices were used for other soil types to obtain consolidation indices using shear strength, Atterberg Limits, and moisture content values. Settlement analyses were performed for marsh creation fill sites, the earthen containment dikes, the terraces, and the three shoreline protection/restoration alternatives.

The primary purpose of the settlement analysis in marsh creation design is to determine the target construction fill elevation and the total volume of material required. The final elevation of the marsh fill (at year twenty) is governed by two forms of settlement: (1) The settlement of the underlying soils in the fill cells caused by the loading exerted by the dredged material, and (2) the self-weight consolidation of the dredged material (see Figure 4).
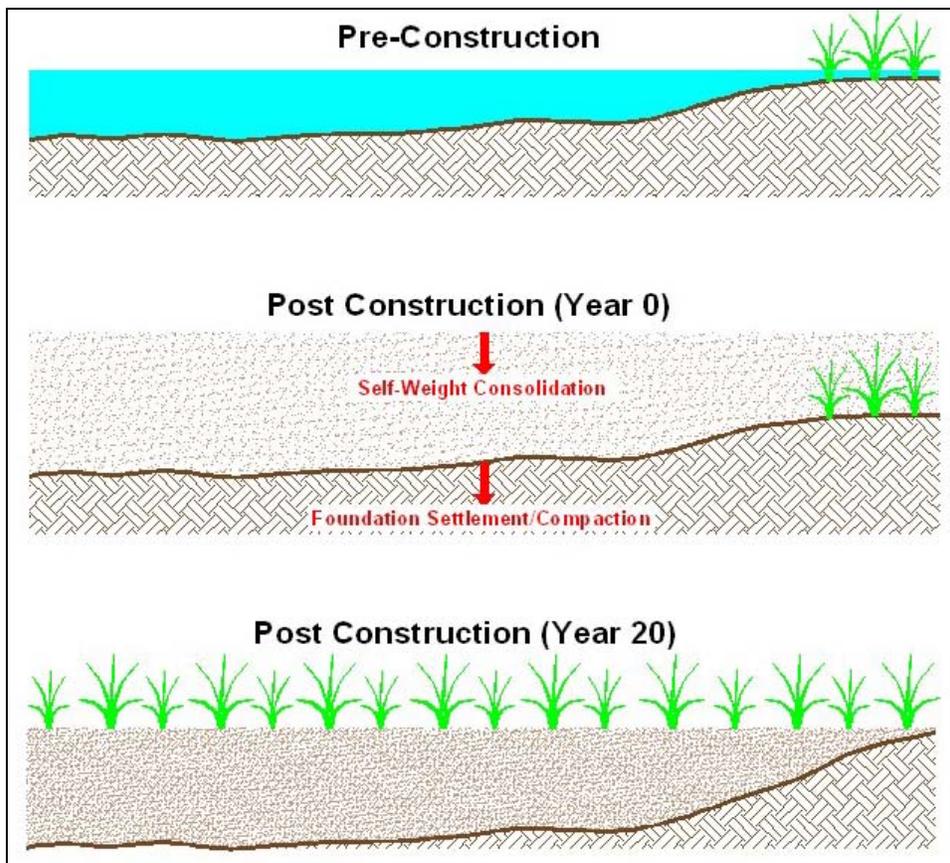
Figure 4 – Marsh Fill Settlement

Data from low pressure consolidation tests was used to calculate the time-rate of settlement of the underlying soils of the fill cells. Self-weight consolidation tests were performed on a composite sample of the borrow site (Mississippi River) material to determine the consolidation of the dredged material after placement. Eustis' settlement and consolidation analyses were performed at fill heights of 3.5 feet, 4.0 feet, 4.5 feet, and 5.0 feet. It was assumed that dredging/filling operations would take place over a one

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year period. Figure 8 of Appendix D includes the Eustis marsh fill settlement curves which show the total time-rate settlement (includes self-weight consolidation) for each assumed fill height.

Settlement calculations for the earthen containment dikes were based on the soil conditions of various borings in the fill sites (see Appendix C for boring logs). An assumed water depth of 1.0 foot was used throughout the project area and dike heights 5.0 feet, 5.5 feet, 6.0 feet, and 6.5 feet were evaluated.   A containment dike construction period of sixty days was also assumed. The purpose of this analysis was to produce a dike height that would maintain an elevation of +3.0 feet NAVD 88 throughout the duration of construction. Figure 11 of Appendix D includes the Eustis containment dike settlement curves which show the time rate settlement for each assumed fill height. A similar settlement analysis was performed for the terrace design with assumed heights of 4.0 feet, 5.0 feet, and 6.0 feet. The purpose of this analysis was to produce a terrace template that would maintain an elevation of +2.3 feet NAVD 88 throughout the twenty year life of the project.

Eustis' settlement analysis for BA-42 also included an estimation of the magnitude of settlement for the rock dike and sand fill features. This analysis assumed that a geotextile separator fabric will be placed beneath these features. Since the calculated breaking wave height of +2.3 feet NAVD 88 is the governing design criteria for the shoreline protection/restoration feature, Eustis assumed the templates for the shoreline protection/restoration features maintain an elevation of +2.3 feet NAVD 88 throughout the twenty year life of the project.

