ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION

ME-18 CWPPRA Priority List 10
LDNR Contract No. 2511-05-08

FINAL DESIGN REPORT

Submitted to

THE LOUISIANA DEPARTMENT OF NATURAL RESOURCES

Submitted by

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ACRONYMS

ACI	American Concrete Institute
ACRE	Applied Coastal Research and Engineering, Inc.
AISC	American Institute of Steel Construction
Chance	John Chance Land Surveyors, Inc.
CUR	Centre for Civil Engineering Research and Codes
CWPPRA	Coastal Wetlands Planning Protection and Restoration Act
DEM	Digital Elevation Model
FEMA	Federal Emergency Management Agency
Fugro	Fugro Consultants LP
G/CS	Gravel or Crushed Stone
GRR	Graded Riprap
LDNR	Louisiana Department of Natural Resources
LWA	Lightweight Aggregate
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MHHW	Mean Higher High Water
MHW	Mean High Water
MSL	Mean Sea Level
MTL	Mean Tide Level
NAVD	North Atlantic Vertical Datum of 1988
NGDC	National Geodetic Data Center
NGVD	National Geodetic Vertical Datum of 1929
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
P-C	Pelnard Considere
PVC	Polyvinyl Chloride
USACE	United States Army Corps of Engineers
W_{50}	Weight of Median Stone

1. EXECUTIVE SUMMARY

Through the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA), the Louisiana Department of Natural Resources (LDNR) Coastal Restoration Division has been aggressively combating the 25 to 35 square miles of coastal erosion that occurs in Louisiana each year. Subsidence, erosion, and the effects of man have been identified as the causes of converting valuable wetlands and beaches to open water. One such area is Rockefeller Wildlife Refuge in eastern Cameron Parish.

Recent studies estimate that erosion claims an average of 35 ft of marsh along the western 9.2 miles of the Refuge each year. This is the equivalent loss of almost 40 acres per year. As a response to this loss, LDNR engaged Shiner Moseley and Associates, Inc. (Shiner Moseley) to conduct a feasibility study to determine if options exist within a $42 million budget to combat the erosion (Rockefeller Refuge Gulf Shoreline Stabilization Project ME-18, CWPPRA Priority List 10). The feasibility study included a Data Collection Report submitted in September 2002 and a Feasibility Study report submitted in March 2003.

The Data Collection Report presented survey results, geotechnical investigations, gathering of reports, data, and other information from prior investigations, and preliminary analyses of waves and storm surge. Aerial photographs were collected and utilized to determine shoreline recession rates since 1998. Results indicted that recent erosion was approximately 50 ft/yr (57 acres/yr). The highest rates were near the inlet at Joseph's Harbor Bayou with approximately 100 ft of erosion occurring per year. The surveys and geotechnical data indicated a relatively uniform beach and subsurface profile along the Refuge.

Key conclusions of the data collection effort included the following:

- The gentle slope of the beach profile and wide nearshore shelf significantly limit the height of waves that approach the shore.
- The subsurface consists of very soft clay to a depth of approximately 40 ft, which eliminated most conventional shoreline protection alternatives due to bearing capacity and settlement issues.

During the Feasibility Study, potential alternatives for protecting the western 9.2 miles of the Refuge were evaluated based on their ability to meet the following criteria:

- Prevent erosion for up to Category 1 hurricane conditions, which were estimated to have a return interval of about 10 years at the project site.
- Be designed, constructed, monitored, and maintained over a 20-year design life for $42,000,000 with a construction cost of about $38,000,000 or $785/ft.
- Where practicable, the shore protection alternative should remain stable for more severe storm conditions up to a 100-year event.

Due to the soft soil at the site and budget limitations, finding viable alternatives that met these goals was extremely challenging. Numerous alternatives were considered, both conventional and unconventional. After analysis, it became apparent that conventional stone breakwaters and beach nourishment techniques would not be appropriate due to the soft soil. During the initial Feasibility Study, the following two alternatives were identified to potentially provide the needed protection for the Refuge:

(1) Reef breakwater with lightweight aggregate core

(2) Concrete panel breakwater

To allow inclusion of a wider array of potential solutions, modified design criteria consisting of a hypothetical increase of the construction budget by 50% and relaxation of the "no erosion under a Category 1 hurricane" requirement were subsequently considered. The following three additional alternatives were then identified:

(3) Beach fill with gravel / crushed stone

(4) Reef breakwater with sand or gravel / crushed stone beach fill

(5) Soil pre-loading[1]

Because of the unique challenges presented by the soft soils and limited budget, questions remained on constructability, design, and performance. Therefore, the five final alternatives were selected for consideration in a prototype test program at the Refuge to help predict their potential for success if installed for the full 9.2 mile project. The test installations are intended to allow detailed evaluation and comparison of each alternative in terms of constructability, ability to deal with the soft soils, wave attenuation, shoreline response, maintenance requirements, cost, and aesthetics. The present report documents final screening of the final five alternatives, presents recommendations for inclusion of each alternative in the testing program, and describes the parameters employed for design of the test sections. This report supersedes the 30% Design Report submitted August 9, 2004 and 95% Design Report submitted May 2, 2005.

The following are primary findings presented herein:

- Evaluation of shoreline response for each alternative will be limited by the length of the test installations and the duration of the evaluation period. The testing program has been optimized to the extent practicable for drawing valid inferences on shoreline response.
- Monitoring of the test installations is recommended for a minimum of one year. Monitoring should include surveying, aerial photography, and wave measurements.
- Gravel or crushed stone (G/CS) is recommended in lieu of sand for the reef breakwater with beach fill alternative due to the steeper, more stable profile that G/CS will take.
- Although overall soil strength remains low, Fugro has determined that settlement may be slightly less than estimated during the feasibility study, allowing slight increases of the crest elevations for the two reef breakwater alternatives. In addition, the total settlement and associated reductions in crest elevations is expected to occur slowly (over a period of decades). These two findings significantly benefit the project.

[1] The soil pre-loading concept consisted of constructing an onshore berm with imported stiff clay, causing displacement and consolidation of the underlying soft clay; after several months, the berm would be armored with riprap.

- Of the five alternatives, the soil pre-loading approach appears to involve the most uncertainty and is the most experimental. If successful, this approach would allow construction of a revetment for the full 9.2 mile project. However, refinement of design wave parameters resulted in an increase in the required armor size and total quantity of rock required for a stable revetment, making this alternative cost prohibitive for the full 9.2 mile project. *Therefore, the soil pre-loading approach is not recommended for testing*.
- Excluding the soil pre-loading alternative, the remaining four alternatives appear feasible and are recommended for inclusion in the test project.
- As documented in the 30% Design Report, a challenging design component has been determination of the needed quantity of imported backfill material for adequate lateral support of the concrete panel breakwater. Limitations on available methods of analysis that can be performed analytically through modeling and other means have led to a recommendation that a lateral load test be performed as verification that the concrete panel breakwater will withstand wave loads during hurricanes, as intended. The lateral load test would be performed on a shorter 44.5 ft section of wall prior to construction of the full test section.
- All four of the recommended alternatives are expected to provide adequate protection under smaller storms but may allow some erosion under a Category 1 storm. For smaller storms, the combined reef breakwater with G/CS beach nourishment may offer the best protection, although it would likely have the greatest construction cost.
- The concrete panel breakwater and reef breakwater are likely to be the most stable under severe hurricanes. Based on comparison to existing breakwaters along the open Gulf coast of Louisiana, the reef breakwater with lightweight aggregate core is expected to be stable under a Category 1 storm, but is expected to be less stable than the other reef breakwater alternative. Stand-alone beach nourishment with G/CS may not be stable under a Category 1 storm due to potential landward transport of the G/CS across the marsh.
- The opinion of probable construction cost for the testing program is approximately $7,300,000. This cost includes an approximate $1,500,000 contingency to place additional material in the event that instantaneous (construction-phase) settlements are up to 18 inches greater than anticipated. However, the cost does not include project monitoring or construction-phase professional services and is based on higher unit prices expected for the smaller quantities required for the test sections.
- Considering the rapid erosion at the Refuge, the 2002 aerial photography and survey information should be updated on the construction drawings and the project baseline should be shifted north prior to construction.

2. INTRODUCTION

Project Overview and Purpose

Currently, 25 to 35 square miles of wetlands are lost each year along coastal Louisiana. The reasons for the loss are many and complex and vary among different locations within the state. One of the most rapidly eroding portions of the Louisiana Gulf shoreline is at the Rockefeller Wildlife Refuge. Estimates of long-term shoreline retreat range from 30 to 40 ft/year (Byrnes *et al*. 1995). Short-term events, such as Tropical Storm Frances in 1998, can cause more than 50 ft of erosion over a few days.

To combat the direct loss of wetlands in the Rockefeller Refuge, the Louisiana Department of Natural Resources (LDNR) teamed with the National Marine Fisheries Service (NMFS) to implement the Rockefeller Refuge Gulf Shoreline Stabilization Project (ME-18, CWPPRA Priority Project List 10). The project intent is to halt erosion along the 9.2 mile portion of the Refuge west of Joseph Harbor Bayou ("Joseph Harbor"). The project is funded and authorized in accordance with the Coastal Wetlands Planning Protection and Restoration Act (16 U.S.C.A., Section 3951-3956). The present work was conducted under LDNR Contract No. 2511-05-08.

The project is divided into two phases. Phase I is non-construction consisting of engineering and design, land rights, monitoring plan development, baseline monitoring, and project administration. Phase II is construction of the project. The intent of Phase I is to evaluate the economic and technical feasibility of several conceptual designs and, if a conceptual design is proven feasible, to provide detailed plans and specifications. Phase I was further separated into a feasibility and design phase.

The feasibility phase evaluated the potential success, cost, and constructability of several alternatives. It required data collection including bathymetry, geotechnical samples, oceanographic data, aerial photography, and historical information. Data collection and reporting were provided in September 2002 in a Data Collection Report (Shiner Moseley 2002) and the 20% preliminary design was provided in March 2003 in a Feasibility Study report (Shiner Moseley 2003). These initial efforts resulted in selection of several alternatives for possible inclusion in a testing program. The present effort provides a continuation of the Phase I activities and supersedes the 30% Design Report submitted in August 2004 (Shiner Moseley 2004) and 95% Design Report submitted in May 2005 (Shiner Moseley 2005). The present report documents analysis and design parameters for final design of prototype test sections within a portion of the 9.2 mile project area.

Site Description

Rockefeller Refuge encompasses approximately 76,335 acres of southwestern Louisiana and borders the Gulf of Mexico for 26.5 miles. The present project is concerned with protecting the western 9.2 miles of the Refuge from Joseph Harbor to Beach Prong (Figure 1). The beach is mostly composed of exposed marine clays with a ridge of crushed shell above the water line that is backed by extensive marsh. The area is exposed to waves and currents from the open Gulf of Mexico. High tides and/or storms, especially during tropical cyclones, produce considerable erosion.

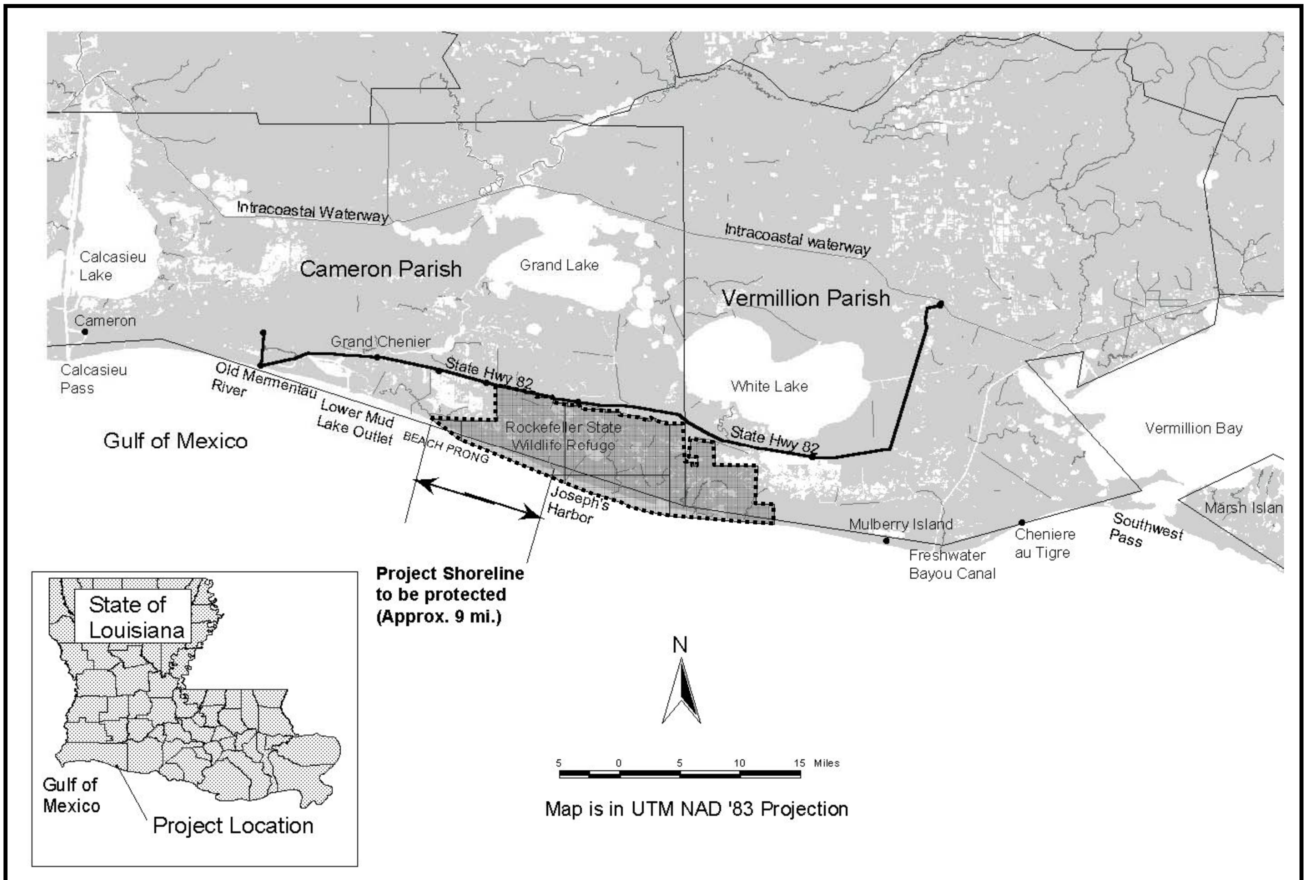

Figure 1: Project Location Map

3. SITE CONDITIONS

Characterization of site conditions involved extensive data collection and analysis which included topographic and bathymetric surveying, geotechnical investigations, aerial photography, review of prior reports and historical information, a wave and water level assessment, and a morphological evaluation. The details and supporting documentation for these efforts are provided in the Data Collection Report and Feasibility Study report. In addition, supplemental geotechnical work was performed for the 30% design effort to better define settlement and bearing capacity, as well as to perform more intensive soil characterization within the actual test-installation area. This additional geotechnical assessment allowed application of less conservative design factors. Detailed results of the additional geotechnical assessment are presented in Fugro (2004). Key findings of the data collection and site characterizations are summarized below.

Geomorphology

Byrnes *et al.* (1995) concluded that modern rates of shoreline recession within Louisiana's Chenier Plain are generally increasing with time, and that the long-term rate along the Rockefeller Refuge is about 35 ft per year. It is well recognized that tropical cyclones play a significant role in contributing to this erosion. During storms, the deposits of shell that are perched atop the beach along the Refuge shoreline can be transported landward by waves as washover deposits onto the marsh. This process results in an exposed zone of fragmented marsh seaward of the beach to be reworked during and/or after the storm.

Depending on storm duration, the stronger storms that generate a large surge may not necessarily produce the most severe erosion since the beachface and marsh become submerged and are somewhat protected by a cushion of water as waves pass overhead. However, storms with a large surge can still produce severe erosion. For example, in June 1957, Hurricane Audrey produced a surge of 9 to 13 ft along the Louisiana coast and caused about 140 ft of erosion (Morgan *et al*. 1958). A more extensive literature review on relevant geology, geomorphology, and sand resources was performed by Applied Coastal Research and Engineering, Inc. (ACRE 2002a).

Storm Surge and Wave Analysis

An analysis of nearshore wave climate and storm surge conditions was presented by ACRE (2002b) as an appendix to the Feasibility Study report. In their analyses, ACRE calculated wave transformation for statistical offshore waves by applying the wave energy dissipation theory of Battjes and Janssen (1978). This approach was somewhat conservative since it neglected dissipation of nearshore waves by fluid mud which has been observed offshore of Atchafalaya Bay (Sheremet *et al*. 2005) and likely occurs along the entire Chenier Plain coast. Research to develop methods for estimating the dissipation of wave energy by fluid mud is ongoing (Kaihatu and Sheremet 2004).

The results of the wave analyses were summarized in plots of wave height versus position across shore for various return intervals. The results indicated that, during storms, nearshore wave heights have a much stronger dependency on surge elevation than offshore wave height or windspeed. Storm waves are primarily controlled by the broad, shallow shelf offshore of the Refuge that attenuates the waves as they travel landward. Therefore, because surge elevation directly controls nearshore wave heights, surge elevation, as opposed to windspeed or offshore wave height, is the key factor in defining the project design waves.

At the time of preparation of the Feasibility Study report, the project design criteria required that a Category 1 hurricane be applied as the design basis for erosion prevention[2]. However, the structural stability of the shore protection structures was evaluated for more severe conditions to minimize dispersal of material during major hurricanes. The goal of the storm surge and wave analysis was thus to define water level and wave conditions for various return intervals and for a Category 1 storm.

ACRE provided surge elevations for storms of various return intervals. However, they did not specifically determine the return interval of a Category 1 hurricane. The surge elevations provided by ACRE are based on modeling performed for Vermilion Parish by the Federal Emergency Management Agency (FEMA) for establishing flood insurance rates and assisting communities with promoting sound floodplain management. The results are also often applied for establishing safe hurricane evacuation routes and to prevent public disasters. In achieving these purposes, FEMA's estimates of surge elevation likely contain a greater degree of conservatism than warranted for estimating nearshore waves at the Refuge.

Another source of surge estimates is the SLOSH (Sea, Lake and Overland Surges from Hurricanes) data compiled by the National Weather Service. The SLOSH model provides estimates of storm surge heights and winds resulting from historical, hypothetical, and predicted hurricanes. The data for a given location are typically presented in terms of a "MEOW" (maximum envelope of water) which represents the surge associated with a "family" of storms having variable direction, speed and intensity. The results are applied as an estimate of the potential range of surge elevations for a given storm category. Based on the SLOSH data distributed by the National Oceanic and Atmospheric Administration (NOAA) in January 2003, the MEOW for a Category 1 hurricane at the Refuge shoreline ranges from +3.8 ft to +5.4 ft NAVD[3].

As a third source of surge data, historic tide records were employed. To be considered appropriate for the analysis, data were required to be from tide gauges located within the project region, at or very close to the open coast, and having been in operation for at least ten years. Bay and river tide gauges often have much higher readings than open coast gauges during storms and were not considered. Tide stations at Sabine Pass, Calcasieu Pass, and Eugene Island were selected as meeting the stated requirements.

A summary of the number of surge events equaling or exceeding given elevations as captured by tide records at these stations is presented in Table 1. In reviewing the historic tide records, it was clear that peak surge events were sometimes missing due to damage to the tide gauges as storms developed. In addition, several of the apparent peak tide events were deemed to be inaccurate data spikes and were excluded.

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[2] Note that, subsequent to completion of the Feasibility Study report, the project design team considered relaxation of the no erosion under a Category 1 storm criteria, allowing evaluation of additional alternatives.
[3] All elevations in this report are referenced to NAVD'88 (North American Vertical Datum of 1988), unless noted otherwise.

Note that only three recorded events had measured surge elevations exceeding +5.5 ft. These were Hurricane Audrey in June 1957, Hurricane Carla in August 1961, and Hurricane Rita in September 2005. At landfall, Audrey and Carla were Category 4 storms and Rita was a Category 3. While Carla crossed the coast in central Texas, both Audrey and Rita made landfall near the Texas-Louisiana border. For the overall period of record, the highest tides in western Louisiana appear to have been during Hurricane Rita. Although tide gauges at Sabine Pass and Calcasieu Pass were damaged and failed to record peak surge elevations during Rita, high water marks suggested the surge was in excess of +15 ft at the town of Cameron just east of Calcasieu Pass (National Weather Service 2005).

Table 1: Recorded Storm Surge Events

Location	Years of Record	Station I.D.	Number of Events Equaling or Exceeding Given Elevation (NAVD)		
			Elev. +4.0 ft	Elev. +4.5 ft	Elev. +5.0 ft
Sabine Pass	1958-1985 (27 yrs)	NOAA 8770590	4	1	1[a]
Sabine Pass North	1985-2005 (19 yrs)	NOAA 8770570	10	4	2[b]
Calcasieu Pass (Daily Record)	1942-2004 (62 yrs)	NOAA 8768111 USACE 73650	10	4	3[c]
Calcasieu Pass (Hourly Record)	1987-2005 (18 yrs)	NOAA 8768111 USACE 73650	7	2	1[d]
Calcasieu Pass East Jetty	2002-2005 (3 yrs)	NOAA 8768094	1	1	1[e]
Eugene Island	1934-2003 (68 yrs)	USACE 88600	9	5	3[f]

Footnotes:
(a) 1961 Hur. Carla (+7.4 ft)
(b) 1998 T.S. Frances (+5.3 ft) and 2005 Hur. Rita (> +6.8 ft)
(c) 1957 Hur. Audrey (est. > +10.0 ft), 1961 Hur. Carla (+6.0 ft), and 1998 T.S. Frances (+5.3 ft)
(d) 1998 T.S. Frances (+5.3 ft)
(e) 2005 Hur. Rita (est. > +15 ft)
(f) 1957 Hur. Audrey (+7.3 ft), 1961 Hur. Carla (+6.4 ft), and 1985 Hur. Danny (+5.5 ft)

During Hurricane Audrey, high water marks at Calcasieu Pass were reported by USACE (2005) and Ross and Blum (1957) to be approximately +12.2 ft and +12.0 ft, respectively. Hurricane Carla had a measured surge elevation of +7.4 ft at Sabine Pass. Other notable storms include Hurricane Edith, Hurricane Danny, and Tropical Storm Frances. Edith made landfall near the western boundary of the Refuge in September 1971 as a Category 2 hurricane and had a recorded surge at Calcasieu Pass of +4.1 ft. Danny made landfall at the Refuge in August 1985 as a Category 1 hurricane and had a recorded surge at Eugene Island of +5.5 ft. Frances occurred in September 1998, approaching the coast very slowly prior to making landfall in central Texas and producing a recorded surge of +5.3 ft at both Sabine Pass and Calcasieu Pass.

Hurricane return intervals published on the National Hurricane Center web page (www.nhc.noaa.gov/HAW2/english/basics/return_printer.shtml) indicate that a Category 1 hurricane has a return interval of about 9 to 12 years for the project site. Based on analysis of the available tide data, it appears that an elevation of +4.5 ft is representative of the storm surge for this range of return intervals. However, considering the relatively limited data available and the fact that tide gauges are often damaged during severe storms and fail to record peak surge events, a more conservative elevation of +5.0 ft NAVD was selected for project design. This elevation is within the MEOW range presented for the SLOSH data.

