CHENIER RONQUILLE
BARRIER ISLAND RESTORATION PROJECT (BA-76)
30% DESIGN REPORT

Prepared for:

Louisiana Office of Coastal Protection and Restoration

Baton Rouge, Louisiana

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April 2011

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EXECUTIVE SUMMARY

Chenier Ronquille is located along the Plaquemines/Barataria Bay Barrier Shoreline, 8 miles east of Grand Isle, 33 miles west-northwest of the Mississippi River delta, and 47 south-southeast of New Orleans. Chenier Ronquille serves as the western anchor of the Plaquemines/Barataria shoreline and forms the eastern boundary of Quatre Bayou Pass. The island is bordered to the east by Pass La Mer, which separates the island from Chaland Headland.

The Louisiana Office of Coastal Protection and Restoration (OCPR), Restoration Division, requested an engineering and design report for the project area. This work is sponsored jointly by the National Marine Fisheries Service (NMFS) and OCPR, and funded through the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA).

Chenier Ronquille Island suffers some of the highest shoreline retreat rates in the nation. Recent shoreline change measurements suggest an average shoreline retreat rate of approximately 44 feet/year, though retreat rates of 108 feet/year have been measured during earlier time periods. The barrier island has been breached, which is negatively impacting the island's function.

The goal of the project is to increase island longevity by restoring the dune and marsh planforms. The project will repair the breaches in the shoreline and prevent creation of new breaches over the 20-year project life, while reestablishing and increasing the island's longevity via dune and marsh creation. In conjunction, the project will restore the shoreline, dune, and back-barrier marsh to increase island habitat utilization by essential fish and wildlife species both on the barrier headland and in the consequently developed quiescent bays. Project benefits are evaluated using the Wetland Value Assessment (WVA) Barrier Island Community Model.

Six alternatives are presented within this report to bracket a variety of solutions and costs. Three beach options and four marsh options were combined to create the six alternatives. All beach options have a 1V:30H slope above +1.0 feet, NAVD and 1V:90H below +1.0 feet, NAVD though the crest elevation, volume and footprint vary. All of the marsh options have a constructed elevation of +2.5 feet, NAVD with a variable footprint. There are two access channel alignments with the primary dike either to the north of the channel (channel backfilled during construction) or to the south of the channel (channel not backfilled during construction).

Alternative 1 is the recommended alternative because it meets the project goal and follows a standard coastal engineering design comprised of a design section, advanced fill, and a wide marsh to limit sediment losses due to overwash. Alternative 1 intends to maintain the pre-construction shoreline location at the end of the 20-year project life. The alternative consists of beach option 1 (which has a dune crest elevation of +8.0 feet, NAVD and a crest width of 270 feet) and marsh option 1 (which is the largest constructed marsh planform). The construction

template contains 437 subaerial acres (above 0.0 feet, NAVD) of which 336 acres will be created due to the project.

Table 1 provides a summary of the volumes, costs, constructed acres, and benefits of the various alternatives. Appendix A provides plan views and cross-sections of the proposed alternatives.

Table 1. Summary of Volumes, Benefits, and Costs of the Various Alternatives

Alternative	Fill Volume (cy) Beach	Fill Volume (cy) Marsh	Subaerial Construction Footprint	Cost Estimate	Net Benefits (AAHUs)	Cost per AAHU ($/AAHU)
1	1,830,000	1,380,000	437	$37,805,000	263	$143,700
2	1,830,000	940,000	381	$34,449,000	236	$146,000
3	1,830,000	590,000	311	$31,776,000	216	$147,100
4	1,840,000	940,000	394	$34,570,000	230	$150,300
5	1,310,000	1,380,000	411	$31,468,000	234	$134,500
6	1,310,000	1,020,000	350	$28,730,000	209	$137,500

Alternative 2 is similar to Alternative 1 in that it consists of beach option 1, but has a smaller marsh platform to reduce project costs (marsh option 2). Marsh option 2 is slightly smaller than marsh option 1, as the primary dike is constructed to the south of the access channel.

The goal of Alternative 3 is to provide the design beach while maintaining a construction cost of approximately $30M. It consists of beach option 1 and marsh option 4. Marsh option 4 is the smallest marsh option (planform and thus volume) in order to meet the intent of the alternative. The access channel is located similarly to the one in the Phase 0 report, which is south of the access channel proposed for marsh options 1 and 2, and must cross existing oil infrastructure. The primary dike is located south of the access channel for this marsh option to minimize costs.