3.5   Results/Recommendations

Marsh Fill:
(see Figure 8 in Appendix D)

Containment Dikes:
- Crown Elevation: +5.5 feet NAVD 88
- Side Slope: 1(V):6(H)
- Crown Width: 5.0 feet
- Accepted Safety Factor: 1.3

Earthen Terraces:
- Crown Elevation: +3.6 feet NAVD 88
- Side Slope: 1(V):3(H)
- Crown Width: 20.0 feet
- Accepted Safety Factor: 1.6

Rock Dike (onshore/offshore):
- Crown Elevation: +3.5 feet NAVD 88
- Side Slope: 1(V):3(H)

- Crown Width: 4.0 feet
- Accepted Safety Factor: 1.3
- Magnitude of Settlement: 1.0 foot (over twenty years)

Sand Berm:
- Crown Elevation: +4.2 feet NAVD 88
- Lakeside Side Slope: 1(V):50(H)
- Marshside Side Slope: 1(V):25(H)
- Crown Width: 50.0 feet
- Accepted Safety Factor: 2.1
- Magnitude of Settlement: 1.5 feet (over twenty years)

3.6   Cut:Fill Ratio Recommendations

LDNR tasked Eustis to determine an estimated cut:fill ratio for BA-42. Two cases were considered in this analysis: (1) the quantity of in-situ borrow material necessary to construct the containment dikes using mechanical dredging techniques, and (2) the quantity of material that will be dredged hydraulically from the Mississippi River and placed in the fill cells. The cut:fill ratio for mechanical dredging was primarily based on the expected transport losses during construction and desiccation of the clayey material in the project area and consolidation of the material under its own weight. With these factors in mind, Eustis recommended a 2:1 cut:fill ratio for mechanical dredging (for containment dikes and terraces). The primary losses associated with hydraulic dredging for marsh creation will result from containment failure, leaking pipelines, and losses during dewatering of the fill sites. With these factors in mind, Eustis recommended that a cut:fill ratio between 1.25:1 and 1.5:1 be used for BA-42. Additionally, the project team assumed that the magnitude of losses would be greater during construction of the sand fill shoreline restoration and the earthen plug due to the unconfined placement of the dredged material. Therefore, a cut:fill ratio of 1.7:1 will be used for the shoreline restoration feature and a cut:fill ratio of 3.0:1 will be used for the earthen plug.

4.0 WIND ANALYSIS

The New Orleans Naval Air Station (Alvin Calendar Field), located approximately 10 miles south of New Orleans, LA, in Belle Chase, was selected to gather historical wind data due to availability and close proximity to the project area. Based on statistical analysis of the hourly wind data available from 1997 to 2003, and the orientation of the Lake Hermitage shoreline, it was determined that the BA-42 wind analysis would only be based on wind directions from 140° to 330° clockwise from north (see Figure 5).

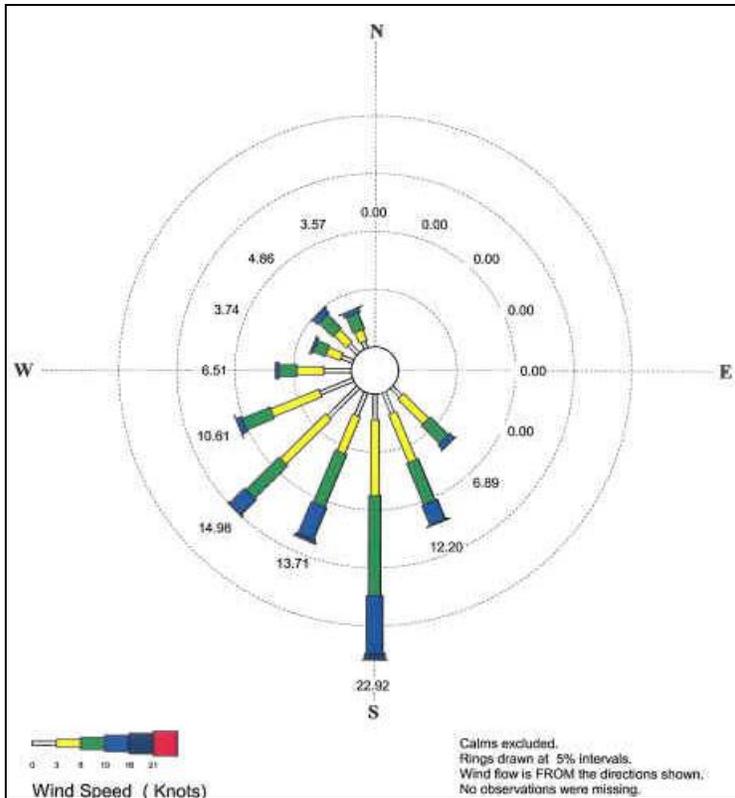**Figure 5 – Wind Rose for New Orleans Naval Air Station, 1993-2000**

A statistical analysis was performed and the 90th percentile wind speed was determined to be 14.0 miles per hour (12.0 knots). The 90th percentile wind direction associated with the 90th percentile wind speed was calculated to be 204° clockwise from north. This wind speed and direction were chosen for the design wave described in the analysis below.

5.0   HYDRAULIC ANALYSIS

5.1   Tidal Datum

LDNR monitoring gage BA04-17 was selected to determine historical water levels due to its proximity to the project area and database availability. This gage is located at 29°31'14.20"N, 89°49'27.87"W, approximately 3 miles southeast of the project area. Hourly water level data was recorded from August 13, 1997 to June 22, 2000.

A normal tidal epoch lasts approximately 19 years. In order to accurately estimate Mean High Water (MHW) and Mean Low Water (MLW) elevations, a data set which has less than 19 years of data should be correlated to a gage which has data from a full tidal epoch using a technique known as the Range-Ratio method. NOAA station #8761724 located at Grand Isle, Louisiana near Barataria Pass at 29°15'48"N, 89°57'24"W was used as the control station for making this correlation. The period of record used for the nineteen (19) year tidal epoch was from January 1, 1985 to December 31, 2003. The results of the tidal datum determination for BA-42 are shown in Table 2. A more detailed summary of how this tidal datum was calculated is shown in Section I of the Design Calculation Packet located in Appendix E.