As mentioned, the tide records summarized in Table 1 include a strong tropical storm (Tropical Storm Frances) that generated a surge elevation of +5.3 ft. The fact that this event was categorized as a tropical storm instead of a hurricane emphasizes the difficulty in correlating storm classification with surge height. Typically, windspeed is the primary parameter considered for categorizing storms, which is partially because reliable tide data are usually lacking. Although it is clear that peak surge elevations can exceed +5.0 ft during strong Category 1 hurricanes, +5.0 ft appears to be representative for the 9 to 12 year return interval stated by the National Hurricane Center for a Category 1 storm within the project region.

Note that the marsh at the project site has an elevation of about +2.0 ft and will be submerged under the +5.0 ft storm surge assigned for a Category 1 hurricane. Existing vegetation combined with the overlying layer of water will help provide some protection to the marsh sediments against waves, depending on wave height, the actual thickness of the water layer, and storm duration.

In addition to information on storm surge, ACRE developed design wave information. A summary of available water level and wave estimates is summarized in Table 2. These conditions are for a structure that is located at the approximate future (20 year) -6 ft contour, as explained in more detail in Chapter 5.

Table 2: Waves and Water Level

Return Interval*, Years	**Conditions at -6.0 ft Contour**				
	Still Water Level, ft (NAVD)	Still Water Level + Wave Setup, ft (NAVD)	H_{max}, ft	H_s, ft	T_m, sec
1	+1.7	+2.3	6.6	4.2	8.5
(Category 1 Storm)	---	+5.0	8.6	5.5	9.2
10	+6.4	+7.1	10.2	6.4	9.7
25	+8.7	+9.8	12.3	7.9	12
50	+10.5	+11.5	13.6	8.6	12
100	+12.0	+13.0	14.7	9.2	12

*Note: Listed return intervals are based on approximations published by FEMA (1984) for Vermilion Parish and are conservative compared to actual tide records. As a comparison, FEMA's (1991) estimate for a 100-year still water level in Cameron Parish is about +11.0 ft. (Note that FEMA did not provide estimates for other return intervals for Cameron Parish.) As discussed in Section 5, FEMA categorizations will not necessarily serve as the basis for design.

Tidal Datums

The tide along the Louisiana coast is mixed, meaning that both semi-diurnal and diurnal signals are present, but the diurnal component is typically dominant. Although the diurnal tide range is defined as an average of the differences between low and high waters, on the Louisiana coast, the seasonal change in water elevation may be comparable to or greater than the diurnal range, particularly in its shallow bays and estuaries. Because the tide range is defined as the difference between daily high and low waters, on the Louisiana coast the tide range does not reflect the much larger range in water elevation that occurs seasonally due to meteorological and other factors.

Although no long-term tide statistics are available for the immediate project area, long-term records and published tidal statistics are available through NOAA for Grand Isle, which is approximately 170 miles to the east, and Sabine Pass, which is approximately 60 miles to the west. The diurnal tide ranges at Grand Isle and Sabine Pass are 1.1 ft and 1.6 ft, respectively. Available information is summarized in Table 3.

Table 3: Elevation Datums

Datum	Elevation Referenced to NAVD, ft	
	Grand Isle, Louisiana NOAA Station 8761724	Sabine Pass, Texas NOAA Station 8770570
MHHW	+1.55	+2.04
MHW	+1.52	+1.91
MTL	+0.99	+1.40
MSL	(Not Published)	+1.40
MLW	+0.46	+0.88
MLLW	+0.44	+0.42
NGVD	+0.28	(Not Published)
Minimum Water Level	-2.32 (02/03/51)	-3.32 (01/19/96)

Geotechnical Conditions

Two geotechnical investigations involving fieldwork, laboratory testing, and engineering analysis were performed by Fugro Consultants LP (Fugro) as part of the feasibility study and 30% design phases. Results of the first investigation are presented in Fugro (2002, 2003) as appendices to the Data Collection Report and Feasibility Study report, respectively. Results of the second investigation are presented in Fugro (2004), which was provided as an appendix to the 30% Design Report. Key results of the geotechnical investigations are provided below.

- The subsurface conditions appear to be relatively uniform alongshore and across-shore. Between approximate elevations +5 feet and 0, the soil is a loose to medium-dense shell with shell fragments. Below this stratum to an approximate depth of 40 feet is a very-soft to soft under-consolidated clay that was reported by Fugro to have a consistency “similar to drilling mud.” Below the stratum of very-soft to soft clay to a depth of at least 100 feet is a stiff to very stiff clay. Figure 2 provides a generalized subsurface profile along the project shoreline from Joseph Harbor (B-20) to the western boundary of the Refuge (B-1).

- Grab sampling did not reveal any significant surface deposits of sand or shell across the submerged portion of the beach profile. The surface sediments appear to be relatively uniform in both the across-shore and alongshore directions, although there is an abrupt change across-shore at the waterline where the surface sediments change from shell particles to silt and clay.

- Due to the stratum of very soft to soft clay to a depth of approximately 40 feet, the allowable bearing pressure of the soil is low (less than 300 psf).

- Total settlement is expected to be in excess of one foot for all alternatives being considered.

- Soil consolidation and settlement due to the bearing pressure of the shore protection structures will occur slowly. Only about 40 to 50 percent of the total settlement is expected over a period of about 8 to 12 years. The remaining settlement will likely occur over a period of 40 to 45 years.

- Instantaneous (construction) settlements associated with soil displacement and elastic compression of the soft clay are challenging to predict and will be highly dependent upon the placement techniques employed by the construction contractor. Through effective application of geotextiles and depending on the placement methods, instantaneous settlements may range from 6 to 24 inches.

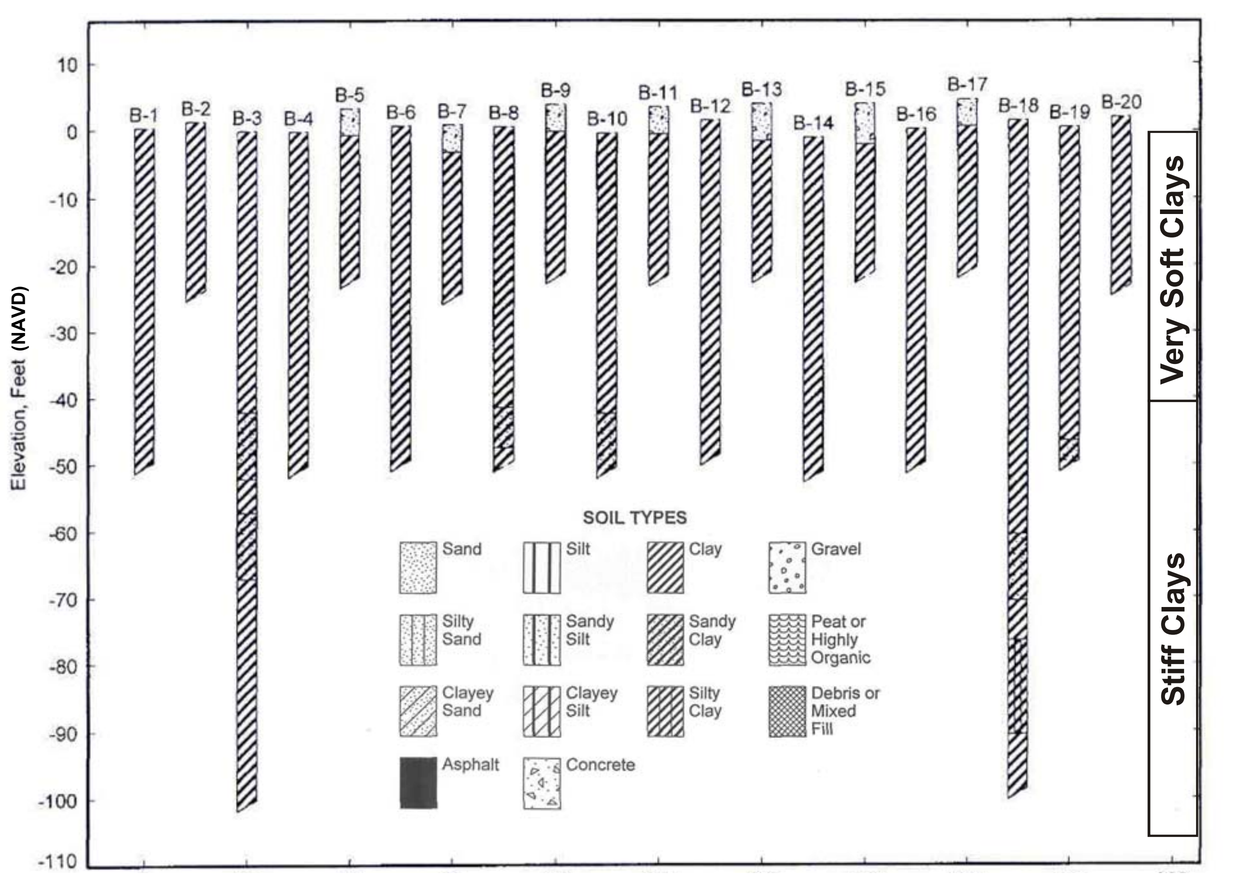

Figure 2: Generalized Subsurface Profile from Fugro

Subsidence and Sea-Level Rise

As documented by the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (1998), little information is available on recent rates of subsidence for the Chenier Plain of Louisiana. According to Gagliano (1998), subsidence rates along the Gulf shoreline of the Chenier Plain are on the order of 0 to 1 ft per century. Kulp (2000) estimated average long-term (over thousands of years) rates to be less than 1 ft per century. However, short-term subsidence rates can be much greater. For example, Swanson and Thurlow (1973) estimated short-term rates for various periods between 1948 and 1970 at Eugene Island and Sabine Pass to be in excess of 0.4 inches per year. Shinkle and Dokka (2004) estimated recent rates within the vicinity of the Rockefeller Refuge to be on the order of 0.4 to 0.6 inches per year. In general, subsidence rates along the Gulf coast appear to decrease with distance west from the Mississippi River.

The combination of vertical land motion (such as subsidence) and eustatic (i.e., global) sea-level rise is termed "relative sea-level rise." Considering that eustatic sea-level rise has been estimated to be on the order of only 0.05 inches per year (Shinkle and Dokka 2004), the estimates of subsidence mentioned above provide reasonable approximations of relative sea-level rise. In addition, rates of relative sea level rise along the coasts of the United States have been estimated by NOAA from water level trends at tide stations having long term (decadal) records (Zervas 2001). These estimates can be viewed online at www.co-ops.nos.noaa.gov/sltrends/sltrends.shtml. Data collected from 1958 to 1999 at Sabine Pass and from 1939 to 1974 at Eugene Island suggest rates of relative sea-level rise of 2.2 ft per century and 3.2 ft per century, respectively.

Based on these trends, relative sea-level rise at Rockefeller Refuge may be anywhere from five to ten inches over the desired 20-year design life of the project. Future increases in relative sea level rise have implications for estimating whether or not significant increases in wave transmission of project alternatives could occur over time. Wave transmission is discussed in more detail in Chapter 5.

Topographic and Bathymetric Survey

John Chance Land Surveyors, Inc. (Chance) performed a combined topographic and bathymetric survey of the project area in late June through early July of 2002. The purpose of the survey was to provide quantitative information about the shape of the beach profile, which is useful in helping to determine local wave conditions. In addition, knowledge of the beach profile shape can be applied towards prediction of future depths at a given location across the profile, which is essential to developing coastal engineering designs such as breakwaters and beach fills.

The survey measured the shape of the beach profile at twenty-two locations from approximately 600 ft landward to approximately 3,500 ft seaward of the shoreline. The survey also included the establishment of three permanent benchmarks in the Refuge, bringing the total to four benchmarks near the shoreline within the Refuge, and a series of twenty-two temporary benchmarks along the beach. A magnetometer survey was conducted concurrently with the bathymetric survey. The Chance report on the surveying work, including a description of the benchmarks, survey data, and field books, along with the survey drawings are included in the Data Collection Report (Shiner

Moseley 2002). Considering the rapid erosion at the Refuge, the survey information should be updated prior to construction.

Chance conducted a preliminary-level magnetometer survey to locate any pipelines or other hazards crossing the project area. Chance company files were reviewed for evidence of any reported man-made features. The listing of wellheads, piles, and obstructions within the Refuge compiled by Refuge staff and provided to Shiner Moseley was in turn provided to Chance and the locations of these obstructions in the project area were verified. The construction contractor will be required to perform a more detailed hazard survey prior to construction.

Results of the combined topographic and bathymetric survey profiles are presented in the Chance surveying report and drawings in the Data Collection Report (Shiner Moseley 2002). To evaluate cross-shore variations along the length of the study area, all profile lines were plotted on a single graph, shown in Figure 3. As seen in the plot of the translated profiles, the shape of the profile is very consistent throughout the project area. Note that profile K-K' and L-L' intersect a levee in the upper portion of the profile.

Across the beach, several features are consistent among profile lines. These features (moving from upland to offshore) were identified as: marsh, back beach, berm (beach) crest, upper beach, lower berm, lower beach face, nearshore zone, and offshore zone. Figure 4 depicts each of these features along with average dimensions.

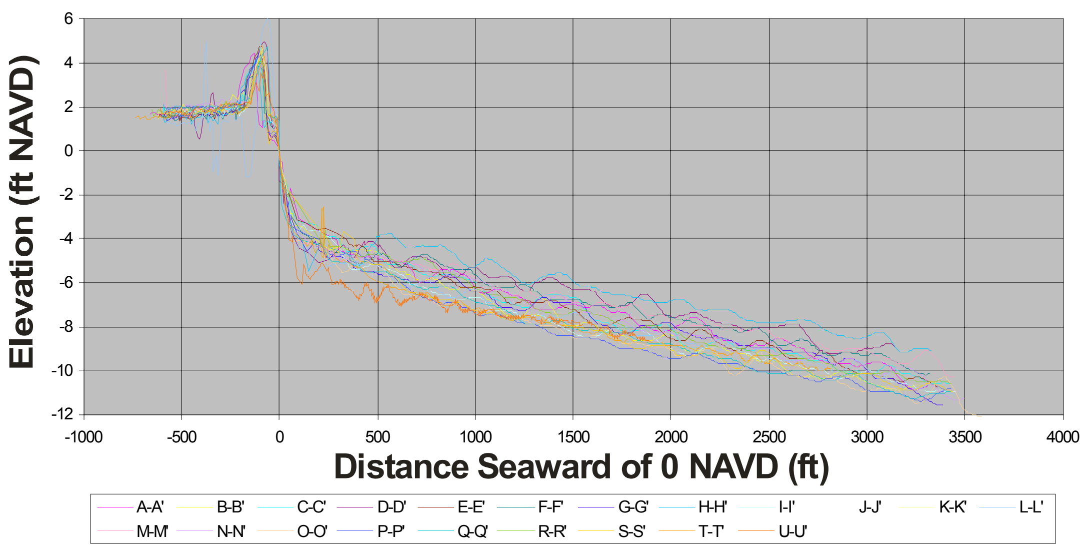

Figure 3: Beach Profiles Smoothed and Translated to Common Shoreline Position

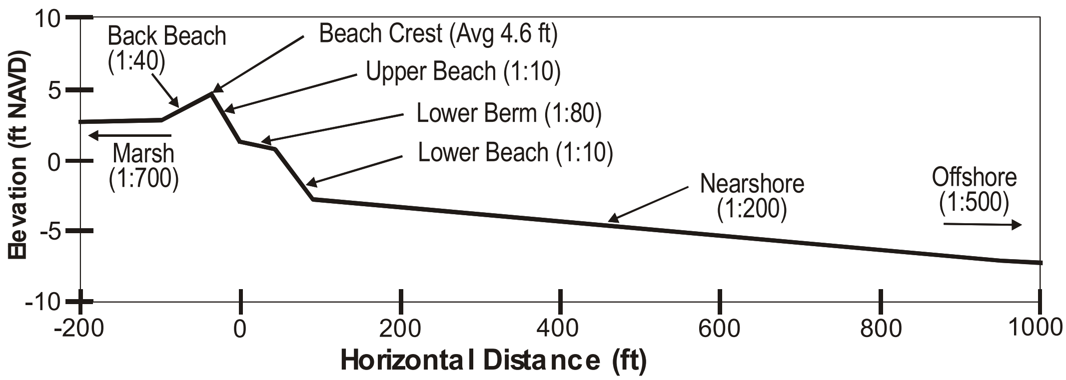

Figure 4: Beach Features and Average Slopes

The back beach and upper beach consist of a veneer of shell hash overlying marsh sediment, which is transported landward by waves during higher tides. The lower berm is predominantly exposed clays with some remnant plant material; this feature, where it exists, is typically much flatter than either the upper or lower beach and forms a terrace. A photograph of a typical lower berm is shown in Figure 5. Note the presence of remnant marsh grass roots in the foreground. Field observations made by Shiner Moseley lead to the conclusion that wave action may undercut the seaward portion of these lower berms beneath the roots of the living or recently dead plants. Some narrow berms were observed with significant "tilt" of the berm surface on which plant material was still growing, indicating that the undercutting was severe. Immediately seaward of the lower berm is the lower beach. There is not a significant amount of sand or shell in this portion of the profile.

Figure 5: Typical Lower Berm

Understanding the wave erosion process is important. During two site visits to the project area (on March 14 and May 22, 2002), wave transformation and breaking in the project area were observed. No offshore line of breaking waves was observed that would indicate the presence of a bar, which is typically found on gently sloping (dissipative) sand beaches. Waves were observed to break on or just seaward of the edge of the lower berm. The waves typically exhibit a combination of spilling and plunging breaking behavior as opposed to purely spilling breaking that is often found on gently sloping beaches. Vertical jets of water are generally associated with plunging breakers; these jets can be highly erosive, especially when waves break directly on a shelf or terrace.

The NOAA National Geodetic Data Center (NGDC) Coastal Relief Model (Vol. 4 – Central Gulf of Mexico) was obtained to evaluate regional bathymetric trends. This is a digital elevation model (DEM) compiled from numerous bathymetric surveys from 1930 to present. Figure 6 is a color coded bathymetric contour map with elevation contours in meters. Note that that the 10 and 14 meter contours are closer together in front of the Refuge and diverge both to the east and west. The closer contours indicate that the bottom slope is steeper and deeper water is closer to shore, which can mean more wave energy and larger waves reach the shoreline.

The beach profile can be expected to translate landward with a constant shape as the shoreline recedes. This expectation is supported by the similarity between the beach profiles and soils in 2002 and in the study by Nichols (1959). An assumption can also be made that erosion will continue offshore in a similar manner after construction of shore protection structures, with a possibility of some increased local erosion caused by the structure. Using this assumption and neglecting subsidence, the future depth at any position across the profile can be estimated by translating the profile landward by the predicted amount of shoreline recession (annual recession rate multiplied by number of years in the future). Using an annual erosion rate of 50 ft per year determined from comparison of recent aerial photographs (Shiner Moseley 2002) and the Byrnes *et al*. (1995) estimate of long term erosion rate of 35 ft per year, the elevations in 20 years at several locations along the typical profile was estimated. This estimate is shown in Table 4. The results show that, if erosion continues unabated, the water depth at the current shoreline will be about 5.5 ft in 20 years (see Figure 7). Consideration of profile lowering is important for design of the proposed shoreline protection structures.

Table 4: Profile Translation and Associated Elevation Changes

Location	Current Elevation, ft (NAVD)	Predicted Elevation in 20 Years, ft (NAVD)	
		50 ft/yr recession rate	35 ft/yr recession rate
Beach Crest	+4.6	-6.2	-5.2
Upper Beach/Lower Berm Interface	+1	-6.4	-5.5
Current -2 ft Contour	-2	-6.5	-5.8
Current -5 ft Contour	-5	-7.2	-6.5
Current -8 ft Contour	-8	-10.0	-8.8

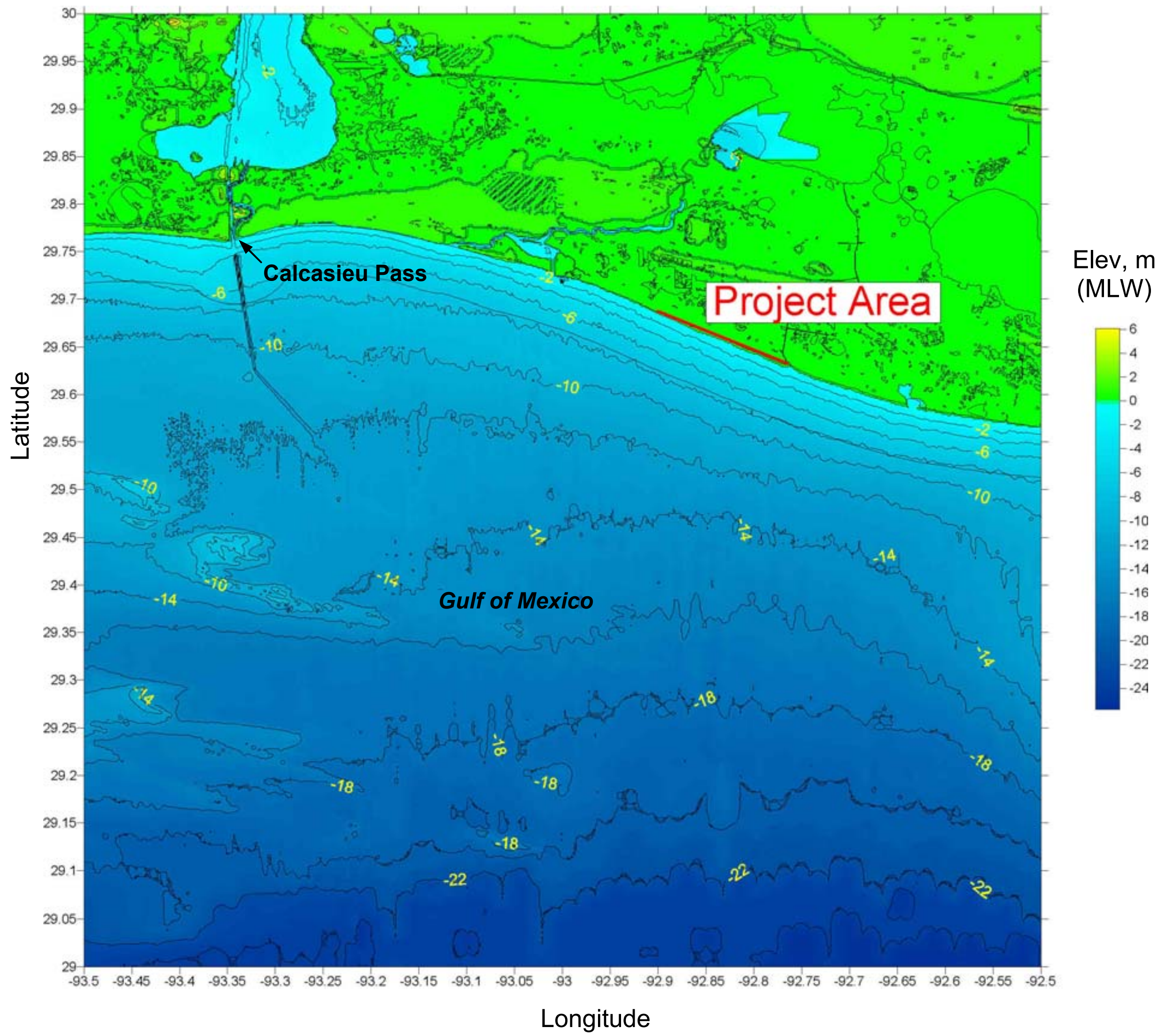

Figure 6: Bathymetric Contours (elevation in meters)

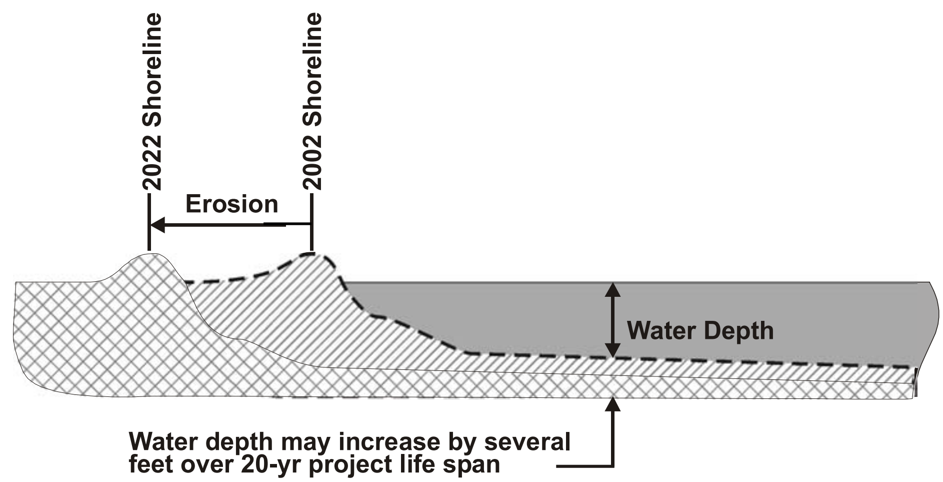

Figure 7: Estimated Profile Translation

Aerial Photography

Black and white aerial photography was obtained for the Refuge coastline on July 18, 2002 by Lanmon Aerial Photography, Inc. The photographs were geo-referenced and a digital mosaic was created for comparison to a March 2, 1998 aerial photograph from the U.S. Geological Survey. This comparison is shown in the Data Collection Report (Shiner Moseley 2002), and a summary is provided in Figure 8. The comparison shows that there was an average of approximately 50 ft/yr of shoreline recession between March 1998 and July 2002, or an average net loss of 200 ft. This rate is slightly greater than the longer-term rate of about 35 ft/year reported by Byrnes *et al*. (1995) and others, but is in general agreement with previously documented rates, especially shorter-term and more recent rates. The magnitude of differences in reported erosion rates is not expected to significantly affect the design approach.