Alternative 4 is similar to Alternative 2 in terms of total beach and marsh fill volume but has a wider (445 feet) and lower (+6 feet, NAVD) dune crest (beach option 2). The marsh platform is the same as Alternative 2 (marsh option 1). This allows a comparison of the effect of dune width and elevation on performance. It was determined that the wider, lower dune resulted in greater overwash and lower overall performance than the higher dune in Alternative 2.

Alternative 5 is comprised of beach option 3 and marsh option 1. Beach option 3 was developed to reduce the beach fill cost and has the smallest beach fill volume (the dune crest width is 150 feet at +8 feet, NAVD). It provides 20 years of advanced fill but no design section. This alternative should be compared to Alternative 1 in order to evaluate the impact of the narrower dune crest on project performance because the same marsh option is employed in both alternatives. Alternative 5 is the most cost effective option.

Alternative 6 is comprised of beach option 3 and marsh option 3. This alternative was developed to minimize construction costs while still trying to meet the project objectives. Marsh option 3 is similar to marsh option 4 with the access channel following the Phase 0 concept. The primary dike is constructed to the north of the access channel, which is then backfilled. Despite the cost to backfill the channel, Alternative 6 is the lowest cost alternative due to the smaller volume of sand required to construct beach option 3.

The beach and marsh fill borrow areas are located approximately 1.7 to 2.8 miles southwest of the project area. The project will use borrow areas initially developed for the East Grand Terre Island Restoration Project (BA-30) and Chaland Headland Restoration Project (BA-38-2). Approximately 3.9M cubic yards of beach fill and 6.5M cubic yards of marsh fill are still available for the construction of the Chenier Ronquille Project. Adequate volumes of beach and marsh fill material are available to construct any alternative. The beach fill borrow areas S-1 and S-2 are composed of 15% and 17% silts, respectively, with a mean grain size of 0.11mm. Borrow area D-1 contains beach fill material overlain by marsh fill material. The beach fill material is composed of 28% silt with a mean grain size of 0.11mm. No impacts to the shoreline are expected due to dredging any of the borrow areas.

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1 INTRODUCTION

This design submittal for the Chenier Ronquille Barrier Island Restoration (BA-76) CWPPRA Project was prepared for the Louisiana Office of Coastal Protection and Restoration (OCPR), the National Oceanic and Atmospheric Administration (NOAA) National Marine Fisheries Service (NMFS). The purpose of this study is to investigate alternative designs for the barrier island and to provide a recommendation for selection of an alternative. This preliminary design submittal includes engineering evaluation and analysis, plan and cross-section view drawings, beach and marsh construction volume computations, project benefits and cost estimates for each alternative. Impacts on the adjacent coastline were also evaluated.

Louisiana's barrier islands are suffering from high shoreline retreat rates and are in need of beach and marsh restoration in order to maintain their geomorphic function. Many of the barrier islands along the Plaquemines/Barataria Barrier Shoreline have either already been restored (Grand Isle, Chaland Headland, Bay Joe Wise, East Grand Terre) or plans are underway to restore the islands (Scofield Island, Shell Island, Pelican Island). Chenier Ronquille was the one island along the Barataria Basin edge that did not have an island restoration project in the planning phase.

Restoration of Chenier Ronquille will restore dune and marsh areas that provide habitat for migratory birds. The restored barrier island would reduce wave and tidal influence in the back-bay marsh areas and would enhance Louisiana's storm surge buffer. The stated goal of the Chenier Ronquille Shoreline Restoration Project is to reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage. Approximately 205 acres of marsh will be created, and an additional 105 acres of marsh will be nourished. The shoreline feature would extend up to 8,000 feet.

2 AUTHORIZATION

The Chenier Ronquille Shoreline Restoration Project was authorized as a candidate project under the 1990 Coastal Wetland Planning, Protection, and Restoration Act (CWPPRA). The project was selected from Project Priority List (PPL) 19 for design phase funding, leading to this report. This work was performed under DNR Contract No 2503-10-17. A Notice to Proceed was issued on June 15, 2010.

3 PROJECT GOALS AND OBJECTIVES

In co-operation with OCPR and NOAA-NMFS staff, the following goal has been identified for the Chenier Ronquille Barrier Island Restoration Project:

> *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

Objectives on which to evaluate and monitor the project were also identified:

1. Prevent island breaching over the 20-year project life.

2. Provide an intertidal marsh platform with tidal exchange by Target Year 4.

3. Maintain an island crest elevation of greater than +4 feet NAVD at Target Year 20.

4. Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.

5. Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.

6. Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline.

4 PROJECT LOCATION

Chenier Ronquille is located within the Plaquemines/Barataria Barrier Shoreline Complex, approximately 47 miles south-southeast of New Orleans and 33 miles west-northwest of the Mississippi River delta. It is approximately 8 miles east of Grand Isle and located between East Grand Terre and Chaland Headland (Figure 1). Chenier Ronquille is bordered by Pass Ronquille to the west, Bay Long to the north, and Pass La Mer to the east.

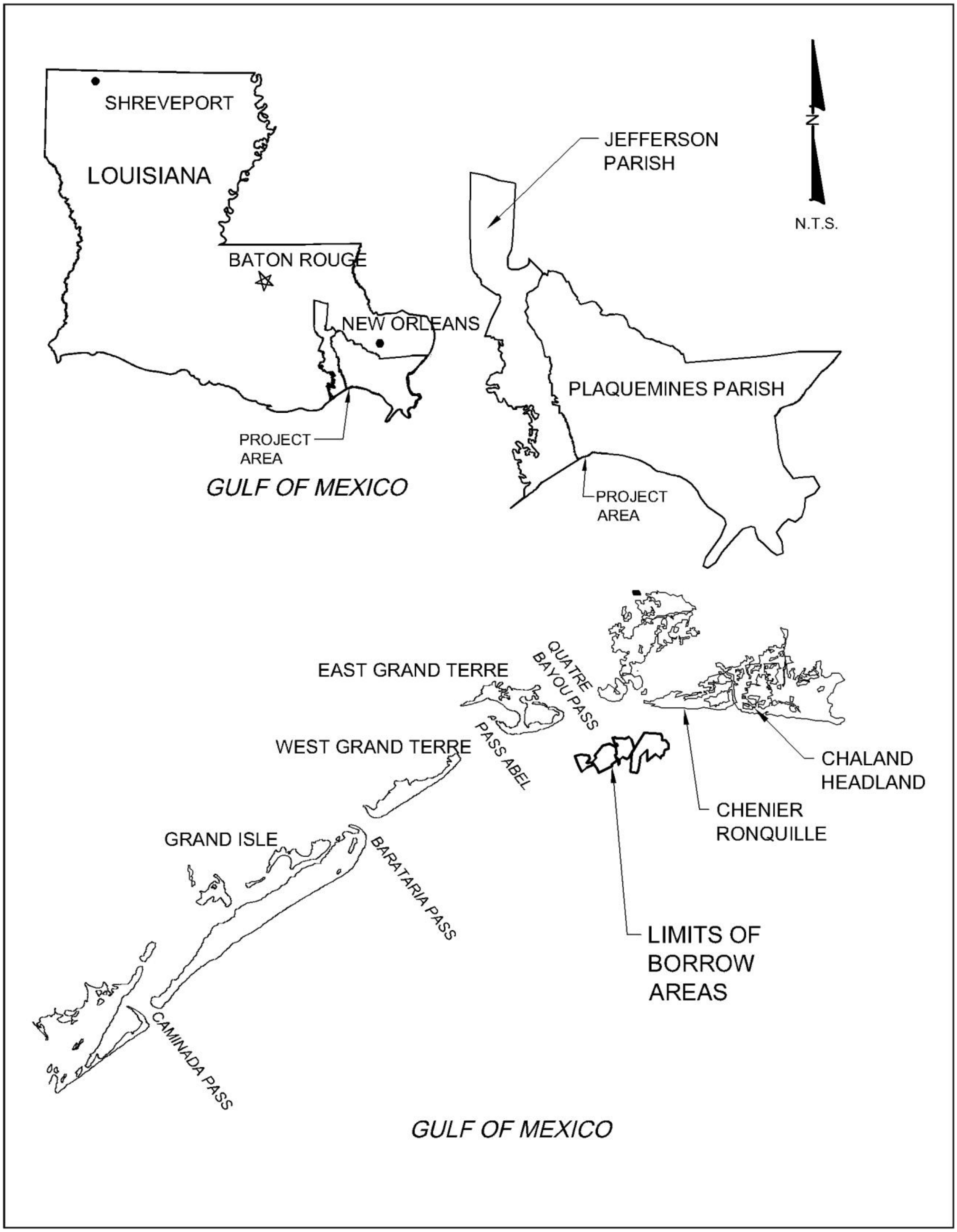


Figure 1. Project Location Map

5 PROJECT AREA DESCRIPTION

Chenier Ronquille is approximately 11,600 feet long along the Gulf of Mexico shoreline. The island is roughly triangularly shaped with the apex located approximately 5,000 feet north of the shoreline. The sandy beach face is very narrow leaving a backing marsh to provide the island width. There are two significant breach areas along the breach face (as of February 2011). The first is located just west of the center of the island and flows into Bay Long. It does not have a clearly defined thalwag but is a combination of shallow flow paths. The second breach located just east of the island's center flows into Bay La Mer. This is a well defined breach with sandy spit features entering the bay. The backing marsh is discontinuous with large open water areas. Several pipelines cross the project area with accompanying pipeline canals and spoil banks, which have contributed to the discontinuous nature of the backing marsh. The oil infrastructure may potentially impact project access and thus project design. This impact is sufficient to merit a separate section to discuss the oil infrastructure (Section 6.6).

5.1 Geologic History

The subsurface geology and surface geomorphology of Louisiana's coastal zone show a complex history of regional plate tectonic events and fluvial, deltaic, and marine sedimentary processes affected by numerous large-magnitude sea level variations (Kulp et al., 2005).

The entire Plaquemines shoreline evolved from major river diversions (avulsions) that promoted large-scale lobes along the Mississippi River delta plain. These delta complexes occur on a millennial (1000-year) time scale. Bay fills are built and abandoned on a centennial (100-year) time scale, whereas smaller sub-deltas exist for an average of 200 years (Coleman and Gagliano, 1964). During the late (destructive) stages of a delta cycle, the abandoned delta complex subsided. As coastal processes reworked the seaward margin, the delta lobe entered a transgressive-destructive phase where erosional headlands, flanking barriers, barrier island arcs, and erosional headlands develop (Penland, Boyd, and Suter, 1988). A mechanism for the genesis and evolution of transgressive depositional systems in the Louisiana deltaic plain was suggested by Penland, Boyd, and Suter (1988) in a three-stage geomorphic model. The evolution began when marine