KNOWN VARIABLES	ELEV. FT NAVD 88
MHWc = 19 YEAR MEAN HIGH WATER AT CONTROL STATION	1.37
MTLc= 19 YEAR MEAN TIDE LEVEL AT CONTROL STATION	0.85
MLWc = 19 YEAR MEAN LOW WATER AT CONTROL STATION	0.32
MRc = 19 YEAR MEAN TIDE RANGE AT CONTROL STATION	1.05
TLc = MEAN TIDE LEVEL FOR THE OBSERVATION PERIOD AT CONTROL STATION	0.95
Rc = MEAN TIDE RANGE FOR THE OBSERVATION PERIOD AT CONTROL STATION	0.92
TLs = MEAN TIDE LEVEL FOR THE OBSERVATION PERIOD AT SUBORDINATE STATION	0.71
Rs = MEAN TIDE RANGE FOR THE OBSERVATION PERIOD AT SUBORDINATE STATION	0.48
CALCULATED VARIABLES	ELEV. FT NAVD 88
MHWs = 19 YEAR MEAN HIGH WATER AT SUBORDINATE STATION (MHWs=MTLs+MRs/2)	0.88
MTLs= 19 YEAR MEAN TIDE LEVEL AT SUBORDINATE STATION (MTLs=TLs+MTLc-TLc)	0.61
MLWs = 19 YEAR MEAN LOW WATER AT SUBORDINATE STATION (MLWs=MTLs-MRs/2)	0.34
MRs = 19 YEAR MEAN TIDE RANGE AT SUBORDINATE STATION (MRs=(MRc*Rs)/Rc)	0.54

Table 2 – Summary of Tidal Datum Determination

5.2   Setup

The wave setup is defined as the difference in still-water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water. This was factored into the wave height calculation to obtain the absolute wave height. The setup for Lake Hermitage was determined using the 90th percentile water and wave conditions from the historical records. The average recorded water level associated with the 90th percentile wind speed and direction is 1.06 feet (0.323m) NAVD88. This value minus the mean high water level yields a setup of 0.18 feet (0.055 m). Section II of the

Design Calculation Packet located in Appendix E includes the spreadsheet used to calculate the setup.

5.3 Deep Water Wave Hind Casting

Bathymetric data for Lake Hermitage was collected during the original survey. From this data, the average depth of Lake Hermitage is 4.6 feet (1.4 meters) deep. Three wave cases were analyzed as shown in Figure 6. The longest fetch associated with the 90th percentile wind direction is 0.90 miles or 4,744 feet (Case 1). The worst case wind-generated wave is Case 3, which includes a fetch distance of 1.35 miles or 7,130 feet.
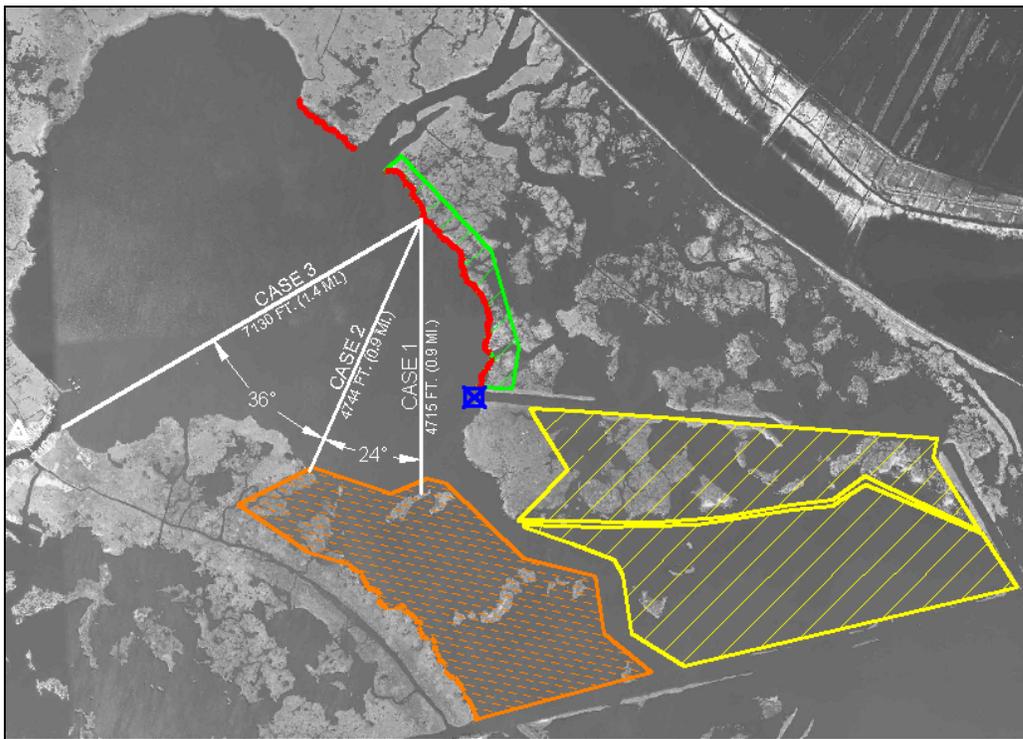
Figure 6 – Fetch Scenarios for Wind Generated Wave

Using the U.S. Army Corps of Engineers Coastal Engineering Manual (USACE CEM), the (Case 3) deep water wave height and period was determined to be 0.46 feet (0.139 meters) and 1.122 seconds, respectively (Interactive CEM, Equations II-2-35 to II-2-38). The values for the deep water wave height from these equations are relative to still-water elevation and represent the wave profile from crest to trough. The deepwater waves generated were not fetch or shallow water limited.

For this design, the components of the absolute deep water wave height include the setup, mean high water level, and relative deep water wave height. Therefore, the absolute deep water wave height (H_{abs} = setup + MHW + H/2) is 0.18 ft + 0.88 ft + 0.23 ft = 1.29 ft NAVD88.

5.4 Wave Transformation

As a deep water wave propagates shoreward along increasing bathymetry, it loses energy and height due to frictional forces. These frictional forces are caused by the reflection and refraction of the wave with the bottom surface. Calculations were performed based on the methodologies in Chapter II of the USACE CEM to determine the height of the 90th percentile wind generated wave in deep water as it is transformed onshore at the BA-42 project area (see Table 3). It was determined that the 90th percentile wind generated wave would break between the 0.0 and 1.0 foot NAVD88 contours assuming an initial wave reflectivity angle of 0 degrees.