An additional source of recent aerial photography was the LaCoast Web page maintained by USGS (http://www.lacoast.gov/). This source provided photographs of the Refuge taken after Hurricane Rita in September 2005. A comparison of the July 2002 and post-Rita shorelines at the east end of the project area indicated an average change of approximately 35 ft/yr, or a net loss of about 155 ft.

Note from the shoreline changes plotted in Figure 8 that the rate of shoreline recession is somewhat variable alongshore, particularly near Joseph Harbor, where as much as 400 ft of recession occurred. The higher rates measured near Joseph Harbor may be a short-term anomaly considering that the photographs represented only a 2.5-year period, or it could be related to readjustment of the shoreline associated with the loss of the small island that recently existed at the mouth of the channel. Longer-term records do not appear to indicate a higher erosion rate near Joseph Harbor.

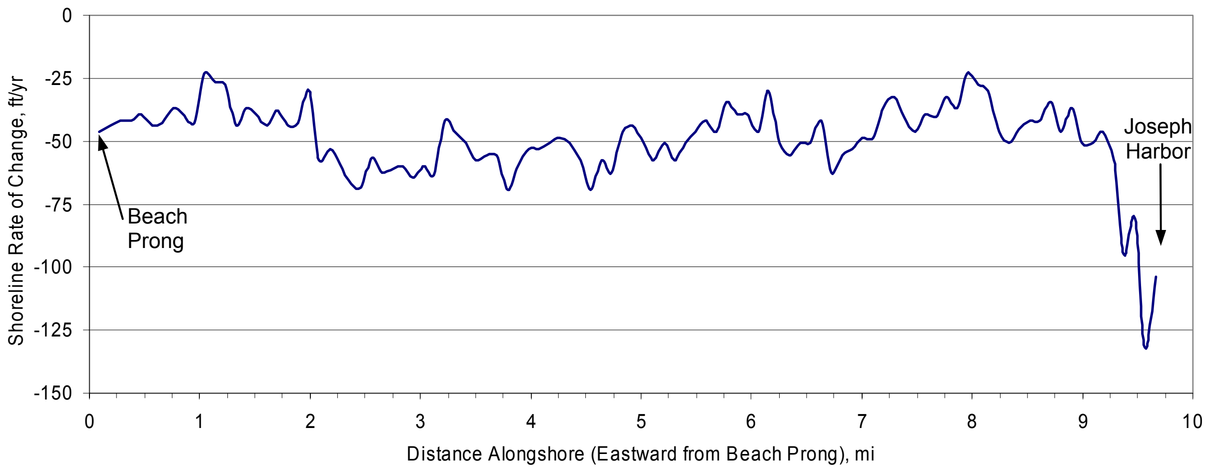

Figure 8: Rates of Shoreline Retreat from 3/2/98 to 7/18/02

4. ALTERNATIVES EVALUATIONS

During the Feasibility Study, potential project alternatives were evaluated based on their ability to meet the following criteria:

- Prevent beach erosion for up to Category 1 hurricane conditions, which were estimated to have a return interval of about 10 years at the project site.
- Be designed, constructed, monitored, and maintained over a 20-year design life for $42,000,000 with a construction cost of about $38,000,000 or $785/ft.

In addition to the criteria stated above, where practicable, the protection should remain stable for more severe storm conditions up to a 100-year event. To find a shore protection alternative that would meet these criteria, an alternatives identification and evaluation was performed. The low bearing capacity of the soils severely limited the type of shoreline protection that could be built and provide the desired protection. Over 80 alternatives and variations of alternatives were considered, as summarized in Table 5.

The initial screening of these alternatives reduced the number of possible alternatives to 14. Design, cost, and construction considerations for these 14 alternatives were then evaluated in more detail. Table 6 provides a synopsis of each of the 14 alternatives. As described extensively in the Feasibility Study report, most of the alternatives were eliminated based on cost and/or the bearing pressure being too great for the soil. After final screening, only the reef breakwater with lightweight aggregate (LWA) core and concrete panel breakwater were recommended for further consideration. Because of the unique site conditions, innovative nature of the proposed alternatives, and lack of definitive design methodology, test sections were proposed for further evaluation.

Subsequent to submittal of the final Feasibility Study report and decision to implement test sections, modified design criteria were considered to allow evaluation of additional alternatives. Under the modified design criteria, a hypothetical increase of the construction budget by 50% (i.e., from $38,000,000 to $57,000,000) and relaxation of the "no erosion under a Category 1 hurricane" requirement were considered. Assessment of the modified criteria was documented in a memorandum to LDNR dated December 15, 2003. A summary of nine additional alternatives that were screened is provided in Table 7. Following this additional screening, a third approach consisting of soil pre-loading for later construction of a breakwater or revetment was selected for further analysis.

During a meeting with LDNR on December 16, 2003, two more alternatives that were previously eliminated during the Feasibility Study based on cost were selected: a reef breakwater combined with beach nourishment, and gravel/crushed stone beach nourishment. Adding these alternatives brought the total number of approaches for further evaluation to five, as listed below.

(1) Beach fill with gravel/crushed stone
(2) Reef breakwater with sand or gravel/crushed stone beach fill
(3) Reef breakwater with LWA core
(4) Concrete panel breakwater
(5) Soil pre-loading

Table 5: Initial Alternatives Screening

Protection Method	Proven Design?	Constructability	Design Life	Maintenance / Operation	Cost	Shoreline Protection	Experimental?	Continue Evaluation	Comments
Detached Breakwaters									
Layout Alternatives									
▪ Shore parallel continuous	Y	Y	H	L	M	M	N	Y	---
▪ Shore parallel with gaps / fish-dips	Y	Y	H	L	M	M	N	Y	---
▪ Angled	Y	Y	H	L	M	M	N	Y	---
▪ Submerged / reef	Y	Y	H	L	M	L	Y	Y	---
▪ Multi-crested	N	Y	H	L	H	U	Y	N	Design parameters and cost
▪ Baffle	N	Y	H	L	H	M	N	N	Cost for materials and performance
Construction Materials Alternatives									
▪ Rock / rip-rap	Y	Y	M	M	M	M	N	Y	---
▪ Caissons	N	N	M	L	H	M	Y	N	Increased cost, soil load, and reflection over rock
▪ Concrete panel	Y	Y	M	M	M	M	Y	Y	Optimize panel spacing and gaps to lower forces
▪ Large-diameter concrete piles	N	N	U	U	H	M	Y	N	Design parameters, cost
▪ Sunken barges	N	N	L	L	M	L	Y	N	Poor performance, design parameters
▪ Metal gabions	Y	Y	L	M	L	L	N	N	Wave heights exceed maximum for gabions
▪ Plastic gabions	Y	Y	L	L	L	L	N	N	Wave heights exceed maximum for gabions
▪ Metal sheet pile	Y	Y	M	H	H	M	N	Y	---
▪ PVC sheet pile	Y	Y	L	L	L	M	N	N	Pile sections with required modulus not available
▪ Timber – "picket fence"	N	Y	L	L	M	M	N	N	No proven design for open Gulf applications
▪ Floating	N	Y	L	L	M	M	N	N	Not effective for longer wave periods of open coast
▪ Sand-filled geotextile tubes / bags	Y	Y	L	L	M	M	N	N	Subject to debris punctures and deflation
▪ Rock/rip-rap armored sand-filled geotextile tubes / bags	N	N	M	M	L	M	Y	N	Subject to debris punctures and deflation
▪ Articulating block mat armored sand-filled geotextile tubes / bags	N	Y	M	M	M	M	Y	N	Wave heights exceed maximum for articulating block mats
▪ Grout / fly ash filled geotextile tubes / bags	N	Y	M	L	M	M	N	N	Tubes subject to rupture due to differential settlement
Base / Sublayer Alternatives									
▪ Geotextile fabric	Y	Y	M	L	L	NA	N	Y	Base / sublayer alternatives to be further evaluated as required during detailed design phase
▪ Plastic geogrid	Y	Y	M	L	M	NA	N	Y	

Table 5: Initial Alternatives Screening

Protection Method	Proven Design?	Constructability	Design Life	Maintenance / Operation	Cost	Shoreline Protection	Experimental?	Continue Evaluation	Comments
▪ Combination geotextile fabric / geogrid	Y	Y	M	L	M	NA	N	Y	
▪ Tensar marine mattress	Y	Y	M	L	H	NA	N	Y	
▪ Small stone / sand	Y	Y	M	L	M	NA	N	Y	
▪ Small stone / sand with geotextile fabric	Y	Y	M	L	M	NA	N	Y	
▪ Sheet pile toe reinforcement	Y	Y	M	L	H	NA	N	Y	
Offshore Levees									
Rock / rip-rap armoring	Y	Y	H	M	M	M	N	Y	---
Articulating block mat armoring	N	Y	L	H	L	M	N	N	Wave heights exceed maximum for ABMs
Angled sheet pile armoring	N	N	L	H	M	M	Y	N	Unproven design
Tensar marine mattress armoring	N	Y	L	H	M	M	Y	N	Wave heights exceed maximum for mattress
Sediment Fill Systems									
Sacrificial berm (clay)	N	Y	M	L	H	M	Y	Y	---
Sand fill	N	Y	L	M	H	M	Y	Y	---
Sand with longshore retention structures – groins	N	Y	M	M	H	M	Y	Y	---
Sand with cross-shore retention structures – sills/walls	N	Y	M	M	H	M	Y	Y	---
Sand with longshore and cross-shore retention structures – groins and sills/walls	N	Y	M	M	H	M	Y	Y	---
Gravel / crushed stone	N	Y	H	M	H	M	Y	Y	---
Gravel / crushed stone with longshore retention structures – groins	N	Y	H	M	H	M	Y	Y	---
Gravel / crushed stone with cross-shore retention structures – sills/walls	N	Y	H	M	H	M	Y	Y	---
Gravel / crushed stone with longshore and cross-shore retention structures – groins and sills/walls	N	Y	H	M	H	U	Y	Y	---
Shoreline Armoring Systems									
Rock / rip-rap revetment	Y	Y	H	L	H	H	N	Y	---
Articulating block mat	N	Y	L	H	L	L	N	N	Wave heights exceed maximum for ABMs
Onshore levees with rock / rip-rap shore protection	Y	Y	H	M	M	M	N	Y	---
Onshore levees with articulating block mats	N	Y	L	M	M	M	N	N	Wave heights exceed maximum for ABMs

Table 5: Initial Alternatives Screening

Protection Method	Proven Design?	Constructability	Design Life	Maintenance / Operation	Cost	Shoreline Protection	Experimental?	Continue Evaluation	Comments
Onshore levees with Tensar marine mattress	N	Y	L	H	M	L	Y	N	Wave heights exceed maximum for mattress
Onshore levees with angled sheet pile (large angle)	N	N	U	U	M	H	Y	N	Unproven design
Metal gabions	Y	Y	L	H	L	L	N	N	Wave heights exceed maximum for gabions
Plastic gabions	Y	Y	M	H	L	L	N	N	Wave heights exceed maximum for gabions
Tensar marine mattress	N	Y	L	H	L	L	Y	N	Wave heights exceed maximum for mattress
Angled sheet pile (large angle)	N	N	U	U	U	U	Y	N	Unproven constructability
Sand filled geotextile tubes	Y	Y	L	L	M	H	N	N	Subject to debris punctures and deflation
Rock / rip-rap armored sand filled geotextile tubes	N	Y	M	M	M	H	Y	N	Subject to debris punctures and deflation
Articulating block mat armored sand filled geotextile tubes	N	Y	M	H	M	M	Y	N	Wave height exceed mats maximum
Grout / fly ash filled geotextile tubes	N	Y	M	H	M	M	N	N	Grout filled geotextile tubes subject to rupture due to differential settlement
Pile / pipe combination structure	N	Y	M	H	M	L	Y	N	Design / constructability
Metal sheet pile	Y	Y	M	H	H	H	N	Y	---
Caissons	N	N	M	L	H	H	Y	N	Increased cost, soil load, and reflection over rock
Angled sheet pile (small angle)	N	N	U	U	H	H	Y	N	Design / constructability
Onshore levees with angled sheet pile (small angle)	N	N	U	U	H	H	Y	N	Design / constructability
Cemented soil columns	N	U	M	H	H	H	Y	Y	---
Cemented soil columns with sheet pile protection	N	U	H	H	M	H	Y	Y	---
Vegetation	N	Y	L	L	L	L	Y	N	Not stable in open coast, may be used in combination with other options

Key: Y = Yes; N = No; M = Medium; L = Low; H = High; U = Undetermined

Table 6: Alternatives Synopsis and Secondary Screening

Alternative	Construction Cost	Advantages	Disadvantages	Recommendations
1. Rock / Rip-Rap Breakwater	$63,158,000	▪ Commonly designed / constructed alternative ▪ Provides some protection in more extreme storms	▪ Exceeds soil bearing capacity ▪ Relatively large armor stone required ▪ High potential for scour problems ▪ Moderately high wave transmission during high energy events ▪ High cost	Not recommended for preliminary / final design.
2. Rock / Rip-Rap Revetment	$48,810,000	▪ Commonly designed / constructed alternative ▪ Soil bearing pressure lower than rock breakwater ▪ Directly armors the shoreline so no gaps needed and lower wave energy impact ▪ Moderate cost	▪ Exceeds soil bearing capacity ▪ Moderately high potential for scour problems ▪ Potential for undermining at crest of structure	Not recommended for preliminary / final design.
3. Steel Sheet Pile Breakwater (no scour protection in gaps)	$73,000,000	▪ Commonly designed / constructed alternative ▪ Relatively low wave transmission	▪ High potential for scour problems ▪ Corrosion of steel sheet pile will require frequent maintenance and may limit life span of structure ▪ Damaged sheet pile is difficult to repair / replace ▪ High cost	Not recommended for preliminary / final design.
4. Rock Reef Breakwater	$28,000,000	▪ Lower soil bearing pressure ▪ Increased water circulation ▪ Lowered potential for scour problems ▪ Smaller armor stone required ▪ Low cost	▪ Exceeds soil bearing capacity ▪ Not commonly designed / constructed alternative ▪ High wave transmission during more severe storms	Not recommended for preliminary / final design
5. Rock Reef Breakwater with Sheet Pile Wall	$51,578,000	▪ Low wave transmission. ▪ Provides some protection in more extreme storms ▪ Smaller armor stone required ▪ Decreased forces on sheet pile wall may allow non-steel cross-section to be used	▪ Exceeds soil bearing capacity ▪ Not commonly designed / constructed alternative ▪ Moderately high potential for scour problems ▪ Damaged sheet pile is difficult to repair / replace ▪ Decreased circulation possible during low tides ▪ High cost	Not recommended for preliminary / final design.

Table 6: Alternatives Synopsis and Secondary Screening

Alternative	Construction Cost	Advantages	Disadvantages	Recommendations
6. Rock Reef Breakwater with Articulating Block Mat Revetment	$61,000,000	▪ Lowered wave height at shore allows ABMs to be used ▪ Directly armors the shoreline so no gaps in ABM needed and lower wave energy impact ▪ Lower soil bearing pressure ▪ Decreased scour potential at revetment	▪ Exceeds soil bearing capacity ▪ Not commonly designed / constructed alternative ▪ Damaged ABM is difficult to repair / replace ▪ More severe storm (25, 50, or 100-year) conditions require a larger block size than is commercially produced ▪ High cost	Not recommended for preliminary / final design.
7. Rock Reef Breakwater with Rock / Rip-Rap Revetment	$60,000,000	▪ Lowered wave height at shore allows smaller armor stone to be used ▪ Directly armors the shoreline so no gaps in revetment needed and lower wave energy impact ▪ Decreased scour potential at revetment	▪ Not commonly designed / constructed alternative ▪ Soil bearing pressure exceeds allowable and is not significantly lower than traditional revetment ▪ Cost no lower than traditional revetment ▪ High cost	Not recommended for preliminary / final design.
8. Soil-Cement Mixing Column Wall without Face / Overtopping Protection	$48,000,000	▪ Gravity type structure with massive weight ▪ Directly armors the shoreline so no gaps needed and lower wave energy impact ▪ Moderate cost	▪ Experimental type design, not previously constructed ▪ Exceeds allowable shear stress in soil ▪ High potential for scour problems ▪ High variability in material properties of soil cement mixture is expected ▪ Unknown ability to withstand daily wave attack ▪ Relatively low crest allows wave overtopping during storms	Not recommended for preliminary / final design.
9. Sand Fill without Retaining Structures	$50,000,000	▪ Dissipates wave energy rather than reflecting it ▪ Lower scour potential ▪ Mimics natural system ▪ Moderate cost	▪ Sand may mix with underlying mud ▪ Sand fill will migrate out of project area ▪ Uncertainty in rate at which sand fill will migrate out of project area and rate of shoreline retreat	Not recommended for preliminary / final design; however, may become economically feasible with lower sand cost and may be fairly effective at slowing shoreline erosion. Could be considered for lowered design criteria.

Table 6: Alternatives Synopsis and Secondary Screening

Alternative	Construction Cost	Advantages	Disadvantages	Recommendations
10. Sand Fill with Reef Breakwater Sill (Perched Beach)	$54,000,000	▪ Dissipates wave energy rather than reflecting it ▪ Lower scour potential ▪ Mimics natural system ▪ Sill reduces amount of sand fill required and rate at which sand migrates out of project area	▪ Sand may mix with underlying mud ▪ Sand fill will migrate out of project area ▪ Uncertainty in rate at which sand fill will migrate out of project area and rate of shoreline retreat ▪ High cost	Not recommended for preliminary / final design; however, may become economically feasible with lower sand cost and may be fairly effective at slowing shoreline erosion. Could be considered for lowered design criteria.
11. Sacrificial Clay Fill without Retaining Structures	$95,000,000	▪ Dissipates wave energy rather than reflecting it ▪ Lower scour potential ▪ Mimics natural system	▪ Uncertainty in rate at which fill and the shoreline will retreat ▪ High cost	Not recommended for preliminary / final design.
12. Gravel / Crushed Stone Fill without Retaining Structures	$54,000,000	▪ Dissipates wave energy rather than reflecting it; however, more reflective than sand fill ▪ Lower scour potential ▪ Small stone size	▪ Gravel may mix with underlying mud ▪ Gravel fill may migrate out of project area ▪ Uncertainty in rate at which sand fill will migrate out of project area and rate of shoreline retreat ▪ High cost	Not recommended for preliminary / final design.
13. Reef Breakwater with LWA Core	$43,690,000	▪ Lower soil bearing pressure ▪ Increased water circulation ▪ Lowered potential for scour problems ▪ Smaller armor stone required ▪ Moderate cost	▪ Not commonly designed / constructed alternative ▪ High wave transmission during more severe storms ▪ Uncertainty in affect of LWA core on armor stone stability ▪ Minimal armor stone cover	Recommended for consideration for preliminary / final design.
14. Concrete Panel Breakwater	$37,291,000	▪ Better wave energy dissipation under higher energy conditions ▪ Lower bearing pressure, within allowable limits, and settlement ▪ Moderate cost	▪ Moderate wave energy dissipation under day-to-day conditions ▪ Not commonly designed / constructed alternative ▪ Moderate potential for scour problems ▪ Damaged panels are difficult to repair / replace ▪ Relatively complex construction	Recommended for consideration for preliminary / final design.

Table 7: Consideration of Additional Alternatives

Alternative	Description and Findings
1. Hesco basket gabions	As stand alone structures gabions are not believed to be stable under the more severe storm conditions of the open Gulf. However, gabions baskets may be viable for encapsulation of lightweight aggregate or other core materials. This would not likely affect soil bearing pressures. Gabions may be further considered as an alternative in final design of any gravity based structures.
2. Concrete raft foundation for rock breakwater alternative(s)	The soil load under breakwater alternatives would be distributed across wider base using a concrete raft foundation. Cost of raft foundation is believed to exceed project budget.
3. Pile supported rock breakwater	Piles would be used to support a portion of the vertical load from a rock breakwater. Cost of piles and rock foundation is believed to exceed project budget.
4. Rock filled crib structure	A vertical, rock-filled crib structure would be supported by piles. Piles would also transfer lateral wave load to stiff clays at deeper depth. Cost of piles is believed to exceed project budget.
5. A-frame concrete panel breakwater	Concrete panel would be suspended from a pile / batter pile pair. Cost of piles is believed to exceed project budget.
6. Sheet pile toe encapsulation on rock breakwater	Low strength (e.g., PVC) sheet piles driven at both toes of a breakwater would be used to contain lateral soil displacement. May help with settlement of breakwater structure.
7. Cellular sheet pile caisson	Sheet piles driven to create soil filled caissons. Cost of sheet piles is believed to exceed project budget.
8. Floating breakwater	Floating structure moored to the seabed reflects some of wave energy. Floating structures are generally ineffective at reflecting or dissipating longer period waves which, in the Gulf, are associated with storms. Storm tides would increase the draft under the structure and increase wave transmission. In addition, mooring the structure would be problematic. These structures may not meet the erosion and stability criteria for the project. Installation would be relatively experimental since little testing has been done and available design methodology is lacking. Also, the cost may exceed the construction budget.
9. Pre-consolidation or pre-loading of soils for breakwater or revetment	Stiff clays would be dredged from offshore and used to create a berm either onshore or just offshore. The berm would displace some of the softer soils (e.g., mud wave). The remaining soft soils under the structure would be consolidated. After soil displacement and/or primary settlement has occurred over approximately one year, some of the berm would be removed and replaced with a breakwater or revetment. It is preliminarily estimated that berm construction would add approximately $300 to $500 per foot to the construction costs.