Contour	Wave Height		
(ft NAVD88)	H/2 (ft)	Water Type	h_{mhw}+Setup+H/2 (ft NAVD88)
-7	0.23	Transitional	1.29
-6	0.23	Transitional	1.29
-5	0.23	Transitional	1.29
-4	0.23	Transitional	1.29
-3	0.23	Transitional	1.29
-2	0.23	Transitional	1.29
-1	0.22	Transitional	1.28
0	0.21	Transitional	1.27
1	0.02	Shallow	1.09

Table 3 - Deep Water Wave Transformation

The details of this analysis are shown in Section II of the Design Calculations Packet located in Appendix E.

5.5 Wave Run-up

The maximum height to which a breaking wave will run up onto the shoreline cannot be calculated using current methodologies. Instead, in order to remain conservative, the minimum breakwater height required to provide protection against the 90th percentile wind generated and breaking wave is taken as the sum of the setup, mean high water level and the wave height corresponding to the design contour. According to Table 3 approaching waves on the eastern shoreline of Lake Hermitage will break when they reach the -1.0 foot NAVD88 contour. For this contour the highest 90th percentile breaking wave height along the project is calculated to be approximately +2.3 feet NAVD88 (0.18 feet NAVD 88 + 0.88 feet NAVD 88 + 1.28 feet NAVD 88). The crown height of the chosen shoreline protection/restoration feature must maintain this elevation in order to provide optimum performance throughout the twenty year design life of the project. To remain conservative, this elevation was also chosen for shoreline protection/restoration features that could be constructed closer inland (on shore) than the -1.0 foot NAVD 88 contour.

6.0   MARSH CREATION DESIGN

This project proposes to create marsh by dredging sediment from the Mississippi River for placement into the designated sites shown in Figure 1 and the Preliminary Design Drawings located in Appendix F. The marsh creation design was separated into three components: the marsh creation fill sites, the dredge borrow site, and the containment dikes. The design and analysis of each component is discussed in the sections below.

6.1   Fill Site Design

The primary goals of the Marsh Creation features are to address the widespread marsh loss in this area and to reestablish the southern shoreline of Lake Hermitage. These goals governed the configuration of the fill cells, which does not vary significantly from the original Phase 0 (planning level) layout. Another factor that contributed to the layout was the presence of an historic bayou that ran between Fill Site A and Fill Site B (see Figure 7). Due to a Louisiana State Water Bottoms statute, and in an effort to improve the natural hydrology throughout the project area, the Project Team decided to restore the alignment of this small channel.
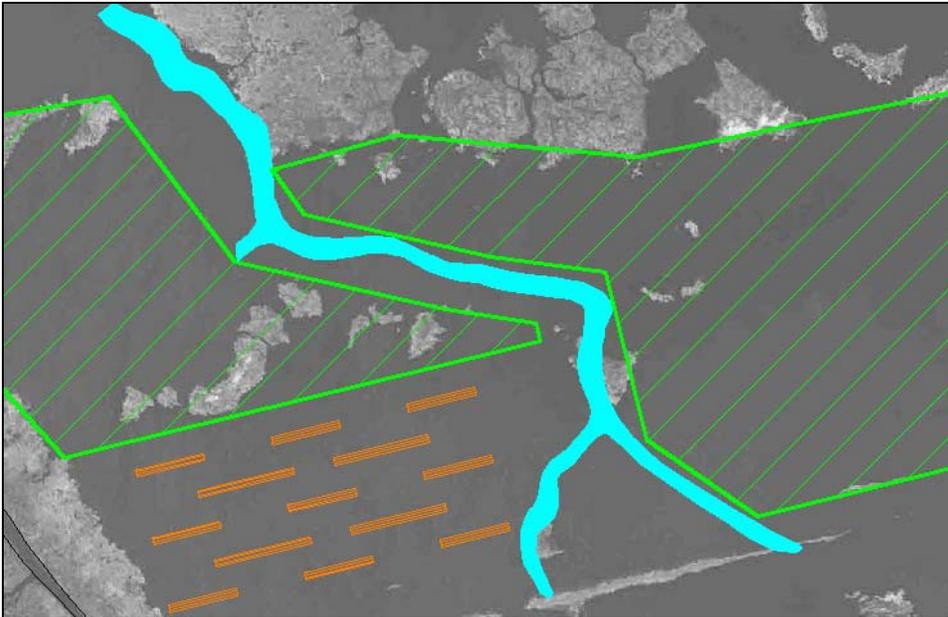
Figure 7 – Alignment of Historic Bayou

In addition to the fill site configuration, a key design component of BA-42 involves the calculation of the fill site volumes. Before this could be accomplished, a target construction fill elevation had to be determined. This elevation was governed by several factors including average healthy marsh elevation, the tidal datum, the physical properties of the borrow material, and the bearing capacity of the foundation soils in each fill site.

The first step of the target fill elevation design involved an examination of the existing marsh conditions. The average marsh elevation survey performed during the Fill Site

Survey revealed that the average marsh elevation of the existing healthy marsh locations was approximately +1.2 feet NAVD 88 (see Section 2.4 for additional details). The calculated tidal datum (MHW=+0.88 feet NAVD 88, MLW=+0.34 feet NAVD 88) discussed in Section 5.1 verified that the existing marsh predominantly fell above the portion of the project inter-tidal zone; the range of elevations that lie between the upper and lower extents of the tidal datum. Since scientists from both USFWS and LDNR preferred the created marsh to be as close as possible to the existing marsh conditions, the project team decided that the criteria for the determination of a twenty year target elevation would be existing healthy marsh elevation. To achieve a sustainable marsh elevation throughout the life of the project, the marsh platform will initially be pumped to a higher elevation during construction and allowed to settle to the desired target elevation over time.

In order to determine the construction fill elevation, LDNR tasked Eustis to perform consolidation settlement calculations for boring locations B-7, B-8, B-11, and B-12, which are located in marsh fill sites. The calculations were performed for potential fill heights of 3.5 feet, 4.0 feet, 4.5 feet, and 5.0 feet. The purpose of these analyses was to assist in the determination of a construction fill elevation that would be as close as possible to the existing marsh elevation after twenty years. The marsh fill consolidation curve produced by Eustis (Figure 8 in Appendix D) indicates that placement of 4.0 feet of fill (to a target fill elevation of +1.8 feet NAVD 88) would ultimately settle to an approximate elevation of +1.3 feet NAVD 88. For constructability purposes, a target fill elevation of +2.0 feet NAVD 88 was chosen for the BA-42 fill sites. The settlement values are composed of foundation settlement and self-weight consolidation. Due to the composition of the dredged material from the Mississippi River, self-weight consolidation settlement is anticipated to occur instantaneously and be on the order of 2-3 inches.

Once the target fill elevation was determined, the marsh fill volumes were calculated. As shown on the Preliminary Design Drawings located in Appendix F, the marsh creation portion of the project is broken up into two fill sites, each analyzed separately. Cross-sectional areas of the transects in each fill site were calculated using the data produced by the Fill Site Survey described in Section 2.2. Fill site volumes were then computed using these cross-sectional areas. Table 4 summarizes the results of the volume calculations for each fill site. A more detailed summary of the fill site design is provided in Section III the Design Calculation Packet located in Appendix F.

FILL SITE	AREA (acres)	VOLUME OF FILL (yd^3)
A	352	2,531,259
B	182	1,194,525
Totals	534	3,725,784

Table 4 – Summary of Marsh Creation Volumes and Acreage

6.2   Borrow Site Design

The controlling factors of this design component include the borrow site location and the size of the borrow site (acreage and depth), as well as USACE restrictions. The borrow site must contain sufficient sediment to meet the calculated marsh fill volume requirements. The following is a list of USACE physical borrow site restrictions:

- All excavations must be at least 750 feet from any protection levee centerline;
- Borrow sites must be outside the USACE maintained navigation channel;
- Excavation in the river must not be made less than 4,000 feet upstream of a bridge crossing;
- The side slopes of the borrow site must be no steeper than 1(V):5(H); and
- The excavation must proceed from landside to riverside limits to minimize the possibility of overburden failure of the bank.

The location for the borrow site was chosen to be between Mississippi River Miles (RM) 49.5 and 52. This stretch of the river is located near the marsh fill site and is shallow enough to be reached using a large hydraulic dredge. Immediately upstream and downstream of this section, the water depths are too great to be dredged by a conventional dredge. This borrow site contains sufficient sediment for the marsh fill sites. Additionally, areas near or adjacent to concrete revetment mats were avoided.

The western boundary of the borrow site is delineated by a 750 foot offset from the centerline of the Mississippi River levee. This boundary exists to ensure that a 1.3 factor of safety remains for the slope stability of the Mississippi River levee. If the elevation of the protected side of the river levee is greater than the elevation of the river side, the elevation of the protected side must be projected towards the river to intersect the 750' offset line. At this intersection, side slopes of 1(V):5(H) are projected toward the river to the intersection of the mudline. This is the point dredging is permitted to begin. A cross-sectional diagram of this USACE regulation is shown in Figure 8 below.
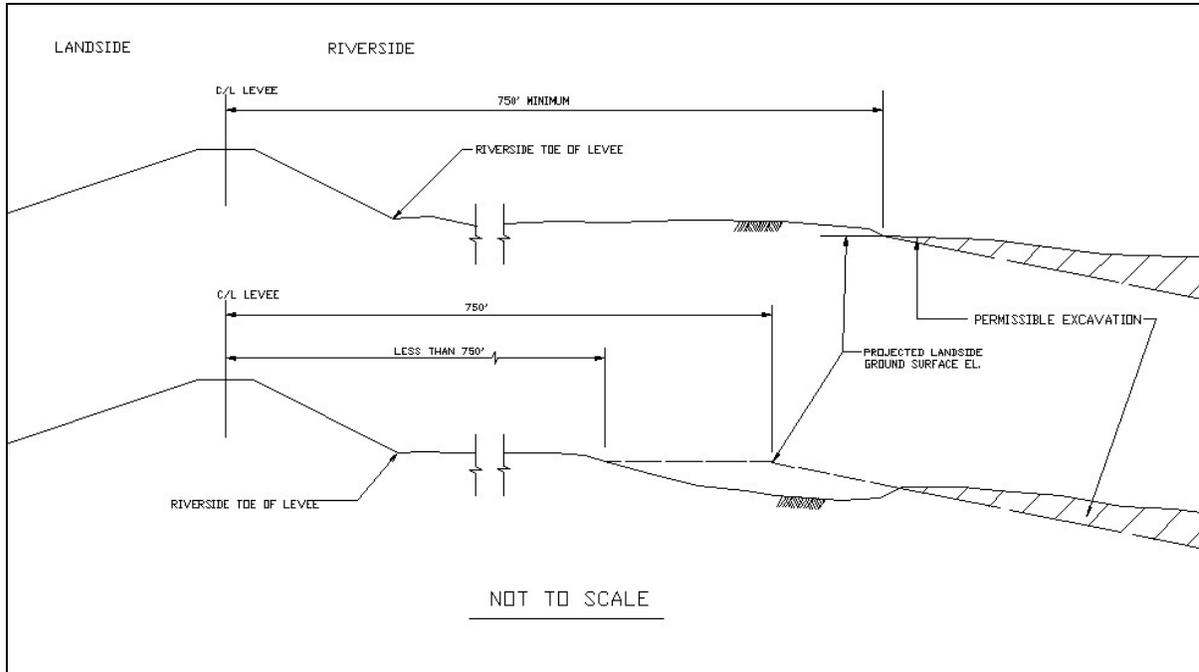
Figure 8 – USACE Mississippi River Dredging Regulations

In this stretch of river, the navigation channel is located near the eastern bank, delineating the eastern boundary of the borrow site. There is no bridge within 4,000 feet of this area. Figure 9 shows the general plan view of the borrow site. There is minimal revetment along the western bank of this river section. The eastern bank of the river is reveted, but is not an issue for this project. Although the magnetometer surveys indicated the borrow site is free of known pipelines, the contractor will be required to perform a magnetometer survey prior to excavation.