Note: The alternatives described in this table were evaluated after submittal of the Feasibility Study report.

5. FINAL DESIGN

Objectives

As described in the previous chapter, based on the results of the Feasibility Study and subsequent supplemental analysis, five alternatives were selected by the project team for further consideration. These alternatives were as follows:

(1) Beach fill with gravel/crushed stone

(2) Reef breakwater with sand or gravel/crushed stone beach fill

(3) Reef breakwater with LWA core

(4) Concrete panel breakwater

(5) Soil pre-loading

This chapter documents final evaluation of these alternatives and recommendations for including each in a prototype test installation. The test installations will allow detailed evaluation and comparison of each alternative in terms of constructability, ability to deal with the soft soils, wave attenuation, shoreline response, cost, maintenance requirements, and aesthetics. Evaluation of the test installations will serve as the basis for implementation of the full 9.2 mile project.

Because the stability of each test alternative is limited by the soft foundation soils, the original design criteria have been somewhat relaxed. For final design, the ability of each alternative to prevent or significantly reduce erosion under a typical Category 1 hurricane remained a target. However, providing stability under the conservative 100-year storm characteristics provided by FEMA and ACRE was found to not be feasible in all cases. Therefore, the goal of the test sections is to provide as much stability as possible given the soft soils and anticipated construction budget.

Although a 50% increase in the construction budget was considered during the feasibility phase to allow detailed evaluation of additional alternatives, the anticipated construction budget for the full 9.2 mile project remains at $38,000,000. As part of the testing program, competitive bids will be solicited from construction contractors so that relative costs for each alternative can be better evaluated. However, the cost per linear foot of each test section is expected to be greater than the cost per foot of a full 9.2 mile project due to the significant differences in project size. Therefore, the unit prices applied in development of the opinion of probable construction cost for each alternative are based on the smaller quantities required for the test sections. Unit prices are expected to be lower for a full 9.2 mile project. Final cost evaluation of each alternative with respect to the $38,000,000 construction budget will be performed at completion of the testing program.

For each alternative, estimates of soil consolidation and settlement by Fugro (2004) were applied to evaluate performance under the lower crest elevations that may exist at completion of a 20-year project life. As defined in this report, "total settlement" excludes instantaneous settlements associated with soil displacement during construction. Fugro and LDNR[4] estimated instantaneous

[4] Personal Communication, Dr. Rickey Brouillette, P.E., September 2005, Engineering Supervisor, Engineering and Design Section, Coastal Engineering Division, Louisiana Department of Natural Resources.

settlements to range from 6 to 24 inches, which will be offset through placement of an adequate quantity of material to achieve the initial crest elevations shown in the contract drawings. Total settlement also excludes loss of elevation associated with relative sea level rise (including regional land subsidence). Instead of explicitly including relative sea level rise in analysis of settlement-dependent performance (such as wave transmission), conservative rates of total settlement were applied. Although Fugro estimated that only about 10% of total settlement would occur over 20 years, up to 100% of total settlement was applied, which is expected to account for relative sea level rise. A description, analysis, and cost of each alternative and the overall testing program are provided below.

Beach Fill

Two of the proposed test methods involve beach fill. For stand-alone beach fill, gravel or crushed stone (G/CS) were proposed in lieu of sand because G/CS would be more stable than sand, especially during storms. To achieve the desired 20-year project life and storm protection through stand-alone beach fill with sand, a relatively large placement density would be required, possibly on the order of 200 yd^3/ft of shoreline. Placement of this quantity of sand would exceed the available project budget even if a relatively cost-effective borrow source were available. For example, approximately 1,750,000 yd^3 of sand was recently dredged from an offshore paleo-channel and placed for the Holly Beach restoration project (Coastal Planning and Engineering 2000) at a cost of about \$7/yd^3. If sand from a similar borrow source could be located and exploited for Rockefeller Refuge at identical unit cost, the cost for placing 200 yd^3/ft would be \$1,400/ft, or over \$68,000,000 for a full 9.2 mile project. Actual dredging costs would likely be greater given the remoteness of the Refuge and its greater distance from a harbor.

For the quantity of sand that would be available with the given project budget, sand is not expected to provide adequate storm protection or project longevity unless applied in combination with retention structures. Of the three alternatives involving breakwaters, the reef breakwater (without LWA core) is expected to allow the greatest wave transmission and would benefit most from the secondary protection offered by smaller-scale beach nourishment. Note that, for application of breakwaters without beach nourishment, downdrift impacts through interruption of sediment supply is not a significant concern considering the lack of sand within the existing littoral system. Therefore, for the test program, sand fill was only considered for application with a reef breakwater, discussed in the next section.

Although gravel (and cobble) beach fills have been successful at several locations around the world, no projects are known where the material has been placed on soft clay. The primary difference between gravel and crushed stone is that crushed stone would typically consist of more angular particles that would interlock more than gravel and may result in a slightly more stable mass. A disadvantage of crushed stone is that, as a mixture, it may have less porosity, resulting in a greater mass unit weight and more settlement. Although both options were initially considered in order to promote competitive bidding and lower prices from contractors, LDNR's preference for crushed stone[5] to maximize hydraulic stability led to the exclusion of gravel from the final construction documents.

[5] Personal Communication, Mr. Herbert Juneau, P.E., October 12, 2005, Lafayette Field Office, Coastal Engineering Division, Louisiana Department of Natural Resources.

As depicted in Figure 9, the proposed cross section for the test program would provide a G/CS beach having a berm width of approximately 65 ft at elevation +2.0 ft. The berm width was set as the maximum possible without covering the adjacent reef breakwater, as described in the next section. For a full 9.2 mile project, the berm width could likely be less since the influence of end losses would be less (refer to end loss discussion on the following page). The dune-like feature landward of the berm has been given the term "backstop" and is included to reduce wave overtopping and associated landward dispersal of the fill material during smaller storms. The crest elevation of the backstop is +6.0 ft. Including the backstop, the placement density is about 17.5 yd^3/ft. Note that the dimensions for berm width and slope shown in Figure 9 represent the design (equilibrated) condition, not the construction template.

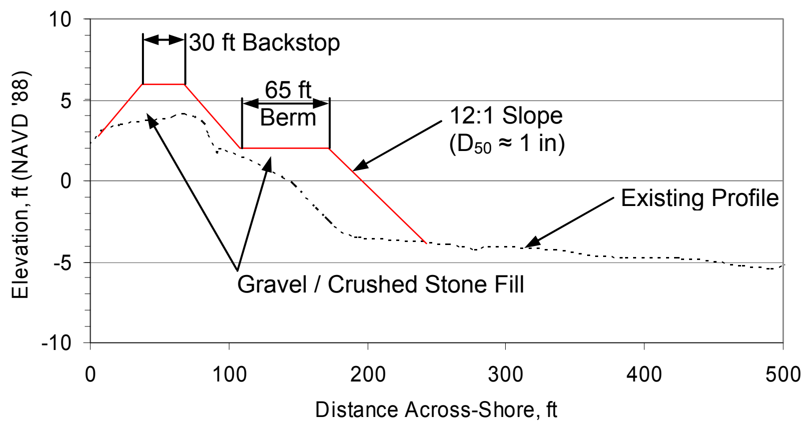

Figure 9: Typical Section of Gravel / Crushed Stone Fill Concept

Fugro estimated the total settlement of the fill to be approximately 2.1 ft for the backstop and 1.3 ft for the berm, providing final elevations of about +3.9 ft and +0.7 ft, respectively. However, the relatively coarse fill material is expected to migrate towards shore under prevailing larger waves (for background on cross-shore transport predictors, refer to Kraus *et al*. 1991). Given that the settlement is expected to occur over a period of decades, settlement of the berm may be counteracted by the tendency for the fill material to migrate onshore. Settlement of the backstop is more problematic since its primary purpose is to prevent dispersal of the G/CS landward across the marsh. Settlement should be carefully monitored during evaluation of the test section.

Based on equilibrium profile theory (Dean 2002) and a median particle size of approximately 1 inch, the submerged slope is estimated to be approximately 12:1 after profile equilibration. Based on the results of the geotechnical investigation performed by Fugro, construction slopes for the submerged and emergent (including backstop) portions of the fill will be specified as 10:1. Although the slopes could normally be constructed steeper, the risk of edge failure and associated slope stability failure due to the underlying soft clay requires placement within a flatter construction profile. A 10:1 slope will require a specified construction berm width of 70 ft, with a design width of approximately 65 ft being achieved after waves reshape the profile to its approximate 12:1 equilibrium slope.

In addition to cross-shore transport and settlement, longshore transport was recognized as a potential forcing mechanism for the test installation. Longshore transport is the process by which sediment is carried alongshore by breaking waves and shore-parallel currents. Longshore transport causes rapid spreading at each end of a beach fill as the fill responds to wave action and smoothly transitions into the adjacent beach. This adjustment results in “end losses” at each lateral end of the fill.

Along the Chenier Plain, prevailing southeast winds cause the net direction of longshore transport to be westward. Although no prior estimates of longshore transport rate along the Refuge are known, Underwood *et al*. (1999) and Mann and Thomson (2003) estimated rates on the order of 40,000 yd^3/yr for southwest Louisiana in Cameron Parish where beaches are characterized by fine sand. As discussed by Van Wellen *et al*. (2000b), methods for estimating longshore transport on coarse-grained beaches are somewhat limited.

A simple estimate of longshore transport of G/CS was developed by applying the CERC equation (USACE 1998) based on the assumption that a typical value of 0.77 for the sediment transport coefficient, K, would produce a rate of 40,000 yd^3/yr *along a sand beach* at the Refuge. An estimate of 3,000 yd^3/yr for a G/CS beach was then calculated with the CERC equation by changing the K value to 0.1 and holding other factors constant. The smaller K value was based on the sediment diameter as presented in Dean (2002). The 3,000 yd^3/yr estimate should be conservative (on the high side) since the CERC equation is calibrated for suspended transport of fine sands rather than the predominant bedload transport that occurs for gravel or crushed stone.

The strong relationship between project longevity and initial length presents a considerable challenge in representing a full 9.2-mile beach fill with a shorter test installation. The performance of a shorter test section will be more influenced by end losses than a longer project. Therefore, to assess the validity of simulating a relatively long project with a shorter test section, an idealized evaluation of fill longevity was performed through application of the Pelnard-Considere (P-C) equation (Dean 2002). Although this approach does not consider the lack of an updrift source of material that exists along the Refuge shoreline, it provides general insight regarding the spreading of the fill alongshore and the relationship between fill geometry (especially length) and longevity. Because there is no updrift source of compatible beach material at the Refuge, the centroid (in plan view) of the actual fill would likely migrate downdrift, a result not included in the P-C solution.

Figures 10 through 12 depict the results of the P-C modeling for initial fill lengths of 500 ft, 1.2 miles, and 5 miles, respectively. For each case, the initial fill width was 65 ft based on the cross-section shown for G/CS in Figure 9. In addition, the following conditions were applied to produce a net longshore transport rate of 3,000 yd^3/yr:

- A value of 0.1 was selected for the sediment transport coefficient, K, based on a median particle size of 1 inch;
- A value of 1.5 ft was selected for the long-term average breaking wave height;
- A value of 8 ft was selected as the closure depth; and
- A value of 2 ft was selected for the berm height.

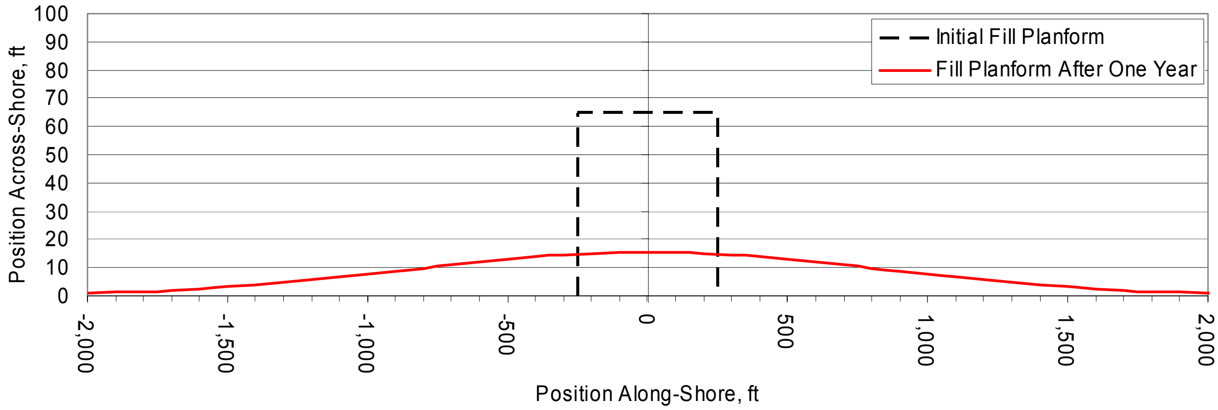

Figure 10: Generalized Beach Fill Evolution (500 ft Fill)

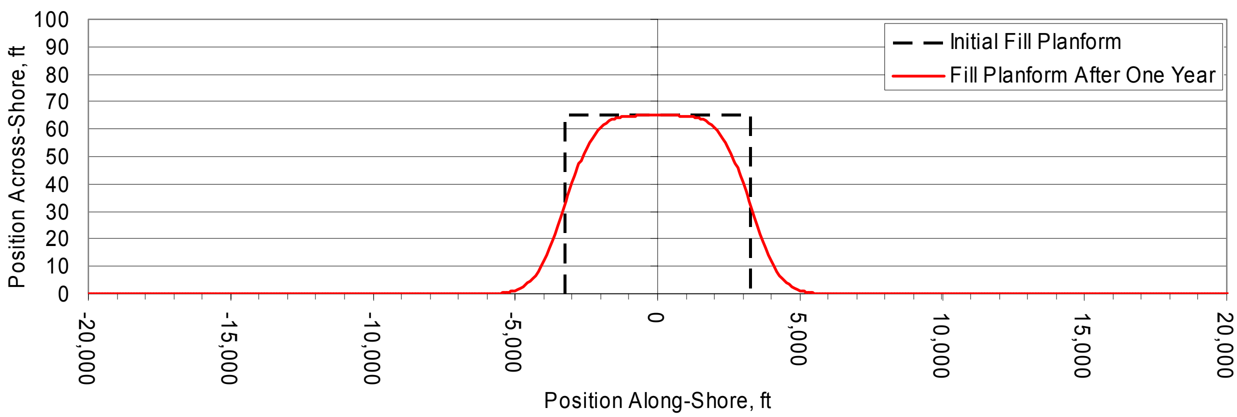

Figure 11: Generalized Beach Fill Evolution (1.2 mile Fill)

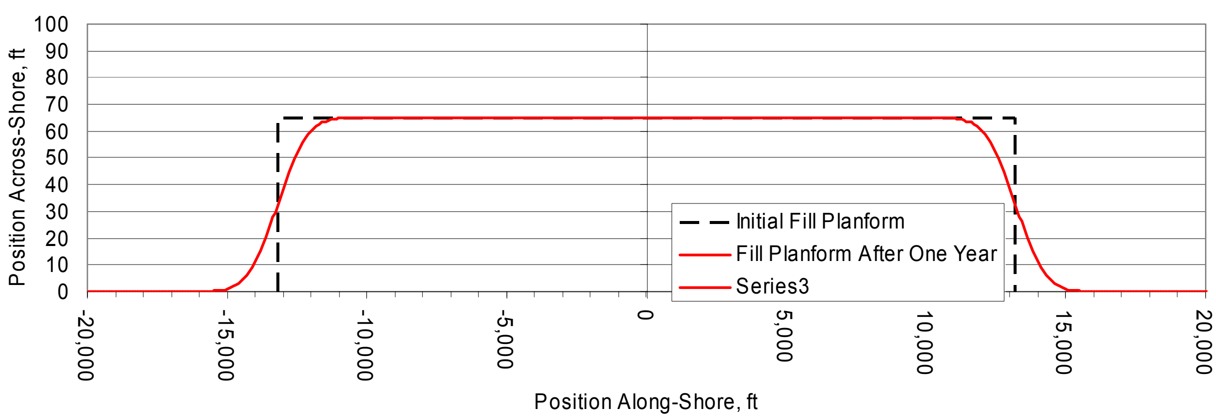

Figure 12: Generalized Beach Fill Evolution (5 mile Fill)

Figures 10 through 12 support the concept that the longevity of the project is very dependent on its length. For the one-year simulations shown, the initial 65 ft fill width for the 500-ft long project in Figure 10 is largely influenced by end losses; as a result, the design width is not maintained. As depicted in Figure 11, a project length of approximately 1.2 miles would be required in order to maintain a berm width of 65 ft at the center of the project. In order to simulate a large-scale project, such as the 5 mile project depicted in Figure 12, the fill would need to be contained at each end to hinder spreading and end losses. Therefore, to avoid an unfeasibly-long test section, a terminal groin is recommended for construction at each lateral end of the test fill. Although the soft soils will limit the allowable weight and associated height of the groins, they are still expected to hinder spreading and reduce the required length of the test section. Additional analysis and recommendations for length of the beach fill test section are provided at the end of this chapter.

The limited prior applications and research related to G/CS beach fills makes prediction of project performance under storm conditions challenging. For sand beaches, several predictive tools exist for estimating cross-shore transport during storms, including the SBEACH model (Larson and Kraus 1989) supported by the USACE. Preliminary application of the SBEACH model for the fill configuration shown in Figure 9 was performed by applying a median grain size of 1 mm, which corresponds to medium sand and is the maximum allowed in the model. The model predicted little cross-shore transport under the Category 1 storm conditions summarized in Table 2, a result that was judged to be unrealistic. Alternative approaches such as Powell (1990) and van der Meer (1989) provide parametric models for assessment of cross-shore profile response of coarse-grained beaches to storms. However, the methods rely on a number of simplifying assumptions and also yielded unrealistic results when applied to the conditions being considered for the Rockefeller Refuge. Research to improve prediction of cross-shore profile response of coarse-grained beaches is ongoing (Van Wellen *et al.* 2000a, Meigs *et al*. 2004).

Observations by Shiner Moseley of a cobble beach fill constructed along Corpus Christi Bay in 1989 and by Allan *et al*. (2003) of a cobble beach fill constructed along the Pacific coast of Oregon in 2000 have shown that such projects can perform well under storm conditions. Downie and Saaltink (1983) describe another successful cobble beach fill constructed along the Gulf of Georgia in British Columbia, Canada. Additional examples and considerations for coarse-grained beaches are provided by Van Wellen *et al*. (2000b). Ultimately, monitoring of the test installation will be the best approach for evaluation of cross-shore transport.

Both gravel and crushed stone are available from numerous sources with much of that placed in southern Louisiana being shipped by barge down the Mississippi River from Arkansas or Missouri. As provided in Appendix B, an opinion of probable construction cost (cost) analysis was performed for the G/CS beach fill. The construction cost was estimated to be approximately $1,656,000.

Reef Breakwater with Beach Fill

Rock breakwaters have been constructed extensively throughout the world with several examples along the Gulf coast of Louisiana being at Raccoon Island, Grand Isle, Holly Beach, and Cheniere Au Tigre. These sites differ from Rockefeller Refuge in that they do not have a 40-ft thick layer of very soft clay as foundation soil. As discussed extensively in the Feasibility Study report, a conventional rock breakwater is not a feasible option at the Refuge due to the soft soils being unable to support the relatively large bearing pressure of the breakwater.

As an alternative, a rock reef breakwater was proposed. Reef breakwaters are rubble mounds of a rock size similar to that found in the armor and/or first underlayer of conventional breakwaters. Reef breakwaters are typically constructed without underlayers or a core of smaller stone and are broad crested in comparison to conventional breakwaters. By traditional definition, reef breakwaters are designed to adjust in cross-section in response to the waves and currents at the site, a characteristic not being considered here. Low, broad crested breakwaters are designed to decrease the wave energy impacting the shoreline, but still allow some wave transmission under day-to-day conditions.

Although reef breakwaters are lower than conventional breakwaters, their much broader crest helps promote breaking and attenuation of waves that may otherwise pass over and through the structure with minimal energy reduction. At the Refuge, the key benefit is reduced bearing pressure on the soil. The first reef breakwater approach discussed herein involves construction of a sand or G/CS beach fill landward of the breakwater. The beach fill is intended to absorb remaining smaller waves that are transmitted across the breakwater.

A typical section of the first reef breakwater concept is shown in Figure 13. As discussed in more detail later in this chapter, all breakwater alternatives will be located along the approximate -4 ft contour approximately 150 ft offshore. The design process yielded a structure having an initial crest elevation of +1.0 ft and crest width of 24 ft. As estimated by Fugro, total settlement may be approximately 1.2 ft, eventually reducing the crest elevation to approximately -0.2 ft.

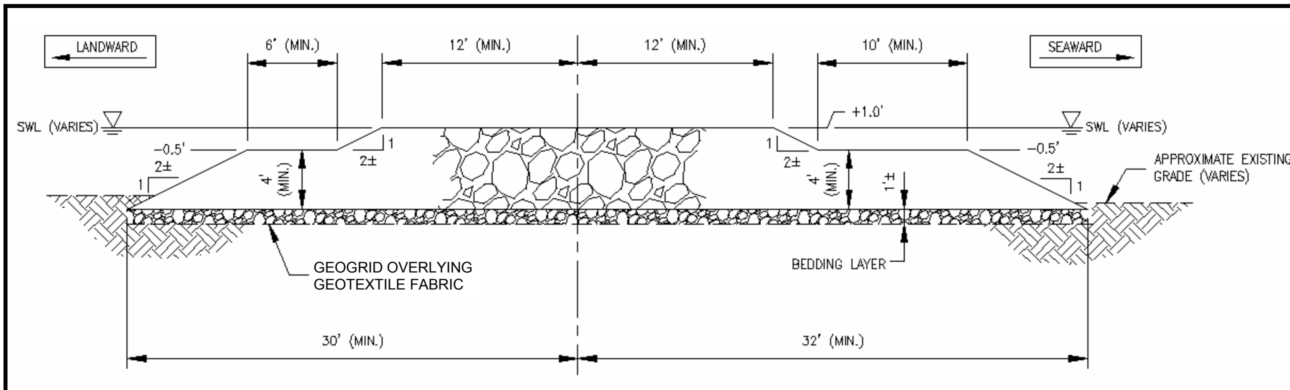

Figure 13: Reef Breakwater Concept

Sizing of the armor stone was based on the methodology of van der Meer for statically-stable submerged breakwaters as described in CUR Report 169 (Centre for Civil Engineering Research and Codes 1995). The calculations were performed for increasing surge elevations up to +13 ft based on the conservative estimate for a 100-year storm surge provided by FEMA (see Table 2). A structure height of 7 ft was applied, which assumes a crest elevation of +1.0 ft (prior to settlement) and 2 ft of seabed lowering seaward of the breakwater. The controlling case was a +1.0 ft surge elevation where the crest of the breakwater was at the still water level and the maximum wave height was 5.5 ft. The stone stability analysis resulted in a required median stone weight of approximately 1,040 lbs for this case. The stability calculations were based on a stone unit weight of 155 lb/ft^3, which is the minimum anticipated for most commercial sources that provide limestone to the western Gulf coast.

To gauge the adequacy of the calculated armor stone size, a comparison was made to the size of armor stone placed at conventional-type breakwaters constructed in Louisiana at sites having Gulf exposure, including Holly Beach, Grand Isle, Raccoon Island, and Cheniere Au Tigre. These breakwaters have been in place for several years and have withstood hurricanes with minimal damage. The breakwaters at Holly Beach, Grand Isle, and Raccoon Island were constructed with stone gradations having minimum median stone weights ($W_{50(min)}$) of approximately 4,300 lb, 3,100 lb, and 1,000 lb, respectively, but the structures have considerably greater crest elevations than the reef breakwater being considered here and would be exposed to greater wave loads. The Cheniere Au Tigre breakwater was constructed with a stone gradation having a $W_{50(min)}$ of only 300 lb.

Based on the stone stability calculations and comparison to existing breakwaters, a standard 1,000 lb Riprap Class gradation in accordance with the Louisiana Department of Transportation and Development's Standard Specifications for Roads and Bridges (2000) was selected. This gradation is the largest commercially-available standard riprap gradation, allowing a median stone size ranging from 1,000 lb to approximately 2,000 lb and a maximum stone size of 5,000 lb. This gradation is nearly identical to a standard 5,000 lb "clean" riprap gradation commonly produced for the USACE and that was placed at Raccoon Island. Although most quarries would generally produce this gradation through hand-picking instead of mechanical screening, cost is still expected to be less than a custom (narrow) gradation or uniform armor stone. Placed in two tiers, this gradation would provide a total layer thickness accommodating the cross-section shown in Figure 13.

A 12-in thick bedding layer will be included to prevent damage to the underlying geotextile layer during placement of the stone. The geotextile layer will include filter fabric and geogrid to further help distribute the weight of the stone and reduce punctures. Minimization of drop height during stone placement will reduce soil displacement and mudwaving.

As with all structures that reflect some wave energy rather than completely dissipating it, localized scour on the seaward side of the structure may occur. The scour potential on mud seafloors is particularly high, but very difficult to predict. Lowering the wave reflection and minimizing a down rushing water jet, which occurs as a wave trough approaches the structure, from impacting the native sediments are effective methods of reducing scour. In the case of the reef breakwater, the low and broad crest of the structure dissipates wave energy and reduces the reflection and scour potential. The toe of the breakwater structure has been configured to minimize scour during most conditions; however, severe storms such as hurricanes may cause some scour damage.

The crest elevation and width of the reef breakwater was evaluated in terms of wave attenuation and shoreline protection during storm events. The estimate of whether the shore will erode under given wave and tide conditions was based on the transmitted wave height. Wave transmission over and through the structure was estimated using the methodologies of Seabrook and Hall (1998), Briganti *et al*. (2003), Calabrese *et al*. (2003), and Friebel (2000). These methodologies may under-predict wave transmission since they were developed based on tests of breakwaters having a core of smaller stone that would decrease permeability. The reef breakwater being considered here would not have a core of smaller stone due to limited space within its cross section.

Table 8 contains estimates of wave transmission for a variety of wave and tide conditions. For each case, the bottom elevation was assumed to be -6.0 ft to account for two feet of seabed lowering seaward of the breakwater over the 20-year life of the project, and the breakwater crest elevation was assumed to be at 0 ft to account for settlement.

Table 8: Wave Transmission Estimates for Reef Breakwater

Parameter	Return Interval		
	day-to-day	1 Year	Category 1 Storm (Approx. 9-12 Years)
Still Water Elevation, ft	+1.5	+2.3	+5.0
Transmission Coefficient, K_t	0.45	0.41	0.56
Significant wave height seaward of structure, H_s (ft)	1.6	4.2	5.5
Significant wave height landward of structure, H_s (ft)	0.7	1.7	3.1
Energy Reduction (%)	81	84	70

During a 1-year storm, the breakwater is anticipated to reduce the wave energy landward of the structure by approximately 84% with a significant wave height of 1.7 ft being transmitted (compared to 4.2 ft seaward of the structure). Both the sand and G/CS fill options would likely be stable under this condition, and the backstop would help prevent the smaller transmitted waves from propagating into the marsh.

During a Category 1 storm, the structure is anticipated to reduce the wave energy landward of the structure by approximately 70% with a significant wave height of 3.1 ft being transmitted (compared to 5.5 ft seaward of the structure). Under this storm condition, some erosion of the backstop is likely to occur, especially if sand were placed as the fill material. Considering the estimated final backstop elevation of +3.9 ft after total settlement, as discussed earlier, wave overtopping would probably transport some of the fill material landward into the marsh. The relatively flat (10:1) side slopes of the backstop would help its stability.

Sand Beach Fill

A sand beach fill placed in combination with the reef breakwater would provide an approximate representation of the natural shoreline conditions seen both east of the Refuge at Cheniere au Tigre, where portions of the shore are reportedly accreting with marine clays (Huh *et al*. 2001), and west of the Refuge at Holly Beach, where the shore is erosional, but at a much lower rate than at the Refuge (Byrnes *et al*. 1995). Sand would assume a profile in equilibrium with the wave climate at the site and would dissipate much of the wave energy rather than reflect it, thus mitigating the scour problems associated with structural alternatives.

As previously discussed, if a nearby sand source could be located offshore of the project site, as was done at Holly Beach, dredged sand could be a relatively cost-effective alternative to G/CS. ACRE (2002) suggested that the best potential offshore borrow sites for Rockefeller Refuge were approximately 35 miles to the west at paleo-channels and reworked shoreline sands in the vicinity of Holly Beach and Calcasieu Pass. These sites were previously identified by Suter and Penland (1985) and reported to contain sediments composed of only about 50% sand, with mean grain size being approximately 0.06 mm. These sites are not considered feasible for Rockefeller Refuge based on their distant location and high percentage of fine-grained sediments. An investigation of other potential offshore sand sources closer to the Refuge as mapped by Suter et al. (1987) was also considered. The cost for seismic surveying and reconnaissance-level vibratory coring to explore these areas was estimated by the Coastal Research Laboratory at the University of New Orleans to be approximately $300,000[(6)]. For reasons explained below, this field work was not performed.

As depicted in Figure 14, the location of the reef breakwater offshore would leave little room for sand fill without burying the breakwater. Based on equilibrium profile theory (Dean 2002) and a median grain size in the range of 0.15 mm to 0.20 mm, the submerged portion of the fill is expected to have a slope of approximately 45:1 after profile equilibration, intersecting the breakwater at an elevation of approximately -1 ft and providing an added berm width of only about 20 ft. In addition to concerns regarding the ability of a relatively narrow berm to accommodate a 20-year project life, the intersection point of -1 ft NAVD between the fill and the breakwater is relatively high and could allow excessive loss of fill over the breakwater and through its gaps. Considering the lesser stability of sand compared to G/CS under storm conditions, the uncertainty and cost of locating a feasible borrow source containing adequate-quality sand, and the limited space landward of the breakwater, sand was not selected for placement with the reef breakwater alternative.

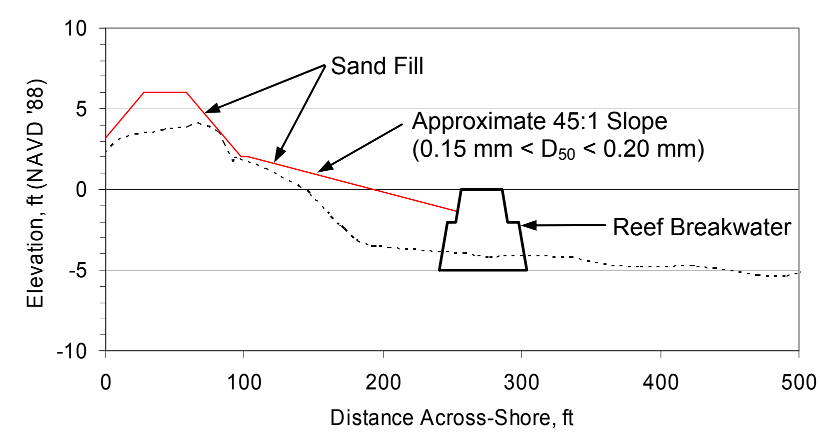

Figure 14: Typical Section of Sand Fill with Reef Breakwater Concept

[6] Personal Communication, Dr. Mark Kulp, September 6, 2002, Coastal Research Laboratory, Department of Geology and Geophysics, University of New Orleans.

Gravel / Crushed Stone Beach Fill

As a more stable alternative to sand fill, G/CS was considered in combination with the reef breakwater. As depicted in Figure 15, the larger particle size of G/CS provides a much steeper profile than sand. Applying the same beach profile shown in Figure 9, the submerged portion of the fill is expected to intersect the existing beach profile near the toe of the reef breakwater and provide a berm width of approximately 65 ft. Note that, as with the stand-alone beach fill alternative already discussed, for a full 9.2 mile project, the berm width could likely be less since the influence of end losses would be less. Based on the steeper profile and greater stability, G/CS is recommended over sand for placement with the reef breakwater alternative.

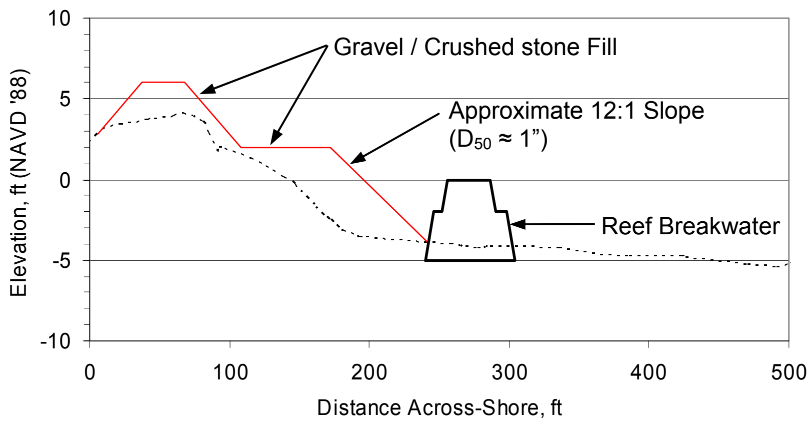

Figure 15: Typical Section of Gravel / Crushed Stone Fill with Reef Breakwater Concept

Both armor stone and G/CS are available from numerous sources with much of that placed in southern Louisiana being shipped by barge down the Mississippi River from Arkansas or Missouri. As detailed in Appendix B, the construction cost for the reef breakwater with G/CS beach fill alternative was estimated to be approximately $2,612,000.

Reef Breakwater with LWA Core

The reef breakwater with lightweight aggregate (LWA) core alternative replaces the rock core of the breakwater with an encapsulated lightweight expanded shale or clay product that is almost neutrally buoyant, decreasing the bearing pressure and allowing greater crest elevation and increased wave attenuation. The greater crest elevation is intended to eliminate the need for secondary protection via beach fill as provided in the previous reef breakwater alternative. A secondary benefit of the LWA core is lower permeability and less wave transmission through the structure, although armor stone stability may decrease with decreased permeability.

Reef breakwaters with LWA cores have been applied on soft clay soils in limited wave exposure areas at three recent projects designed in Louisiana[7], but no information has been located on such structures being constructed in a more aggressive wave climate. A potential weakness of this

[7] Personal Communication, Mr. Brad Sticker, National Resource Conservation Service, Alexandria, Louisiana.

alternative is that armor stone placed on the LWA core may not be stable under larger waves from the open Gulf of Mexico.

For the test program, the same standard riprap gradation selected for the previously-discussed reef breakwater was chosen for the reef breakwater with LWA core. However, depending on success of the test section, the reef breakwater with LWA core may require application of a custom riprap gradation providing a greater W_{50} for the full 9.2 mile project. Challenges associated with predicting stability of the reef breakwater with LWA core include the following:

- Available guidance on design of reef breakwaters is limited and does not include possible destabilizing effects of a LWA core.
- The added height of the reef breakwater with LWA core will result in less wave energy transmitted over the structure and associated greater wave loads on the armor stone. Hydraulic stability will increase as the structure settles and the crest elevation decreases.
- The less-permeable core may further decrease hydraulic stability of the reef breakwater with LWA core, requiring larger armor stone than the previously-discussed reef breakwater for equivalent crest elevations.
- Depending on the actual median stone size provided, limited space within the armor layer may make placement of two tiers of stone challenging. Although more expensive to process at a quarry, a narrower (more uniform) gradation of armor stone may be needed to better accommodate two tiers of stone. A narrower gradation would also help minimize void space between the individual stones, reducing exposure of the encapsulated LWA to puncturing by wave-borne debris and geotextile degradation from ultraviolet radiation.

A typical section of the reef breakwater with LWA core is shown in Figure 16. As discussed in more detail later in this chapter, all breakwater alternatives will be located along the approximate -4 ft contour approximately 150 ft offshore. The height of the LWA core was set as the minimum required to achieve bearing pressure requirements while maximizing thickness of the overlying armor layer. The design process yielded a structure having a 3.75-ft high LWA core to be initially covered by approximately 4 ft of armor stone, resulting in an initial crest elevation of +3.25 ft. Fugro estimated that settlement will lower the crest elevation to approximately +1.9 ft over several decades. The structure would have a crest width of 18 ft. Again, note that the less-permeable core and approximate 2.25 ft greater crest elevation of the reef breakwater with LWA core will result in less stability than the previously-discussed reef breakwater.

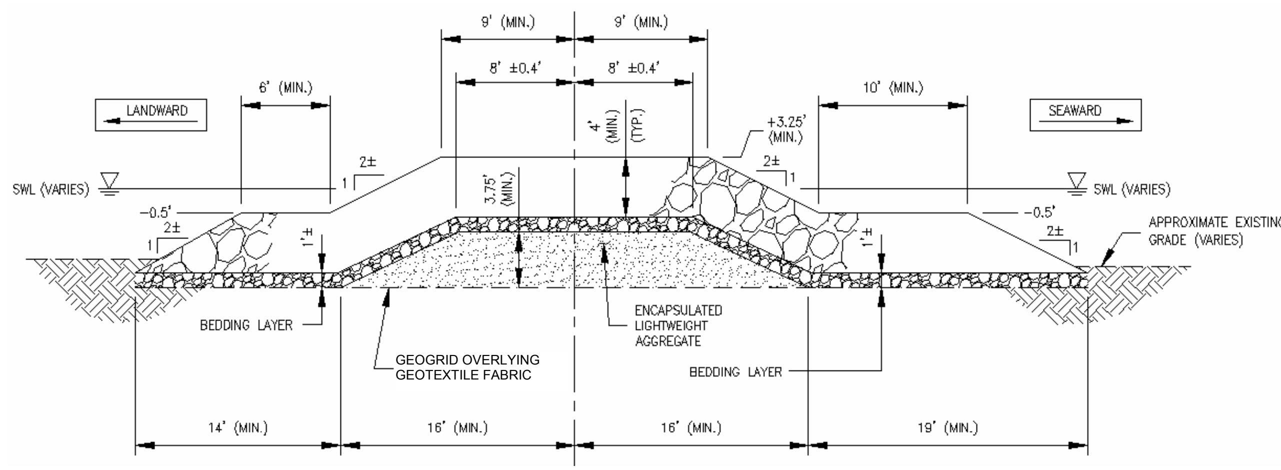

Figure 16. Reef Breakwater with LWA Core Concept

The reef breakwater with LWA core is expected to be reasonably effective at protecting the shoreline from storm events with a 1-year return interval. To estimate whether the shore will erode, a 1 ft erosion threshold typical for spartina marshes (USACE 1995, Shafer *et al*. 2003) was applied. For cases where the breakwater will be completely submerged, wave transmission was estimated using the methodologies of Seabrook and Hall (1998), Briganti *et al.* (2003), Calabrese *et al.* (2003), and Friebel (2000). For cases where the crest will be emergent, the methodology of d'Angremond *et al*. (1996) was applied. Table 9 contains estimates of wave transmission for a variety of wave and tide conditions. For each case, the bottom elevation was assumed to be at -6.0 ft to account for profile lowering over the life of the project, and the breakwater crest elevation was assumed to be at +1.9 ft to account for settlement.

Table 9: Wave Transmission Estimates for Reef Breakwater with LWA Core

Wave Parameter	Return Interval		
	Day-to-Day	1 Year	Category 1 Storm (Approx. 9-12 Years)
Still Water Elevation, ft	+1.5	+2.3	+5.0
Transmission Coefficient, K_t	0.14	0.26	0.48
Significant wave height seaward of structure, H_s (ft)	1.6	4.2	5.5
Significant wave height landward of structure, H_s (ft)	0.2	1.1	2.6
Energy Reduction (%)	98	93	79

During a 1-year storm, the breakwater is anticipated to reduce the wave energy landward of the structure by approximately 93% with a significant wave height of 1.1 ft being transmitted (compared to 4.2 ft seaward of the structure). Under this storm condition, the marsh would have approximately 0.5 ft of water above it. It is estimated that these conditions will result in minor erosion during the storm.

During a Category 1 storm, the structure is anticipated to reduce the wave energy landward of the structure by approximately 79% with a significant wave height of 2.6 ft being transmitted (compared to 5.5 ft seaward of the structure). Under the Category 1 storm, the marsh would have up to approximately 3 ft of water above it so that much of the wave energy would pass over the soils. Under these conditions, erosion could still occur, but to a much lesser extent than would be expected without the breakwater.

It will be helpful and important to encourage marsh vegetation to grow out into the water to provide additional protection from the wave energy. Marsh vegetation both absorbs wave energy and shields the soil from the waves' erosive effects. Deposition of sediments, especially shell hash, may occur between storm events; however, it is not known whether the deposition of sediment will be sufficient to balance out erosion during storm events.

The LWA material is manufactured at several locations around the country; previous projects in southern Louisiana have used material manufactured in Texas and shipped to the site by barge. It is anticipated that the LWA material will be placed in high-strength geotextile bags (3 cubic yard bags measuring 3 ft wide by approximately 2 ft high and 11 ft long have been used previously), sewn shut, and placed from a barge; however, the construction contractor will be allowed to propose alternate methods of encapsulating the LWA. Based on recommendations by personnel involved with prior LWA core projects in Louisiana, encapsulation of the LWA will be required prior to placement.

The construction process would begin by placing a geotextile fabric and plastic geogrid material on the seabed at the breakwater location. Bedding layer material such as stone or crushed concrete could then be placed on the seaward and landward toes to prevent movement of the fabric. Geotextile bags filled with lightweight aggregate would next be placed on the geotextile layer. The LWA core and bedding layer would immediately be covered with armor stone. As detailed in Appendix B, the cost for the reef breakwater with LWA core was estimated to be approximately $1,084,000.

Concrete Panel Breakwater

The concrete panel breakwater is a relatively unconventional structural approach that consists of concrete piles for vertical support, steel sheet piling for lateral resistance, a concrete cap, and concrete wall panels above the cap. For improved lateral support, some of the upper soft clay would be excavated and replaced with imported sand or gravel that would then be capped with armor stone for scour protection and additional lateral support. As explained in the Feasibility Study report, the initial concept was that the wall panels could be separated by relatively small gaps to let part of the wave energy through, but still significantly reduce the energy at the shoreline.

As part of preliminary design of the concrete panel breakwater, gap size versus the length of the wall panels was analyzed to determine the wave energy reduction. Several combinations were considered and it was found that a pattern of 5 ft gaps followed by 10 ft wall panels could provide the needed reduction in wave energy, and that sections of the breakwater could be pre-fabricated in nominal 50 ft lengths. This general approach has been carried forward for final design.

An important aspect to be considered was the design storm conditions. The initial project goals called for the concrete panel breakwater to protect the shoreline for up to a Category 1 hurricane and for the structure to be stable for up to a 100 year storm. To evaluate performance and stability of the wall under storm conditions, a key factor was crest height of the wall panels. A higher crest provides greater wave energy reduction at the shoreline, but will result in a larger wave load on the panels. As discussed below, it appears that a +5.0 ft crest elevation provides an acceptable balance between the desired reduction in wave height with allowable wave load. Typical section and elevation views of the concrete panel breakwater are shown in Figures 17 and 18, respectively.

In evaluating horizontal wave loads on the concrete panel breakwater, both landward- and seaward-directed forces associated with impacts by wave crests and troughs[8], respectively, were considered. Wave forces were calculated using the methodologies developed by Goda (1974, 2000). Conditions for up to a +9.8 ft storm surge were considered, with the maximum surge condition being the controlling case. Based on evaluation of historical tide records as presented in Table 1, a +9.8 ft surge has been exceeded only twice (by Hurricanes Audrey and Rita) along the open coast of Louisiana's Chenier Plain within the past approximate seventy years. For a +9.8 ft surge, a maximum (breaking) wave height of 12.3 ft, and a wall crest elevation of +5.0 ft, the landward and seaward design loads (averaged along the length of each 44.5 ft panel) were calculated to be approximately 5.1 kip/ft and 2.0 kip/ft, respectively.

The soft clays to a depth of approximately 40 ft will provide limited resistance to the lateral wave loads, and transferring the load to the deeper stiff clays would be too costly. Therefore, a portion of the upper soft clay will be replaced with imported backfill material to help provide the needed magnitude of lateral resistance. To provide a material that will consolidate relatively quickly and have an angle of internal friction of at least 30 degrees, the imported backfill material will be specified as gravel or well-graded sand containing less than approximately 10% of silt and clay-sized particles.

The concrete piles, cap, and wall panels were analyzed in accordance with requirements of ACI 318 (American Concrete Institute 2002) concrete design code with applied safety factors of 1.2 for dead loads and 1.6 for wave loads using the same load combination. The concrete piles will support the vertical load of the breakwater panel system and resist downdrag forces associated with settlement of the surrounding soil. Downdrag forces were evaluated based on the pile capacity curve provided by Fugro (2004).

The steel sheet piling was analyzed for bending and shear stresses in accordance with Allowable Stress Design (American Institute of Steel Construction 1989). Analysis of lateral wave loads on the sheet piling was performed with the LPILE model developed by Ensoft, Inc. (Reese and Wang 1997). LPILE is a soil-structure interaction program that evaluates the behavior of sheet pile under loading conditions and determines deflection, shear stress, bending moment, and soil response with respect to depth in nonlinear soils. Soil behavior is modeled with p-y curves internally generated by the program following published recommendations for various soil types.

[8] For a discussion on seaward-directed forces associated with wave troughs, refer to McConnell et al. (2000).