Figure 9 – Designated Borrow Site

The size of the borrow site is governed by the volume of material calculated to fill the marsh creation fill sites as discussed in Section 6.1. The borrow volume is computed by simply multiplying the fill volume by the cut:fill ratios for hydraulically dredged material mentioned in Section 3.6. The maximum depth of cut is assumed to be elevation -66.0 feet NAVD88. A conventional dredge can cut to a maximum of approximately 70.0 feet below the water surface. Historical water surface elevation data in the Mississippi River at Alliance and Venic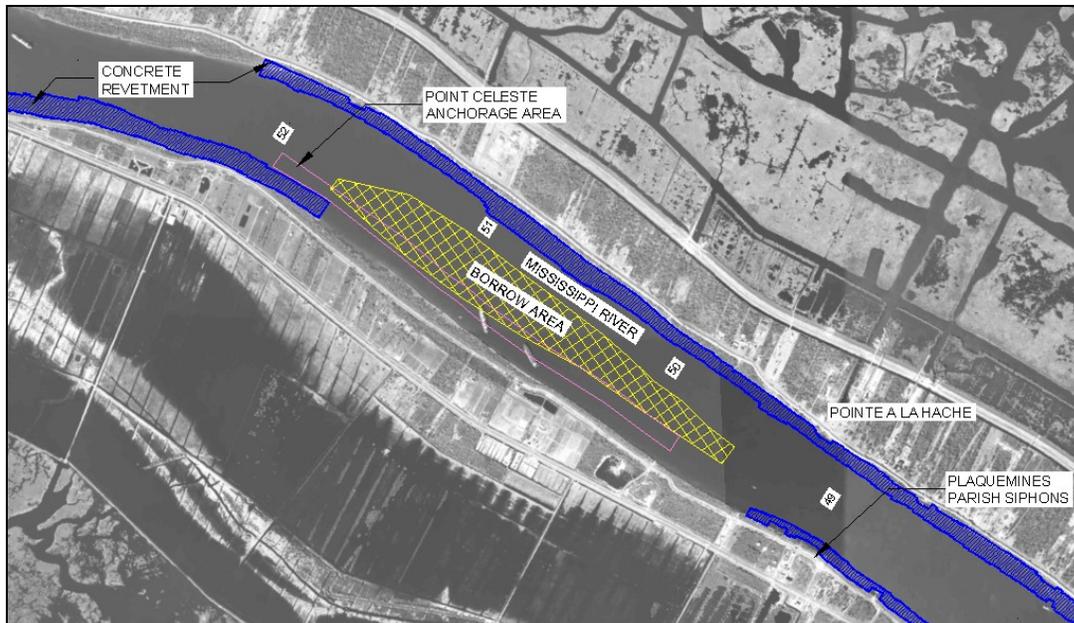e shows that the water elevation in the summer typically fluctuates between +3.0 feet NAVD 88 and +4.0 feet NAVD 88. Since this is the most likely time for the material to be dredged, the maximum depth of cut was estimated to be -66.0 feet NAVD88 to account for the water level elevation of +4.0 feet NAVD88. The total volume of available sediment in this reach of the river is 6,247,664 cubic yards. The total fill volume required is 5,251,841 cubic yards, (including refilling containment dike borrow sites). Details on the borrow site design are shown in Section VII of the Design Calculations Packet located in Appendix E.

6.3 Containment Dike Design

The primary design parameters associated with the containment dike design include crown elevation, crown width, and side slopes. LDNR tasked Eustis to determine these parameters using slope stability and settlement analyses. Eustis recommended that the containment dikes be built to a +5.5 feet NAVD 88 crown elevation, with a 5 feet crown width and 1(V):6(H) side slopes to maintain a factor of safety of 1.3. Constructing the dikes to a crown elevation of +5.5 feet NAVD 88 should insure that an elevation of +3.0 NAVD 88 is maintained throughout construction (assumed to be one year). This recommendation is based upon the assumption that the marsh buggy excavator contractor would demobilize his equipment once the construction of the containment dikes is complete. LDNR construction specifications require that the contractor maintain the containment dikes throughout construction. After discussing the matter with Eustis, it was decided that an initial constructed and maintained crown elevation of +3.0 feet NAVD would suffice. The containment dikes shall be constructed using in-situ material from within each fill site. Once these parameters were determined, cross-sectional areas and containment volumes were calculated using the methods described in Section IV of the Design Calculations Packet located in Appendix F. As recommended by Eustis, a mechanical dredging cut:fill ratio of 2.0:1 was applied to the calculated fill volumes. Table 5 provides the approximate segment lengths and volumes of each containment dike segment.

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Segment	Avg. Base Elevation (ft. NAVD 88)	Avg. Height (ft.)	Segment Length (ft.)	Fill Volume (yd³)	Borrow Volume (yd³)
A1	-1.85	4.85	972	5951	11902
A2	-1.27	4.27	5382	26061	52122
A3	0.94	2.06	1824	2416	4832
A4	-0.60	3.60	2269	8048	16096
A5	-0.26	3.26	2971	8811	17623
A6	0.39	2.61	1727	3449	6898
A7	-0.77	3.77	925	3567	7133
A8	-2.55	5.55	398	3131	6262
A9	-2.50	5.50	2202	17043	34086
A10	-1.06	4.06	1234	5450	10899
B1	-1.66	4.66	3511	19971	39942
B2	-1.65	4.65	133	753	1506
B3	-1.35	4.35	659	3303	6605
B4	-1.68	4.68	1549	8884	17767
B5	-2.05	5.05	1484	9796	19593
B6	1.00	2.00	755	950	1901
B7	-2.43	5.43	1545	11678	23355
B8	0.02	2.98	1140	2878	5756
B9	-0.36	3.36	764	2392	4783
B10	0.56	2.44	1573	2791	5583
B11	-0.18	3.18	216	614	1227
B12	-1.04	4.04	261	1143	2286
B13	-1.04	4.04	776	3393	6787

Table 5 – Summary of Containment Dike Quantities

7.0 EARTHEN TERRACE DESIGN

Another proposed project feature is to create approximately 7,300 linear feet of earthen terraces by excavating material from adjacent borrow sites as shown in the Preliminary Design Drawings located in Appendix F. The terrace field will play a major role in dampening the wave action in the project area and should provide ample area for marsh and vegetation to establish itself. The design and analysis of the terraces is discussed in the sections below.