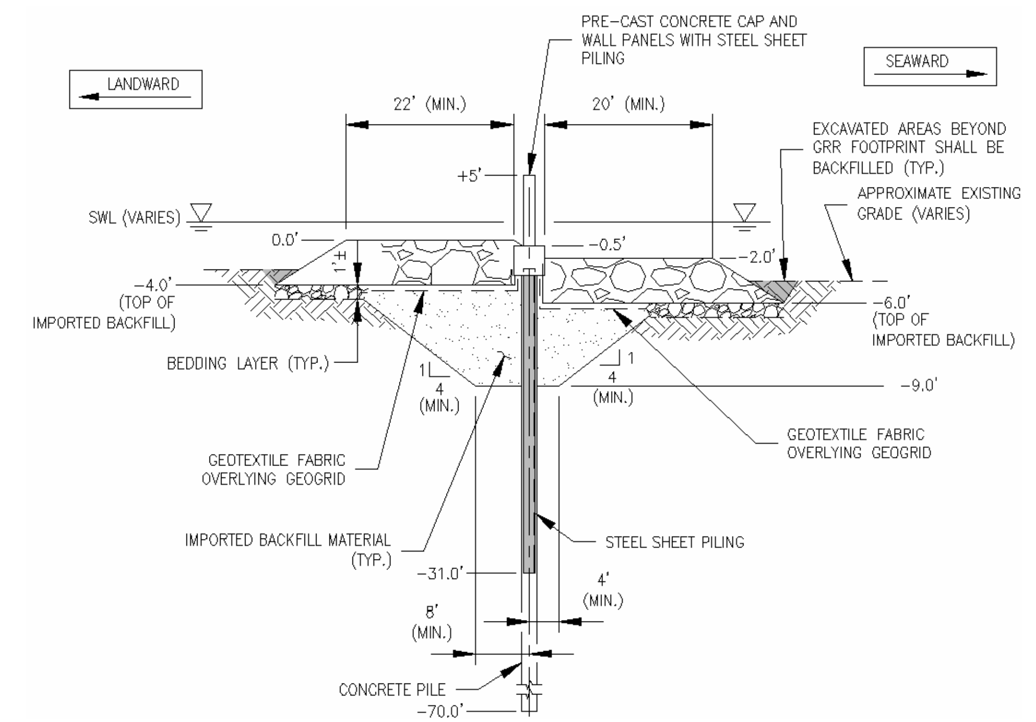

Figure 17. Typical Section of Concrete Panel Breakwater Concept

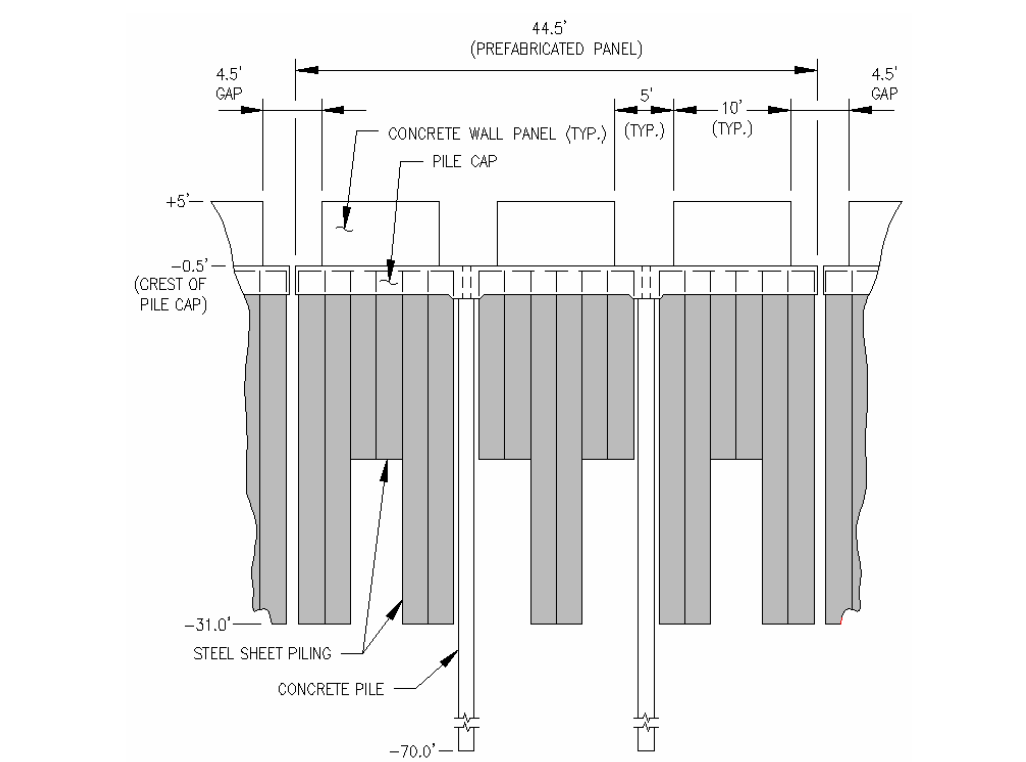

Figure 18. Typical Elevation View of Concrete Panel Breakwater Concept

Due to the soft upper soils that provide limited lateral resistance, deflection of the structure at the pile head and sheet pile tip, as opposed to overstressing the structural elements, was the controlling design condition. Lateral resistance at the pile head relies heavily on the imported backfill material and armor stone. Pile penetration to approximately -31 ft will be required to set the sheet pile tips at a nearly fixed position.

A limitation of lateral load analysis with LPILE was that the model assumed the upper layers of imported backfill material and armor stone would be infinitely wide. In reality, these layers will have limited width. If not placed wide enough, the imported backfill material and armor stone could be displaced laterally, resulting in excessive translation or failure of the wall. Because no known analytical method exists for determining the required placement width of the imported backfill material, a lateral load test has been proposed for a single 44.5 ft section of the wall prior to construction of the full test section.

To prevent excessive settlement of the structure, vertical piles will transfer the loads to the stiff clays below 40 ft. These piles will hold the wall at the correct elevation while the imported backfill material settles around it. Settlement becomes a non-issue except in predicting the depth of backfill material needed. Because the imported backfill material will weigh more than the clay it replaces, settlement of the backfill can be expected. Fugro has estimated the total settlement of the backfill to be on the order of 1.2 ft, depending on what type of material is placed.

Working in the surf zone is not conducive to driving steel sheet pile and forming and pouring concrete. Simply getting concrete mix to the site will take significant effort. It is proposed that the wall panels and cap be pre-cast offsite with the sheet piles in manageable sections of 44.5 ft length. These pre-fabricated sections of wall panels, cap, and sheet piles could then be transported to the site. After excavation of the soft clay needed to provide a trench for backfill placement, the concrete piles would be driven. The 44.5 ft sections would then be set on top of the concrete piles. As the cap is set on the concrete piles, the steel sheet piles are expected to cut into the soft clay without the need for driving. Once each section is set, the imported backfill would be placed and capped with armor stone matching the gradation and minimum 4 ft layer thickness selected for the reef breakwater alternatives. This approach would minimize the time that an uncompleted section would be exposed to waves.

A secondary benefit of the pre-cast concrete approach is corrosion resistance. Only the corrosion-resistant concrete would be located in the highly corrosive splash zone. The concrete would encapsulate the steel sheet piles to an elevation of -3 ft.

Another important aspect of the design was analysis of wave diffraction. Diffraction is the process by which energy spreads laterally perpendicular to the dominant direction of wave propagation. A simple illustration of wave diffraction is presented in Figure 19 in which waves propagate normal to a breakwater, with diffraction occurring on the sheltered side of the breakwater into the "shadow zone."

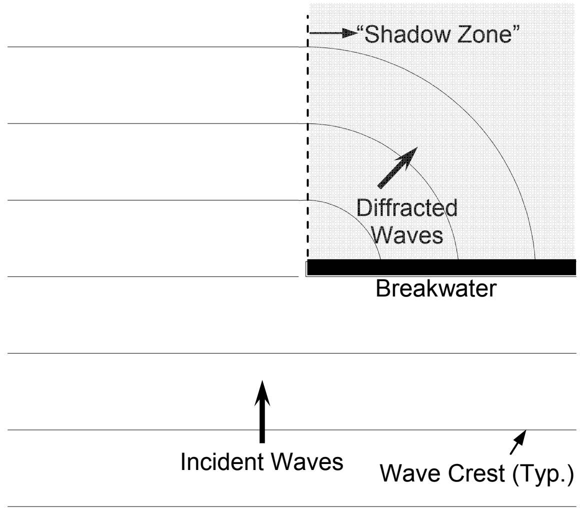

Figure 19: Idealized Wave Diffraction

As discussed in the Feasibility Study report, the gaps in the concrete panel breakwater are small compared to a typical wavelength, but the gap ratio is relatively large. Under these conditions a simple diffraction analysis to estimate wave transmission becomes less reasonable. Therefore, ACRE performed more detailed numerical modeling of the concrete panel breakwater alternative to estimate wave transmission through the gaps. The model employs a complex diffusion equation that describes wave diffraction and assumes constant water depth. Wave transmission due to overtopping was estimated by Shiner Moseley through methods presented in Goda (2000).

Table 10 shows the results of the wave transmission calculations. For the 1-year event, the 84% reduction in wave energy and transmitted wave height of 1.7 ft is a significant benefit. In the Category 1 storm, the reduction drops to 64%, which is also significant, but there are still 3.3 ft waves at the shore. As discussed for the reef breakwater with LWA core alternative, under the Category 1 storm, the marsh would have up to approximately 3 ft of water over it, reducing erosion potential. However, some erosion is still expected to occur.

Table 10: Wave Transmission Estimates for Concrete Panel Breakwater

Wave Parameter	Return Interval		
	Day-to-Day	1 Year	Category 1 Storm (Approx. 9-12 Years)
Still Water Elevation, ft	+1.5	+2.3	+5.0
Transmission Coefficient through gaps, K_t	0.4	0.4	0.4
Transmission Coefficient for overtopping, K_{ot}	0.0	0.1	0.4
Total Transmission Coefficient, K_{tot}	0.4	0.4	0.6
Significant wave height seaward of structure, H_s (ft)	1.6	4.2	5.5
Significant wave height landward of structure, H_s (ft)	0.6	1.7	3.3
Energy Reduction (%)	86%	84%	64%

Note that the wave transmission estimates for the concrete panel breakwater are similar to those for the reef breakwater without LWA core. Because the anticipated costs of these two types of breakwaters are similar for the full 9.2 mile project, it is possible that G/CS beach fill could also be applied in combination with the concrete panel breakwater to provide redundancy in protection during larger storms. As detailed in Appendix B, the cost for the concrete panel breakwater was estimated to be approximately $1,452,000.

Soil Pre-Loading

For the soil pre-loading alternative, an approximate 6-ft deep trench would be excavated parallel to the shoreline landward of the beach. The soft clay excavated from the trench would be placed in continuous berms on each side of the trench. The trench would then be overfilled with stiff clay imported to the site on barges. The stiff clay would be placed to an approximate elevation of +10 ft, creating a load that would exceed the underlying soft clay's bearing capacity and causing soil displacement due to punching failure (which results in mud waving) and edge failure (which results in slope stability failure). Over the following approximate 12 months, the fill would be left to partially consolidate.

For the full 9.2 mile project, the next step would be to remove an upper portion of the stiff clay and replace it with a rock revetment. To prevent additional soil failure, the revetment would be designed such that the bearing pressure of the rock would not exceed that of the removed clay. Note that construction of the revetment would not be included in the test project. For the test project, the imported clay would be specified to have properties similar to the deeper stiff Pleistocene clays located offshore of the Refuge so that, if eventually implemented for the full 9.2 mile project, the stiff clay could be hydraulically dredged at a much lower unit cost instead of imported on barges.

Note that soil pre-loading was selected in lieu of soil pre-consolidation since Fugro has indicated that soil consolidation is expected to occur very slowly (on the order of decades) for the soft clays at the site. Although wick drains could be added to increase the rate of consolidation, the cost of adding wick drains would cause this option to exceed the available budget for the full 9.2 mile project. Since consolidation dictates settlement, the soil pre-loading approach would still need to consider relatively large settlement similar to other alternatives evaluated.

The preliminary design involved evaluating the process of placing the imported clay, exceeding the underlying soil's bearing capacity, and causing it to fail. The goal was to be left with a relatively continuous mound of stiff clay having the height and width needed for later construction of a rock revetment. In addition, the potential for additional soil failure during rock placement and the settlement of the revetment over the next 10 to 20 years were considered.

During analysis, a key focus was the potential for a workable platform to be provided despite initial failure of the underlying soil. Recognition was given to the fact that the magnitude of failure is very dependent on the construction method and the post-failure shape of the mound would be challenging to predict. Different construction techniques could be specified for different segments of the test to evaluate whether rapid placement or slower placement in controlled lifts would be more effective.

To determine what post-failure shape and height of the mound would be required to accommodate subsequent construction of the rock revetment, an estimate of the required rock size was performed. Initially, the revetment would be exposed to relatively small waves that would partially dissipate across the shallow foreshore and beach. However, as summarized in Table 4 and depicted in Figure 7, significant lowering of the beach profile would occur over the desired approximate 20 year project life, eventually exposing the revetment to deeper water and larger waves. In addition, since the crest of the revetment would be relatively high as compared to the reef breakwater alternatives, less wave energy would pass over the revetment, subjecting the revetment to greater wave loads.

Prior to performing armor stone stability calculations for the revetment, a crest elevation needed to be selected. For setting the crest elevation, complete prevention of wave transmission over the revetment during a Category 1 storm was not considered feasible. Wave overtopping could damage some of the marsh directly behind the revetment, but not result in large-scale erosion since most of the wave energy would be absorbed as waves break on the revetment. Because the stability of the revetment could be influenced by overtopping, back-slope protection would be required, giving the revetment a trapezoidal cross-section similar to a conventional breakwater. A crest elevation of +7.5 ft was selected to provide approximately 1.8 ft of freeboard during a Category 1 storm.

Armor stone was sized using the methodology of van der Meer for statically-stable low-crested breakwaters as described in CUR Report 169 (Centre for Civil Engineering Research and Codes 1995). For a Category 1 storm, this approach yielded a required median stone weight of approximately 5,200 lbs for 2H:1V side slopes. Note that this stone size is much larger than was calculated for the reef breakwater alternatives. The larger stone size is a result of the greater crest elevation, which would reduce the amount of wave energy that passed over the revetment, increasing the wave loads.

Placed in two tiers, this stone size would require a total armor layer thickness of approximately 6.4 ft. The relatively large size of the armor stone would require a bedding layer of stone to prevent excessive damage to the filter fabric. Following the guidelines of USACE (1998), the required bedding layer thickness would be approximately 3 ft, making the total revetment thickness 9.4 ft.

For lower unit costs based on a full 9.2 mile project, the cost of this approach, not including the soil pre-loading, would be in excess of $2,000/ft, greatly exceeding the available construction budget. In addition to the high cost, successful implementation of the revetment approach could be significantly hindered by additional bearing capacity failures during placement of armor stone. Fugro has suggested that initial failure of the soft clays and associated soil remolding could decrease bearing capacity by up to 50%.

Due to the high cost of the revetment, other alternatives were considered, such as application of a thinner bedding layer using a Triton marine mattress, constructing a sheetpile wall through the imported stiff clay, constructing a reef breakwater onshore, or constructing a clay berm as a backstop for the G/CS beach fill. Unfortunately, none of these approaches appeared feasible or significantly beneficial. Therefore, considering the large degree of uncertainty involved in stacking the stiff clay and the high cost of subsequent armoring, *the soil pre-loading alternative is not recommended for inclusion in the testing program*.

Layout of Testing Program

In designing the layout of the testing program, the following two primary factors were considered:

(1) To the extent practicable, each test section needs to be long enough to infer valid conclusions regarding performance of a full 9.2 mile project. Performance will be evaluated in terms of constructability, settlement, structural stability, wave attenuation, shoreline and beach profile response, aesthetics, and other factors.

(2) To the extent practicable, the test sections should have enough separation such that they do not influence each other and can be evaluated discretely.

From a realistic standpoint, perfect representation of the full 9.2 mile project can not be achieved regardless of the test layout due to the significantly smaller lengths of the test installations and shorter evaluation period. As is evident in Figure 8, shoreline change at the Refuge is variable alongshore due to subtle differences in soil conditions, beach slope, wave exposure, proximity to inlets, and other factors. The degree that this variability influences evaluation of project performance is expected to increase as the observation period decreases.

At a minimum, it is recommended that the test installations be monitored for one year to allow exposure to a full range of seasonal conditions. However, even over one year, the natural variability in shoreline change could influence evaluation of how the shoreline responds to the test installations. In addition to the difficulties associated with shoreline change, evaluating settlement could be challenging since total soil consolidation is expected to occur over a period of decades, with only on the order of 10% occurring over the first 6 to 12 months.

The location of the testing program was selected to be at the eastern end of the 9.2 mile project area a minimum of 2,000 ft from Joseph Harbor. This location was selected to offer Joseph Harbor as a possible offloading point and shelter from waves for construction contractors. A minimum offset of 2,000 ft was selected to minimize the potential influence of the inlet on the test installations.

The proposed layout for the testing program is provided in Figure 20. Specific design issues that served as the basis for the layout are provided below.

Breakwaters

The influence of breakwater crest elevation, width, and permeability on wave transmission has already been discussed. For development of the test layout, additional parameters including breakwater distance from the shore, gap dimensions and ratio (width of gaps versus width of breakwater segments), and the orientation to the shoreline were also considered. As discussed in the Feasibility Study report, two general layouts were considered for the full 9.2 mile project – shore parallel breakwaters and angled breakwaters. Angled breakwaters could be oriented to provide better protection against the predominant southeast waves, but would provide less protection against storm waves that could propagate from the southwest. To maximize protection during storms, it is recommended that the breakwaters be aligned parallel to shore.

The distance between the breakwater and the shoreline influences the size of locally-generated wind waves landward of the breakwater. If the breakwater is placed too far from the shoreline, winds may generate erosive waves between the breakwater and the shoreline. Therefore, it is desirable to place the breakwater relatively close to the shoreline. In related project experience at a marsh restoration project in West Bay, Texas, Shiner Moseley observed that under typical wind conditions, a fetch of about 200 ft effectively limited erosive waves on un-vegetated, under-consolidated soils. Any recession of the shoreline after breakwater placement increases the distance to the shoreline.

For construction from barges, some minimum water depth must be maintained to avoid the requirement (and associated cost) for construction of an access channel. Generally, 4 to 5 ft of water depth is the minimum required for barge-based operations. Therefore, a water depth of approximately 5 ft, corresponding to a bottom elevation of -4 ft, was targeted to set the offshore distance of the breakwaters. The -4 ft contour is located, on average, approximately 150 ft from the shoreline. However, because at least some additional water depth will likely be desirable to most contractors and reduce overall construction costs, temporary flotation channels will be permitted. The channels would be required to be backfilled at the completion of construction.

Potential erosion seaward of the breakwater was also considered in setting location. As discussed in Chapter 3, continued erosion seaward of any protection structure at the Refuge is expected since lowering of the profile will continue over time regardless of the presence of a structure. (This does not include any local scour induced by the structure.) Because the beach profile is steep near the shoreline but gently sloping offshore, lowering of the profile occurs more rapidly closer to shore, as depicted in Figure 7.

For the full 9.2 mile project, breakwater gaps will be desirable for circulation of water and marine life around the breakwater segments. Gaps will also improve seaward flow of groundwater runoff and return flow of tidal surge. Smaller or too few gaps could increase potential scour of the seabed at the gaps. It is expected that rock will be placed across the bottom of the gaps. Design of the gaps is considered a fine-tuning item that may be evaluated independently of the testing program. Therefore, gaps will not be included in the test layout, with the exception of the 5 ft gaps in the concrete panel breakwater.

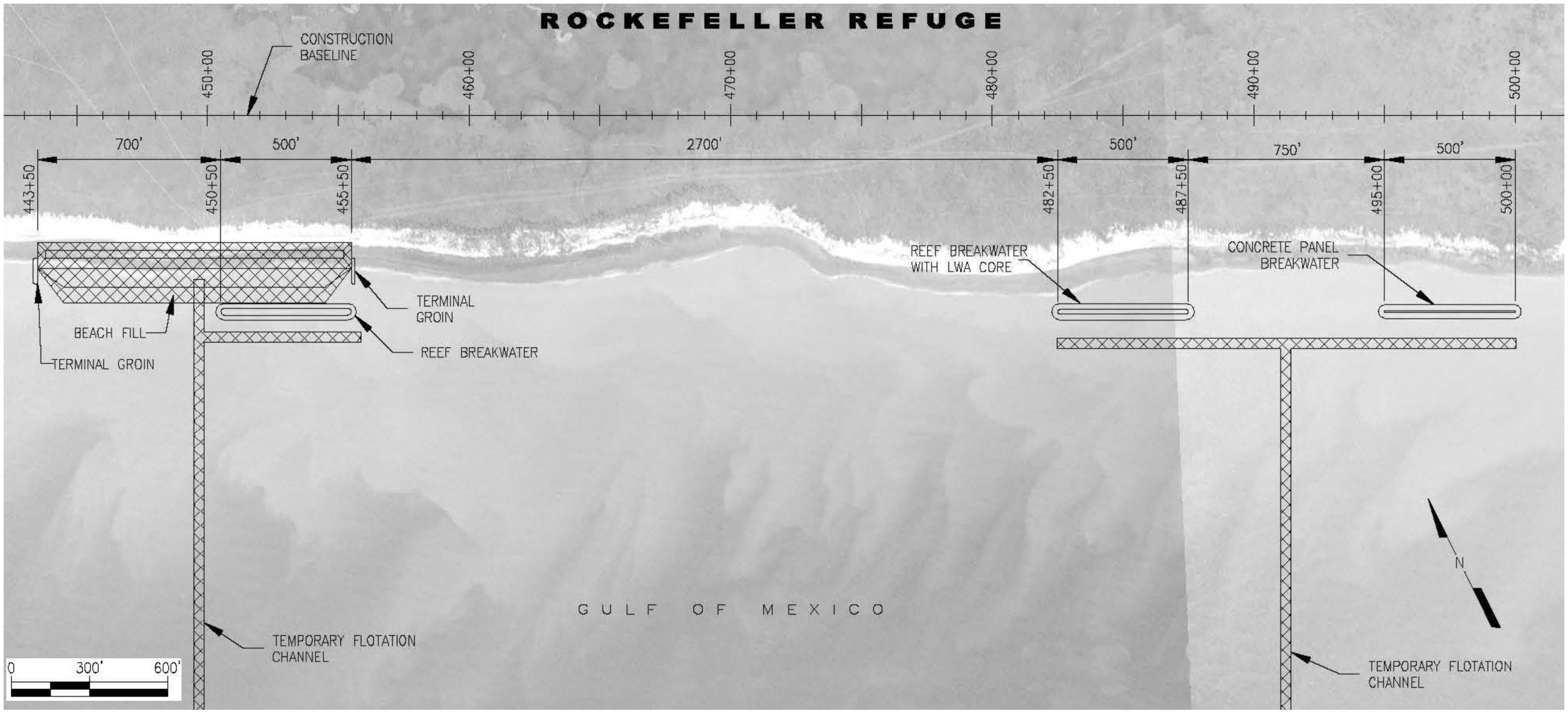

Figure 20: Layout of Testing Program

To estimate the breakwater length needed to infer valid conclusions for a full 9.2 mile project, the influence of wave diffraction at the breakwater tips was evaluated through application of REF/DIF 1, which is a phase-resolving parabolic refraction-diffraction model for wave propagation (Kirby and Dalrymple 1994). In addition, the method developed by Kaihatu and Chen (1988), which provides a simple analytical solution that includes both reflection and diffraction but not varying water depth, was applied. The calculations were performed for 1-year storm waves propagating from angles of up to 30 degrees from shore normal. Results of the REF/DIF modeling are plotted in Figures 21 and 22. For both methods of analysis, the results suggested that the diffracted waves could influence shoreline response within approximately 150 ft of each end of the breakwaters. Therefore, each breakwater test section was designed with a length of 500 ft, providing approximately 200 ft of protected shoreline that is expected to be beyond significant influence of diffracted waves.

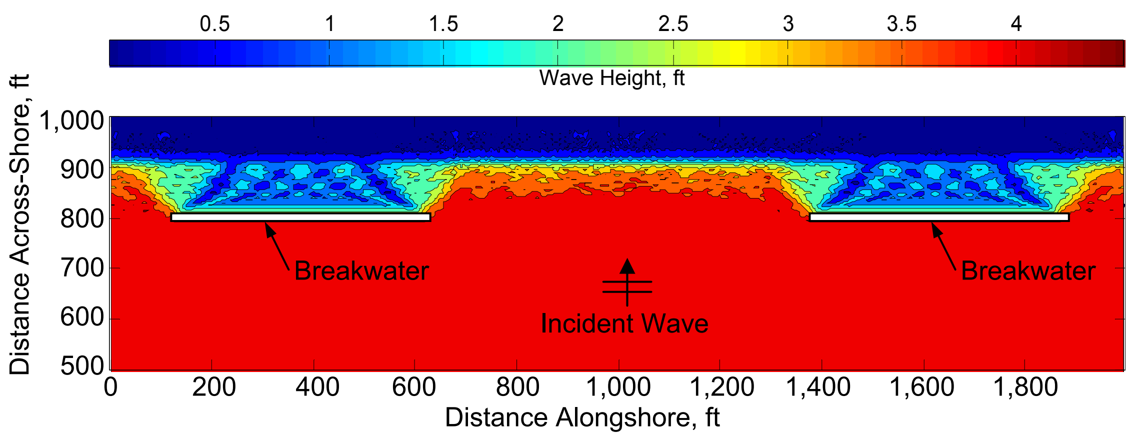

Figure 21: Wave Refraction and Diffraction (1-Year Storm, 0-deg Incident Wave)

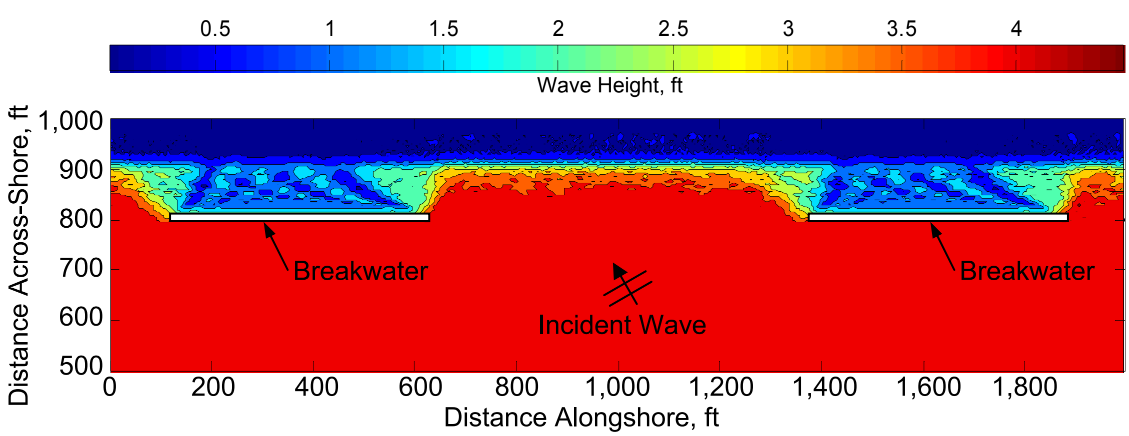

Figure 22: Wave Refraction and Diffraction (1-Year Storm, 30-deg Incident Wave)

Wave diffraction was also considered for spacing of the breakwater alternatives. General guidance in USACE (1984) indicates that a breakwater spacing that exceeds five times the wavelength will allow the breakwaters to function independently of each other. Following this guidance, for a typical wavelength of 150 ft, the breakwater separation required for discrete evaluation of the test alternatives was selected as 750 ft. This spacing appears conservative compared to the diffraction calculations presented in the previous paragraph and shown in Figures 21 and 22, which suggest that significant influence of each breakwater would not extend beyond approximately 150 ft to the east or west of each breakwater.

Beach Fill

As previously explained in this chapter, representation of large-scale beach fill through construction of a relatively short test section is complicated by the strong influence of end effects and associated lateral spreading of the fill. As shown in Figure 11, more than 1.2 miles of beach fill could be required for a representative one year simulation of the full 9.2 mile project. To make the test length feasible, terminal groins will be constructed at each end of the fill to reduce spreading. The fill would still migrate alongshore, but would remain primarily between the groins.

The dominant processes that need to be evaluated during the beach fill tests are cross-shore and longshore transport and settlement, all of which dictate shoreline change. Of these components, longshore transport is expected to be most influenced by the groins. Cross-shore transport and settlement can be evaluated relatively easily through surveying, regardless of overall shoreline change or influence by the groins. However, longshore transport will be significantly influenced by the groins since, without the groins, most of the fill would quickly spread beyond the initial placement area. Therefore, focus was placed on understanding the relative importance of longshore transport so that valid inferences on performance of the full 9.2 mile project could be made.

In assessing the relative importance of longshore transport, it was recognized that longshore transport causes shoreline change only where there are gradients in the rate of net transport. For the full 9.2 mile project, longshore transport is not expected to be the dominant factor in influencing shoreline change considering the following factors:

- The project area has relatively straight and parallel depth contours, which reduces the potential for gradients in wave energy and net longshore transport;
- Except at each end of the project area (at Joseph Harbor and Beach Prong), the area is void of any sources or sinks of sediment, which also reduces the potential for gradients in net longshore transport; and
- Given the relatively mild wave climate and large particle size of the gravel and crushed stone fill being considered, the net longshore transport rate is expected to be very low (on the order of 3,000 yd^3/yr). Adding reef breakwaters would result in an even lower transport rate.

Figure 23 shows a generalized solution for the one-year evolution of a 4,400-ft long beach fill with 100 ft terminal groins at each end. The solution is based on the P-C equation, which considers longshore transport but not cross-shore transport or wave diffraction. In the simulation, the net longshore transport rate was 3,000 yd^3/yr and the groins were assumed to be impermeable. The results suggest that only a 200 ft section of shoreline at the center of the fill would be unaffected by the groins. The construction of a 4,400 ft test section would likely cost more than $10,000,000, which is considered to be excessive for the test installation.

To provide a shorter test project having a more reasonable cost, the length was reduced to 1,200 ft. As depicted in Figure 24, the entire length of a 1,200 ft project would be influenced by the groins, with the updrift and downdrift ends receding and advancing, respectively, by approximately 15 ft.

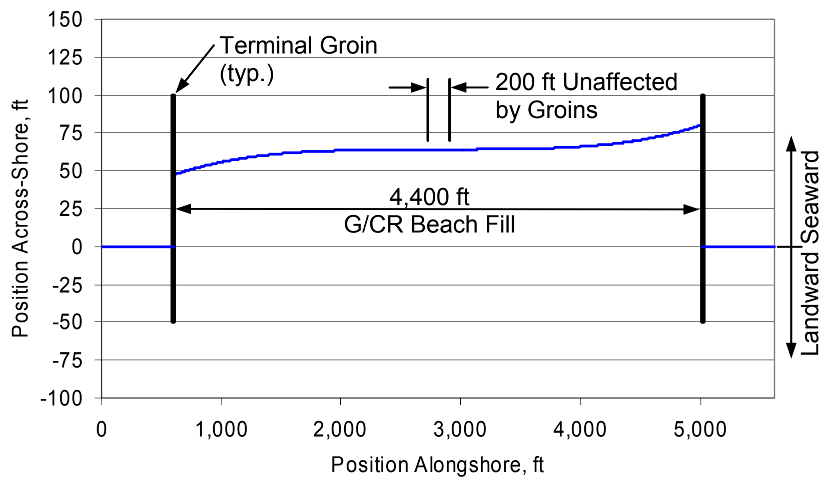

Figure 23: Generalized Solution for Evolution of Beach Fill with Terminal Groins

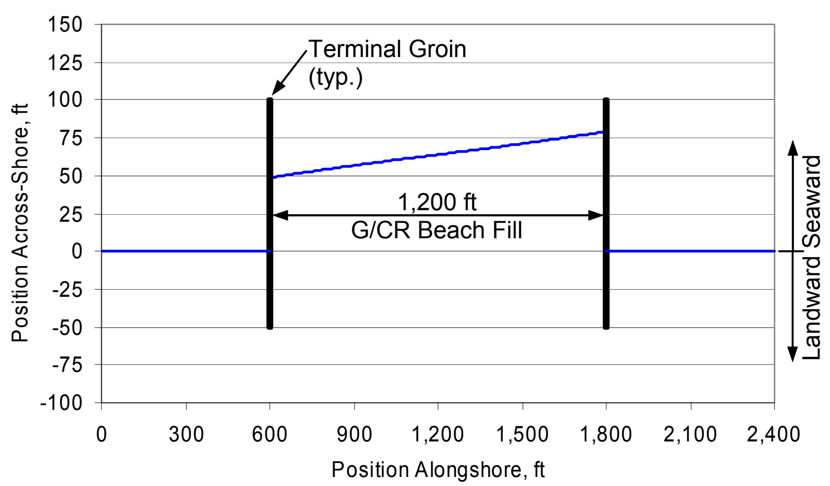

Figure 24: Generalized Solution for Evolution of Beach Fill with Terminal Groins

As mentioned, the P-C solution does not include wave diffraction, which may also influence shoreline response. By again applying the methodology of Kaihatu and Chen (1988) for estimating diffraction, it appears that, for one year storm waves propagating from angles of up to 30 degrees from shore normal, the shadow effect of the groin (considering diffraction only) should be limited to an area within 150 ft of the groin. This shadow area is much shorter than the approximate 1,200 ft area influenced by longshore transport as depicted in Figure 24. Net longshore transport would clearly be the dominant forcing mechanism for interaction between the groins and fill material.

Because of the obvious benefits of constructing a longer beach fill, it is recommended that the two beach fill alternatives be joined to create a continuous 1,200 ft fill with a terminal groin at each end. The reef breakwater would be located within the eastern 500 ft of the fill area, with the remaining 700 ft being unprotected fill. Evaluation of the fill alternatives in terms of shoreline response would need to consider the anticipated recession and advance at the respective east and west ends of the fill. Given that impacts of wave diffraction from the reef breakwater are expected to be limited to an area within 150 ft of its west end, the center 200 ft of the fill area can be applied as a buffer that separates the two fill alternatives.

The groins could be constructed of rock similar to that being placed for the reef breakwaters or of gabions filled with the beach fill material. However, the crest of the groins could not be much higher than the beach fill due to the limited bearing capacity of the underlying soft clay. As a result, waves may transport some of the fill over the groins. In addition, the groins would not extend far enough offshore to completely prevent transport of fill around their ends. To reduce the risk of transport of escaped fill material into adjacent test areas, the fill alternatives were located to the west (net downdrift) of the other two alternatives.

6. CONCLUSIONS AND RECOMMENDATIONS

The final design for the testing program of the Rockefeller Refuge Gulf Shoreline Stabilization project carried forward work performed in prior data collection and feasibility study efforts and involved final screening and evaluation of the following five alternatives:

(1) Beach fill with gravel/crushed stone

(2) Reef breakwater with sand or gravel/crushed stone beach fill

(3) Reef breakwater with lightweight aggregate core

(4) Concrete panel breakwater

(5) Soil pre-loading

These alternatives were evaluated in terms of anticipated performance and cost. Although each of the alternatives still contain some level of uncertainty regarding performance due to the very soft soil at the site and lack of detailed design methodology for the unique approaches being considered, analysis performed indicates that the testing program is feasible. Of the five alternatives, all *except soil pre-loading* are recommended for inclusion in the testing program.

The beach fill with gravel/crushed stone (G/CS) alternative would be constructed with material having a median particle size of approximately one inch. The large size of the particles compared to sand should make it relatively stable under typical and one-year storm conditions, but landward transport of the G/CS into the marsh during more severe storms remains a concern. A feature termed a "backstop" that is similar to a wide dune has been included to help address transport into the marsh; however, the backstop could undergo significant lowering due to settlement. Total settlement may be on the order of two feet, but it is likely that the settlement will occur slowly over a period of decades.

Terminal groins will be constructed at each end of the beach fill to reduce spreading alongshore and allow a shorter test length having a feasible cost. The groins may cause longshore transport to have a larger influence on shoreline change than would be expected for a full 9.2 mile project. Despite possible over-representation of longshore transport, the beach fill test is recommended to allow needed evaluation of cross-shore transport and settlement, which are the mechanisms that are expected to dominate shoreline change for the full project. The berm width for a full 9.2 mile project could likely be less than the 65 ft (in its equilibrium condition) berm proposed for the test section since spreading alongshore will be much less of a concern for a longer project. The opinion of probable construction cost (cost) for a test installation of the beach fill with G/CS alternative is $1,656,000.

For the reef breakwater with a sand or G/CS beach fill alternative, G/CS is recommended due to its greater stability and because sand would create a relatively flat profile that could bury the breakwater. This alternative will therefore employ a fill cross section identical to the stand-alone beach fill alternative. As with stand-alone beach fill, the berm width for a full 9.2 mile project could likely be less than that proposed for the test section. The reef breakwater will help stabilize the fill and may provide significant improvements by attenuating waves during larger storms. Anticipated pre- and post-settlement crest elevations of the reef breakwater are +1.0 ft and approximately -0.2 ft, respectively. The cost for a test installation of the reef breakwater with G/CS beach fill alternative is $2,612,000.

The reef breakwater with lightweight aggregate (LWA) core is anticipated to provide more wave attenuation than the reef breakwater without LWA core. However, this alternative may be less stable due to placement of armor stone on the encapsulated LWA and greater wave loads associated with the higher crest. In addition, without the redundancy of a beach fill landward of the breakwater, erosion by transmitted waves could be significant during larger storms. The integrity of the geotextile fabric encapsulating the lightweight aggregate should be carefully evaluated during construction and monitoring. Anticipated pre- and post-settlement crest elevations of the reef breakwater are +3.25 ft and approximately +1.9 ft, respectively. The cost for a test installation of the reef breakwater with LWA core alternative is $1,084,000.

The concrete panel breakwater is the most unique approach being considered and the most challenging to analyze. Re-evaluation of geotechnical conditions allowed refinement of placement geometry for the imported backfill material and pile penetration depths. The crest elevation of the panels is recommended to be +5.0 ft with minimal lowering expected to occur over the life of the project. However, more than one foot of total settlement is expected for the rock scour apron and imported backfill material, which provide lateral support for the wall. A lateral load test is recommended for this alternative. Wave attenuation and cost (for the full 9.2 mile project) is expected to be comparable to the reef breakwater alternative. Therefore, it is possible that G/CS beach fill could also be applied in combination with the concrete panel breakwater to provide redundancy in shoreline protection during larger storms. The cost for a test installation of the concrete panel breakwater alternative is $1,452,000.

Of the five alternatives, soil pre-loading underwent the least analysis during the initial feasibility study. As documented in the 30% design report, more detailed analysis eventually led to concerns over provision of a suitable berm after failure of the underlying soft clay. Although testing could be carried forward to better evaluate the extent of soil failure, refinements to the design wave and required stone size for subsequent armoring of the berm suggested that the total cost of this approach would significantly exceed the available construction budget for a full 9.2 mile project. *Therefore, soil pre-loading was removed from the testing program*.

A comparison of the four alternatives recommended for inclusion in the test program is provided in Table 11. Construction of prototype test installations will allow more detailed evaluation and comparison of each alternative in terms of constructability, ability to deal with the soft soils, wave attenuation, shoreline response, cost, maintenance requirements, and aesthetics. Evaluation of the test installations will serve as the basis for implementation of the full 9.2 mile project. However, the lesser lengths and shorter exposure period of the test installations versus the full 9.2 mile project will impose certain limitations on the conclusions that can be inferred.

Table 11: Summary of Wave Transmission and Cost

Alternative	Transmitted Wave Height (H_s), ft for Given Return Interval			Cost
	Day-to-Day	1 Year	Category 1 Storm (Approx. 9-12 Years)	
G/CS Beach Fill	N/A	N/A	N/A	$1,656,000
Reef Breakwater with G/CS Beach Fill	0.7	1.7	3.1	$2,612,000
Reef Breakwater with LWA Core	0.2	1.1	2.6	$1,084,000
Concrete Panel Breakwater	0.6	1.7	3.3	$1,452,000

To the extent practicable, the layout of the testing program has been optimized to infer valid conclusions. The most significant limitation is related to evaluating shoreline change of the beach fill tests. As already mentioned, the relatively short lengths of the beach fill test sections and their containment by terminal groins may cause migration of the beach fill material to be dominated by longshore transport. However, the primary shoreline-change mechanisms for the full 9.2 mile project are expected to be cross-shore transport and settlement, not longshore transport. This limitation is considered acceptable, and it is recommended that the fill alternatives be included in the testing program.

The total cost of the testing program, including mobilization and demobilization, is $7,300,000. This cost includes an approximate $1,500,000 contingency to place additional material in the event that instantaneous (construction-phase) settlements are up to 18 inches greater than anticipated. However, the cost does not include project monitoring and other professional services during construction, and is based on higher unit prices expected for the smaller quantities required for the test sections. In consideration of the ongoing rapid erosion at the site, the hydrographic and bathymetric data and aerial photography should be updated prior to solicitation of construction bids. Monitoring of the test installations is recommended for a minimum of one year and should include surveying, aerial photography, and wave and tide measurements.

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APPENDIX A

MONITORING PLAN

Background

Through the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA), the Louisiana Department of Natural Resources (LDNR) Coastal Restoration Division has been aggressively combating the 25 to 35 square miles of coastal erosion that occurs in Louisiana each year. Subsidence, erosion, and the effects of man have been identified as the causes of converting valuable wetlands and beaches to open water. One such area is Rockefeller Wildlife Refuge in eastern Cameron Parish.

To combat the direct loss of wetlands in the Rockefeller Refuge, LDNR teamed with the National Marine Fisheries Service (NMFS) to implement the Rockefeller Refuge Gulf Shoreline Stabilization Project (ME-18, CWPPRA Priority Project List 10). The project intent is to halt erosion along the 9.2 mile portion of the Refuge west of Joseph Harbor Bayou ("Joseph Harbor"). The project is funded and authorized in accordance with the Coastal Wetlands Planning Protection and Restoration Act (16 U.S.C.A., Section 3951-3956). The present work was conducted under LDNR Contract No. 2511-05-08.

The Final Design Report identifies the unique nature of the Refuge and the challenges associated with protecting the shoreline. The analyses determined that:

- Incident storm waves are depth limited for a considerable distance offshore;
- The beach profile is relatively uniform alongshore and characterized by:
    - Shell hash above the high water line
    - A very gentle offshore slope
    - Limited relief landward with a wide expanse of marsh
    - A submerged scarp of 3 to 5 ft along the shoreface;
- Surface soils are characterized by an approximate 40 ft stratum of highly under-consolidated, very soft clay;
- The beach contains little to no sand

Because of the very soft soils and no readily-available sand sources, conventional shoreline protection alternatives are not feasible at the Refuge. Therefore, test sections for less conventional alternatives are to be constructed and evaluated. The alternatives included are:

- Beach fill with gravel/crushed stone
- Reef breakwater with sand or gravel/crushed stone beach fill
- Reef breakwater with lightweight aggregate core
- Concrete panel breakwater

Figures A1 and A2 show the proposed layout and locations for the test sections. As described in the report, considerable effort was made by the design team to place the alternatives in locations that would have little to no influence on adjoining alternatives.

Objective

Because of the unique site characteristics, conventional designs are not feasible. For the unique alternatives proposed for the test program, limited or lacking available design guidance posed a challenge in predicting wave dissipation and transmission and soil/structure interaction. In addition, coastal processes along mud coasts are not well understood, and the innovative designs may have some redundancy or limitations based on the assumptions made. To alleviate some of the unknowns and help understand the performance characteristics of the constructed alternatives, a monitoring plan was developed. Although the data collected could be also be applied for other research purposes (such as wave dissipation through suspended mud), the monitoring plan has not been specifically structured to do so. The key objectives of the monitoring plan are:

- Collect data that will allow for design optimization of each section;
- Determine performance characteristics of each alternative tested;
- Determine which alternative is most suitable for a full 9.2 mile project.

The monitoring plan will consist of land-based and aerial photography, wave and tide gauging, bathymetric and topographic surveying, and measurement of settlement with settlement plates. The monitoring will be initiated during construction of the test sections and last for a minimum of one year. Along with the test sections, a 2,000-ft long control area has been identified that is outside of the influence of the test sections, as shown in Figure A2. Any other alternatives added to the test program will also be monitored.

It should be noted that the monitoring program, as designed, does not provide special provisions for evaluation under tropical storm conditions. If a storm does occur during the monitoring period, additional evaluation and supplemental surveying of the site should be conducted to determine the impacts of the storm. At a minimum, post-storm evaluation should include a site visit for qualitative assessment of shoreline change and structural integrity of the test sections. Based on the site evaluation, additional monitoring such as surveys or aerial photographs may be warranted.

Monitoring Components

Surveying
Beach profiles will be surveyed to determine the effectiveness of each alternative in reducing shoreline erosion and their influence on the adjacent shoreline. Structures will be surveyed to determine settlement, scour and structure stability. The surveys will provide a direct comparison of alternatives and the control section to determine the effectiveness of each alternative and its relative effectiveness to the other alternatives.

Locations for survey transects are shown in Figure A2. Transects shown are spaced 50 to 100 ft along each test section and should be surveyed pre-construction, post-construction, and quarterly for one year for a total of five surveys. Additional transects should be surveyed at 100 ft, 300 ft, and

500 ft beyond each test alternative to evaluate updrift and downdrift impacts. The pre-construction and post-construction survey will be conducted by the construction contractor. The three remaining surveys should be conducted under a separate contract. Transect spacing within the control area will be 200 ft. At a minimum, transects should be 600 ft long, extending 400 ft offshore. The initial survey of the control site will coincide with the completion of the test sections. All topographic and bathymetric surveying must be conducted by a surveyor registered in the State of Louisiana and be produced in accordance with the survey requirements spelled out by the Louisiana Department of Natural Resources.

Aerial Photography

Aerial photography will be collected along the western boundary of Rockefeller Refuge from just east of St. Joseph's Harbor to the west. The photographs will be georeferenced using visible targets placed on the ground at pre-determined control points and will be collected concurrent with topographic and hydrographic surveys for a total of five sets of photographs over one year. Aerial photography will provide a view of the effectiveness of the different alternatives and a comparison of the erosion rates of the Refuge beyond the control section. Although less accurate than surveys, the aerial photography will provide a larger-scale evaluation of the test sections.

Ground Photography

Ground-level photography will provide cost effective small-scale evaluations that may be missed in the surveys or aerial photos. Shifting armor stone, small scale shoreline changes, and local slumps due to scour or soil failure are some of the examples of additional information that may be obtained from ground-based photography.