7.1 Terrace Design

The main design component of the terrace field involves the establishment of the terrace template. Before this could be accomplished, a design elevation must be determined. The construction crown elevation of a terrace is typically governed by the significant wave height, the physical properties of the borrow material, and the bearing capacity of the foundation soils in the terrace area. Although the construction of the adjacent marsh creation cells would eliminate most wave action at the proposed terrace field location, the calculated wave height of +2.3 feet NAVD 88 was chosen as the governing factor in the terrace template design. Eustis' recommended construction crown elevation of +3.6 feet NAVD 88 was chosen in order to maintain a design elevation of +2.3 feet NAVD 88 throughout the twenty year life of the project.

The terrace layout includes an overlapping configuration with 500 foot spacing between terrace rows. Per Eustis' recommendation, the terrace borrow sites should be located at least 25.0 feet from the southern toe of each terrace. The borrow site dimensions include a cut depth of 10.0 feet, a 1(V):3(H) side slope, and a bottom width of 2.0 feet. Cross sectional areas and volumes were calculated based on these parameters using the methods described in Section V of the Design Calculations Packet located in Appendix E. As recommended by Eustis, a mechanical dredging cut:fill ratio of 2.0:1 was applied to the calculated fill volumes. Table 6 shows the approximate lengths and volumes of each terrace.

Terrace	Avg. Base Elevation (ft. NAVD 88)	Avg. Height (ft.)	Terrace Length (ft.)	Fill Volume (yd^3)	Borrow Volume (yd^3)
T1	-1.92	5.42	500	3639	7279
T2	-2.20	5.70	500	3916	7832
T3	-2.08	5.58	500	3796	7593
T4	-2.01	5.51	700	5218	10437
T5	-2.15	5.65	700	5412	10825
T6	-2.35	5.85	500	4068	8136
T7	-2.03	5.53	500	3747	7494
T8	-2.08	5.58	500	3796	7593
T9	-1.99	5.49	700	5191	10382
T10	-1.92	5.42	700	3639	7279
T11	-2.42	5.92	500	4140	8279
T12	-1.67	5.17	500	3400	6800
T13	-1.78	5.28	500	3504	7009

Table 6 – Summary of Terrace Quantities

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7.2   Terrace Construction

Because of the relatively high water depths and poor soil conditions, construction of the BA-42 terrace field will require special provisions. Based on the LDNR's experience with terrace construction, the design template could be obtained if the contractor utilizes a staged construction process. This is accomplished by first building the terrace to just above MLW elevation and allowing it to dewater. Once this "base" has been established, the remainder of the terrace template can be constructed. Several lifts are anticipated to obtain the desired elevation of +3.5 NAVD 88. Once constructed, a new contract would be issued for terrace planting.

8.0   SHORELINE PROTECTION/RESTORATION DESIGN

8.1   Design Alternatives

Several shoreline protection/restoration alternatives were considered for the eastern rim of Lake Hermitage. An offshore rock dike, a rock dike placed on the shoreline, and sand fill template placed on the shoreline. The shoreline protection feature proposed in Phase 0 (planning) included the placement of 6,000 feet of rip rap along the eastern shoreline of Lake Hermitage. Since Lake Hermitage has an average water depth of 4.6 feet, it is anticipated that approximately 2.6 miles of access channel would have to be dredged to mobilize barges of rock to the project site. Additionally, the project team concluded that the relatively mild wave climate in Lake Hermitage did not warrant the construction of a "hard" shoreline protection feature. The Project Team then investigated hydraulically pumping sand fill onto the shoreline to restore the degraded shoreline. This feature would not require the contractor to dig access, and would not result in a significant increase in the mobilization cost of the project as the rock feature would.   The geotechnical analysis indicated that the sandy material in the borrow site would be suitable for constructing this type of feature. For these reasons the Project Team elected to move forward with the shoreline restoration feature.

The shoreline restoration feature will consist of sand fill template placed along the existing shoreline. The shoreline restoration template will be designed to maintain its integrity against the design wave based on the twenty year design life of the project. The approximate materials quantity calculations are shown in Section VI of the Design Calculation Packet in Appendix E.

8.2   Typical Cross Section

The shoreline restoration template parameters recommended by Eustis were utilized for the design of this feature. These parameters include a crown width of 50 feet, a lakeside slope of   1(V):50(H), and a marshside slope of 1(V):25(H). To insure that a crown elevation of +2.2 feet NAVD is maintained throughout the twenty year life of the project, Eustis recommended the shoreline restoration template be constructed to a crown elevation of +4.2 feet NAVD 88. For constructablity purposes, a crown elevation of +4.0 feet NAVD will be used. The typical cross section for the shoreline restoration feature is shown in Figure 10.
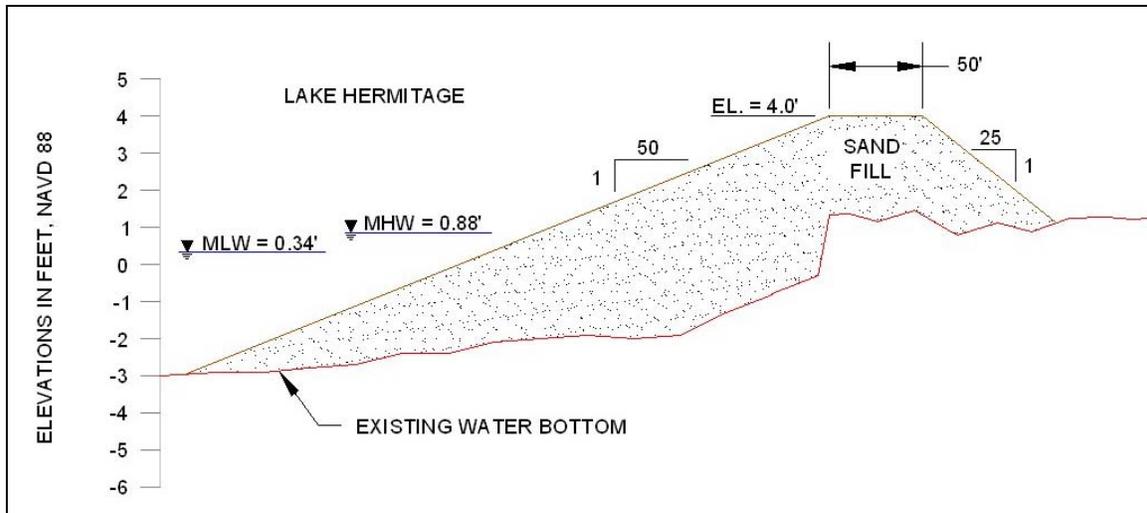
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Figure 10 – Typical Shoreline Restoration Section