The ground-level photography will be collected during each survey. The photography will help document shoreline change, integrity of the breakwaters, wave attenuation, and other aspects of the project. At a minimum, each survey transect, the ends of each alternative, the adjacent shoreline, and the control site will be photographed. To the extent practicable, the photographs will be taken from set locations.

Wave Gauging

Both wave and tide data will be applied to evaluate design estimates of wave transmission at each breakwater. This information will be used to calibrate the predicting equations and optimizing the design for the full 9.2 mile project. In combination with the beach profile survey data, the wave data can be applied to determine and compare erosion thresholds at test and control areas.

Five wave gauges will be installed to measure wave attenuation at the breakwaters. One wave gauge will be installed offshore of the structures to measure incident waves. A gauge will also be located leeward of each of the three breakwaters. A fifth gauge will be located in the control area along the same depth contour as the three in the lee of the structures to measure unaffected ambient waves. Wave data should be collected for six months, preferably from September to March. These months may be adjusted based on construction timeline. During this period there are typically numerous frontal passages which are preceded by strong southeast winds that increase water level and wave height.

Proposed locations for the wave gauges are shown in Figure A2. Wave gauges should be highly accurate pressure transducers (or equivalent instruments) that are capable of capturing wave height and period as well as providing water depth. Sampling bursts should be frequent (minimum every 30 minutes) and long enough to capture several wave passages. Tide readings must be referenced to NAVD'88 and be accurate to within 0.1 ft.

Tide Gauge
As described in previous analyses, wave heights are depth-limited for a significant distance offshore of the Refuge, and the prediction of wave transmission is depth-sensitive. Tide data are critical to understand the measured transmission characteristics for each of the alternatives. Combined with wave data, the tide data will provide refinement of the transmission coefficients for the structures and their performance.

A tide gauge will be installed and operated concurrent with the offshore wave gauge to measure water surface elevations. The tide data could then be correlated to data from other stations along the Louisiana coast (such as at Calcasieu Pass) for which long-term records exist so that long-term water level trends at the Refuge can be better inferred. For the Rockefeller Refuge, the wave gauges will likely be pressure sensors such that they will be able to obtain the tide measurements. The tide data should be applied with the wave data for calibration of wave transmission equations and optimization of the full 9.2 mile project.

Settlement Plates
The extremely soft soils offshore of the Refuge will consolidate over time. Predictions were made on the settlement rate for each alternative. The measured settlements will be used to refine the design template and determine if design modifications are necessary.

Settlement plates will be installed to measure the magnitude and rate of settlement of each structure. The settlement plates will be installed during construction and surveyed by the contractor. Settlement of the plates will be measured during each monitoring survey over the next year. Locations for the settlement plates are shown in Figures A3 and A4. Approximately 16 settlement plates will be installed by the construction contractor (details are provided in the plans and specifications).

Additional Alternatives
Additional shoreline protection alternatives may be constructed outside of the test area by other private or public entities. They will be constructed under a separate permit and be outside the immediate vicinity of the test sections described herein, but within easy travel distance (1 mile or less) of the project site. Construction of any additional alternatives should closely coincide with the construction of the described test sections. At the discretion of the Louisiana Department of Natural Resources, monitoring of these alternatives may be included.

If the alternative is shore-based, i.e. revetments, beach nourishment, or similar shoreface projects, surveying will be required on similar spacing and length as described for the alternatives above. The transects should be have 50 ft spacing within the body of the project, with spacing increasing to 100, 300 and 500 ft beyond the end of the project. For alternatives constructed offshore, such as a detached breakwater, floating breakwater, or similar, a wave gauge will be required in the lee of each structure. Data analyses and project evaluations will be included into the analyses described below.

Analyses and Results

The monitoring plan is to provide detailed design evaluations and relative performance of each of the alternatives. At a minimum, the following analyses will be conducted for the monitoring plan:

Comparative Survey Analyses
As-built construction surveys will be used as comparison to post- construction surveys to determine shoreline changes, volume changes, and changes in the general cross section of the profile. This analysis will be the basis for the relative effectiveness of each alternative constructed. The surveys will be directly compared to the control site to determine the effectiveness of each alternative in reducing or abating erosion or accretion.

Wave Transmission
Wave and tide data will be applied to refine the transmission estimates for the detached breakwaters under storm conditions. This refinement will be used to extrapolate transmission coefficients outside the collected data ranges and refine the potential erosion scenarios for Rockefeller Refuge. The revised transmission coefficients will then be used to refine the alternative designs; if necessary, recommendations will be made for potential modifications that would help the project.

Settlement
Settlement of each plate will be measured and compared to the predicted settlement. Modifications to the prototype design templates will be accomplished based on the settlement plate data. Small changes in the settlement rate and/or soil bearing capacity will directly affect the construction template. This is true for all the alternatives tested.

Shoreline Change
In addition to the shoreline change directly within the project and control areas, the aerial photos will be applied to evaluate shoreline change along adjacent areas of the Refuge for the entire year. In addition, aerial photographs collected prior to construction will be used to evaluate longer-term shoreline change. Representative erosion rates will be calculated and the relative effectiveness of the alternatives will be evaluated based on the longer term shoreline data.

Storm-Induced Profile Change
If the test installations are impacted by a significant storm during the monitoring period and post-storm data are collected, the wave, tide, and beach profile data can be applied to calibrate existing empirical models for predicting cross-shore profile response under a full range of storm conditions. This analysis will help with evaluation and final design of the gravel/crushed stone beach fill if it is selected for the full project.

Preliminary Opinion of Probable Monitoring Cost
(Note: This estimate does not include related costs presented in the construction estimate.)

Item	Quantity	Unit Price	Extension
Instrumentation and Data Collection			
Surveys	3	$30,000	$90,000
Wave Gauging(1)	5	$28,000	$140,000
Aerial Photographs	3	$3,000	$9,000
Data Analyses and Reporting			
Survey Analyses	5	$4,120	$20,600
Settlement Plate Evaluations	4	$2,250	$9,000
Wave Transmission Analyses	5	$5,200	$26,000
Aerial Photo Geo-Referencing	3	$2,000	$6,000
Report Writing			$22,700
		Subtotal:	$323,300
		Contingencies (15%):	$48,500
		Estimated Total:	**$371,800**(2)

Notes:
(1) Cost of wave gauging includes data collection and preliminary data screening and refining.
(2) Estimated total cost includes monitoring for Gravel / Crushed Stone (G/CS) Beach Fill, Reef Breakwater with G/CS Beach Fill, Reef Breakwater with Lightweight Aggregate Core, and Concrete Panel Breakwater. Any alternatives added to the test program will be monitored at additional cost.

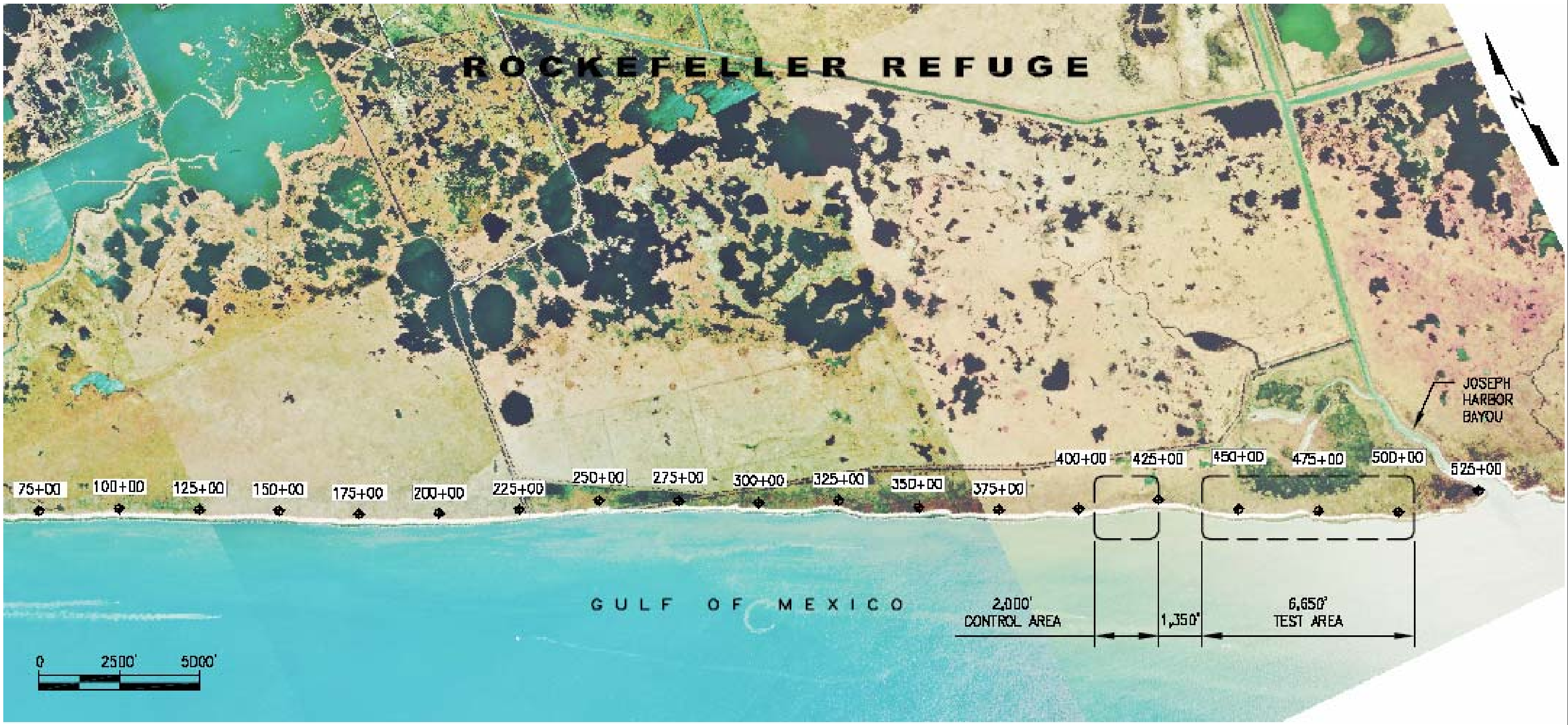
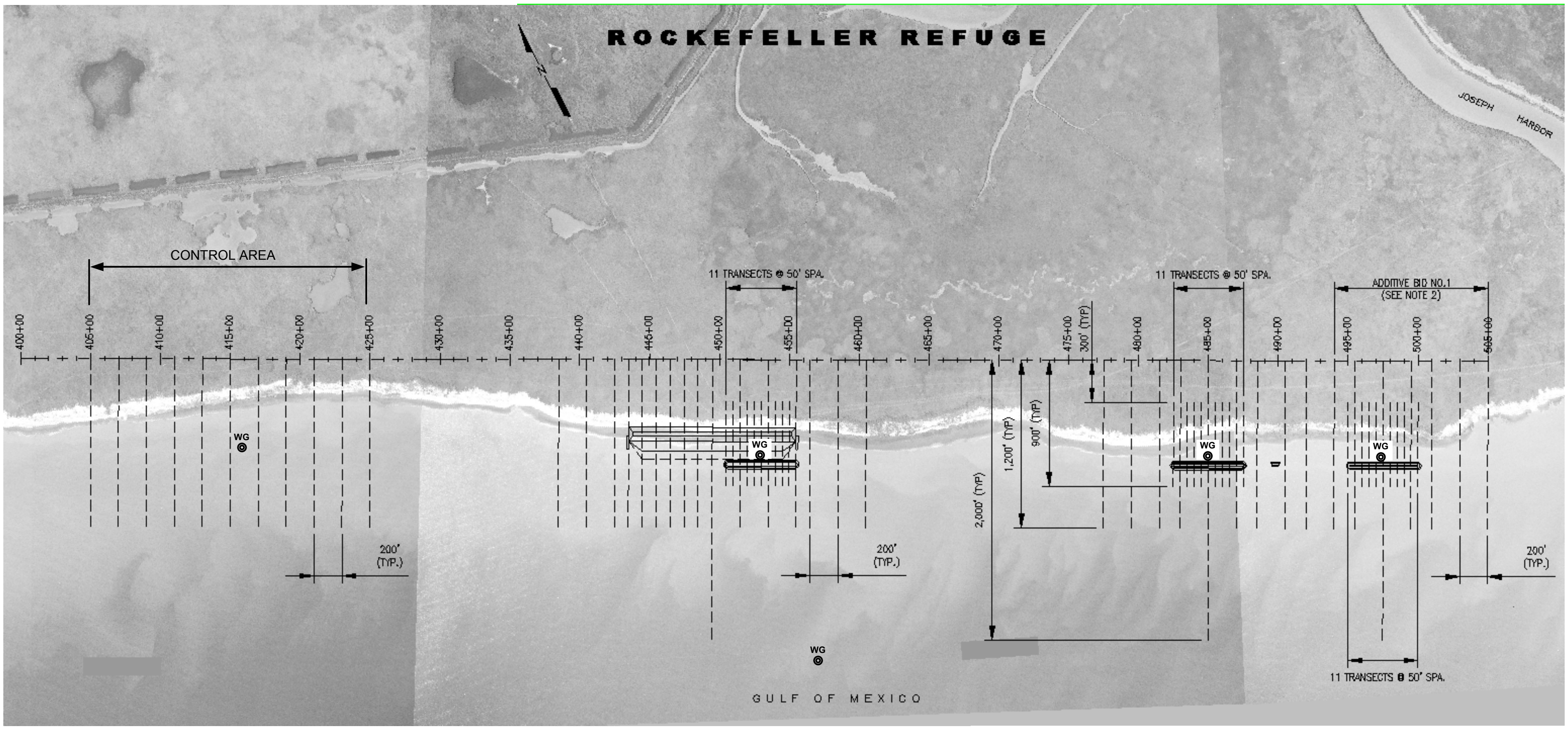


Figure A1: Location of Test and Control Areas



LEGEND

WG WAVE GAUGE

– – – – – SURVEY TRANSECT

Figure A2: Layout for Monitoring Plan

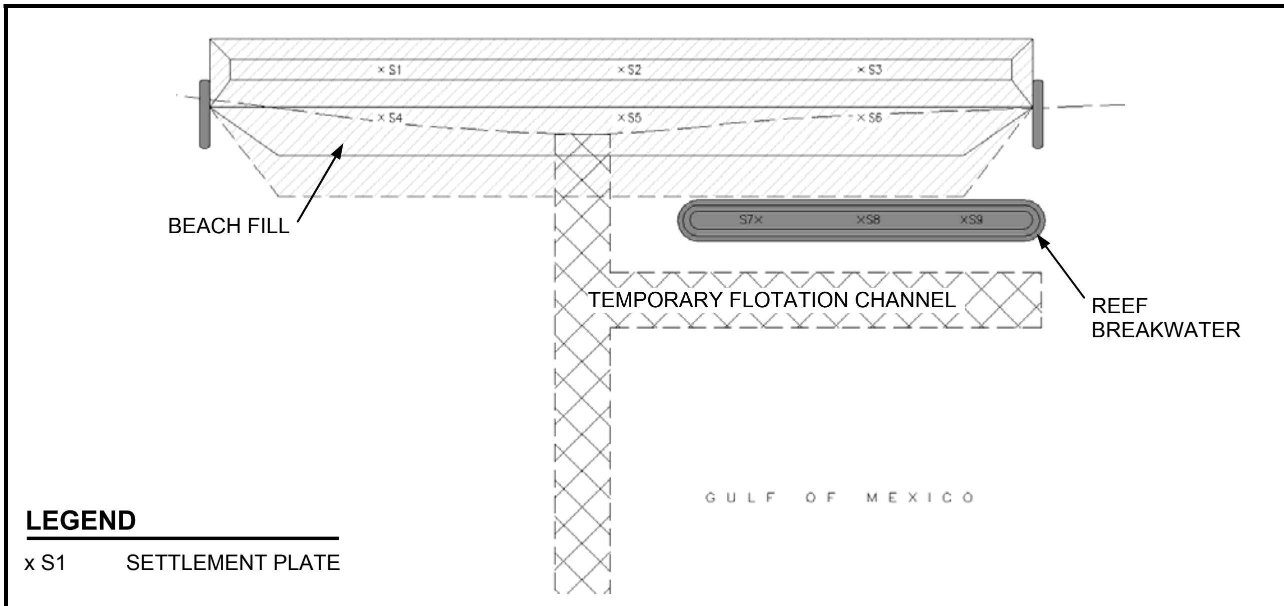


Figure A3: Settlement Plate Locations

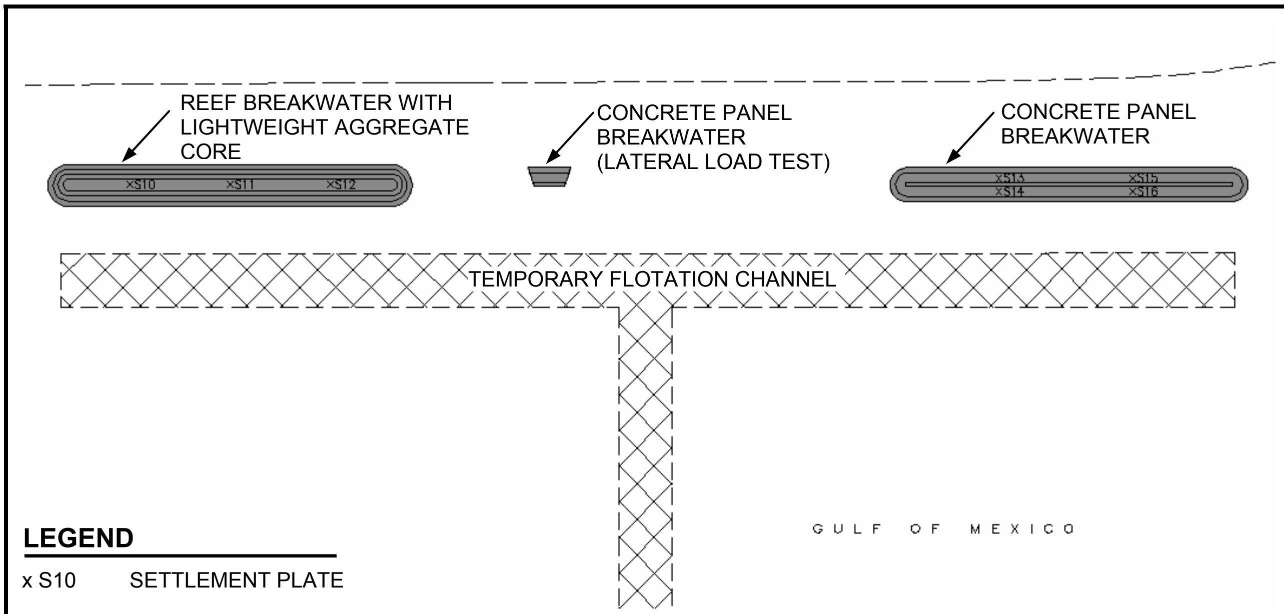


Figure A4: Settlement Plate Locations

APPENDIX B

OPINION OF PROBABLE CONSTRUCTION COST

ITEM	QUANTITY	UNIT	UNIT PRICE	EXTENSION
Mobilization / Demobilization	1	LS	$ 200,000	$ 200,000
Excavate / Backfill Flotation Channel	30,000	CY	$ 4	$ 120,000
Lights / Daybeacons (Warning Signs)	3	EA	$ 2,000	$ 6,000
Topo / Hydro Surveying	1	LS	$ 75,000	$ 75,000
Hazard Survey	1	LS	$ 30,000	$ 30,000
		CONTINGENCIES (15%):		$ 65,000
			SUBTOTAL:	**$ 496,000**
BEACH FILL				
Gravel / Crushed Stone (600 LF)[1,2]	21,900	TON	$ 45	$ 986,000
Terminal Groin	150	LF	$ 600	$ 90,000
Settlement Plates	2	EA	$ 2,000	$ 4,000
Supplemental Gravel / Crushed Stone (Addt'l 18" Settlement)[3]	8,000	TON	$ 45	$ 360,000
		CONTINGENCIES (15%):		$ 216,000
			SUBTOTAL:	**$ 1,656,000**
REEF BREAKWATER WITH BEACH FILL				
Armor Stone (500 LF)[2]	7,750	TON	$ 70	$ 543,000
Bedding Layer	1,900	TON	$ 35	$ 67,000
Grid Geocomposite	5,000	SY	$ 7	$ 35,000
Settlement Plates	4	EA	$ 2,000	$ 8,000
Gravel / Crushed Stone (600 LF)[1,2]	21,900	TON	$ 45	$ 986,000
Terminal Groin	150	LF	$ 600	$ 90,000
Supplemental Armor Stone (Addt'l 18" Settlement)[3]	2,600	TON	$ 70	$ 182,000
Supplemental Gravel / Crushed Stone (Addt'l 18" Settlement)[3]	8,000	TON	$ 45	$ 360,000
		CONTINGENCIES (15%):		$ 341,000
			SUBTOTAL:	**$ 2,612,000**
REEF BREAKWATER WITH LWA CORE				
Armor Stone (500 LF)[2]	7,100	TON	$ 70	$ 497,000
Bedding Layer	2,000	TON	$ 35	$ 70,000
LWA Core	1,700	CY	$ 90	$ 153,000
Grid Geocomposite	5,000	SY	$ 7	$ 35,000
Settlement Plates	3	EA	$ 2,000	$ 6,000
Supplemental Armor Stone (Addt'l 18" Settlement)[3]	2,600	TON	$ 70	$ 182,000
		CONTINGENCIES (15%):		$ 141,000
			SUBTOTAL:	**$ 1,084,000**
CONCRETE PANEL BREAKWATER				
Lateral Load Test[4]	1	LS	$ 99,000	$ 99,000
Excavation	3,250	CY	$ 4	$ 13,000
Prestressed Concrete Piles (16" Sq. x 70' Long)	1,540	LF	$ 45	$ 69,000
Steel Sheet Piles	250,129	LB	$ 0.60	$ 150,000
Precast Panels / Concrete Cap	250	CY	$ 400	$ 100,000
Erection / Assembly	7	DAY	$ 5,000	$ 35,000
Imported Backfill Material (Sand or Gravel)	2,500	CY	$ 40	$ 100,000
Grid Geocomposite	3,200	SY	$ 7	$ 22,000
Armor Stone	5,800	TON	$ 70	$ 406,000
Bedding Layer	1,100	TON	$ 35	$ 39,000
Settlement Plates	4	EA	$ 2,000	$ 8,000
Supplemental Armor Stone (Addt'l 18" Settlement)[3]	2,400	TON	$ 70	$ 168,000
Supplemental Imported Backfill Mat'l (Addt'l 18" Settlement)[3]	1,350	CY	$ 40	$ 54,000
		CONTINGENCIES (15%):		$ 189,000
			SUBTOTAL:	**$ 1,452,000**
			TOTAL:	**$ 7,300,000**

Notes:

(1) An extra 200 ft beach fill section is included as a buffer between the two beach fill alternatives. The cost of the buffer was split equally between the two alternatives.
(2) The quantities for Gravel / Crushed Stone and Armor Stone assume 6" of sinking into soft clay during placement.
(3) The quantities for Supplemental Armor Stone and Gravel / Crushed Stone are based on potential additional 18" of sinking into soft clay during placement.
(4) Cost of Lateral Load Test includes materials for construction and removal of 44.5 ft test section. Costs for instrumentation rental and technical oversight by engineering consultant are not included.
(5) Unit prices for the full nine mile project are expected to be lower than the prices applied for the relatively small test sections being considered here.
(6) The cost of monitoring is not included.