8.3   Shoreline Restoration Alignment

Design surveys revealed that a small ridge with an average crown elevation of +0.9 feet NAVD 88 exists along the eastern shoreline of Lake Hermitage. The alignment of the shoreline restoration template places its centerline onto the centerline of the existing shoreline ridge.   The shoreline restoration feature will be constructed in strait line segments. These straight line segments will create a more efficient alignment for the beach to protect against wave energies. Additionally, construction surveying and stake out will also be more uniformly facilitated using straight line segments. The plan view for the alignment of the shoreline restoration feature is shown on the Preliminary Design Drawings located in Appendix F. Once constructed, a new contract would be issued for plantings on the new shoreline template.

9.0   EARTHEN PLUG DESIGN

In order to restore the hydrology of the eastern portion of the project area and to help direct West Point a la Hache Siphon waters (from the Mississippi River) towards the degraded marshes, a plug was proposed at the location canal north of the Fill Site A. Design surveys revealed that the maximum bottom elevation of the proposed plug location was on the order of -7.2 feet NAVD 88. The originally proposed feature included the construction of a steel sheetpile wall at this location. The rising costs of steel, mobilization costs and access for sheetpile driving equipment, and the availability of sand from the Mississippi River lead the Project Team to change this feature to an earthen plug. The plug will be constructed using dredged slurry from the Mississippi River Borrow Site. The parameters shown in the Preliminary Design Drawings in Appendix F were based on assumptions and comparison to similar project features from past projects. A stability analysis and settlement estimation will be performed by the LDNR-CED staff after the Preliminary Design Phase is complete.

10.0   DREDGE PIPELINE TRANSPORT

The dredge slurry discharge pipeline will cross the Mississippi River levee in the Plaquemines Parish tract of land surrounding the West Pointe a la Hache siphon. A suitable levee crossing shall be built as per the USACE's requirements shown in the Preliminary Design Drawings located in Appendix F. A casing will be installed underneath Highway 23 in this tract of land in accordance with all Louisiana Department of Transportation and Development specifications. From Highway 23, the pipeline will be placed on Plaquemines Parish property until it reaches the outfall of the West Pointe a la Hache siphon. It will then run parallel with the pipeline canal indicated in the figure below. Figure 11 shows the proposed pipeline route in the vicinity of the Mississippi River Levee and Highway 23.

Figure 11 – Proposed Pipeline Crossing

11.0 CONSTRUCTION COST ESTIMATE

Lake Hermitage Marsh Creation		Date:	11-Dec-07	Revised:	22-Jul-08
Computed by: Rudy Simoneaux, E.I.		*Project Priority List 15*			
Item	Work or Material	Quantity	Unit	Unit Cost	Amount
1	Mobilization/Demobilization	1	LS	$2,763,251	$2,763,251
2	Construction Surveys	1	LS	$300,000	$300,000
3	Grade Stakes and Flagging	84	EACH	$500	$42,000
4	Hydraulic Dredging for Marsh Creation	4,843,519	CY	$4.00	$19,374,076
5	Hydraulic Dredging for Shoreline Restoration	390,231	CY	$4.00	$1,560,924
6	Shaping Grading/Earthwork-Shoreline Restoration	1	LS	$100,000.00	$100,000
7	Vegetative Plantings for Shoreline Restoration	5,487	LF	$2.00	$10,974
8	Geotextile Separator Fabric (non-woven)	283,425	SY	$4.00	$1,133,700
9	Earthen Containment Dikes	34,268	LF	$28.62	$980,750
10	Earthern Terraces	7,300	LF	$45.18	$329,814
11	Vegetative Plantings for Earthen Terraces	7,300	LF	$4.00	$29,200
12	Hydraulic Dredging for Earthen Plug	18,090	CY	$4.00	$72,360
13	Shaping and Grading/Earthwork-Earthen Plug	1	LS	$10,000.00	$10,000
14	Marsh Fill Settlment Plates	6	EA	$2,500.00	$15,000
15	Jack and Bore Highway	200	LF	$600	$120,000
16	Jacking Pit	1	EA	$30,000	$30,000

ESTIMATED CONSTRUCTION COST   **$26,872,049**
ESTIMATED CONSTRUCTION + 15% CONTINGENCY   **$30,902,856**

12.0   MODIFICATIONS TO APPROVED PHASE 0 PROJECT

As a result of Phase 1 activities, the approved Phase 0 project has undergone some minor modifications.   The Phase 0 project included 593 acres of marsh creation and nourishment. The Phase 1 project includes 534 acres of marsh creation. Additionally, the Phase 0 project included a 300-acre terrace field with approximately 16 subaerial acres. A 182-acre portion of that terrace field was replaced with a marsh creation cell to reestablish the southern shoreline of Lake Hermitage. The Phase 1 terrace field consists of 107 acres with approximately 6.5 subaerial acres. The foreshore rock dike proposed at Phase 0 has been replaced with the shoreline restoration/sand fill feature discussed in Section 9.0.

13.0   REFERENCES

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