processes transformed the abandoned delta complex into a Stage 1 erosional headland with flanking barriers. During this stage, sediment was supplied to flanking barrier development by shoreface erosion. Relative sea level rise, land loss, and shoreface erosion lead to submergence of back barrier lands and the separation of the Stage 1 barrier from the mainland shoreline, forming the Stage 2 barrier island arc. When relative sea level rise and overwash processes overcame the ability of the barrier island arc to maintain subaerial integrity, submergence initiated the formation of inner shelf shoals (Stage 3). Following submergence, the drowned barrier islands continued to be re-worked by marine/coastal processes to form a marine sand body on the inner shelf, the remnants of which comprise Chenier Ronquille.

The geomorphic history of Chenier Ronquille results in the island being composed of mixed sediment. This type of island typically undergoes rapid shoreline retreat and this is observed at Chenier Ronquille. Campbell (2005) developed a dynamic morphosedimentary model that explains the rapid shoreline recession and observed features on the island. The rapid shoreline retreat results in the exposure of mixed sediment. This concentrates sand on the front face of the island and releases silt to the offshore profile. The sand is then overwashed during large storm events re-exposing the mixed sediment and continuing the cycle.

Chenier Ronquille is a sand deprived barrier island that is disintegrating as evidenced by the numerous breaches and low lying areas along the gulf shoreline. It is most closely classified as a washover flat according to Julie Rosati's (Rosati, 2009) conceptual barrier island types. A washover flat is defined as a "sand-deficient system with a maximum elevation of +1 meter MSL that becomes frequently inundated and overwashed; vegetation exists only when enough time has elapsed between storms; vegetated bayside sediment may be exposed as slightly more erosion-resistant 'islands' in the midst of the sand barrier; back barrier is a vegetated wetland; spits may exist on the flanks" (Rosati, 2009). Due to the dilapidated state of the island and inadequate supply of sandy sediment, natural processes continue to result in a net loss of sediment and subaerial acreage.

5.2 Site Visit Observations

The CWPPRA environmental and engineering work groups conducted a site visit to Chenier Ronquille on June 10, 2009 as part of the Phase 0 investigation. The project team for the 30% Design Report visited the island again on September 16, 2010.

Chenier Ronquille is typical of an eroded Louisiana barrier island with a low cross-shore profile extending no more than a foot or two above mean high water with overwashed sand flats, marsh outcrops, and a discontinuous shoreline (Photo 1).

Photo 1. View West Along Chenier Ronquille Showing the Gulf Shoreline

The two site visits approximately 1 year apart provided an opportunity to photograph changes in shoreline location. The canal plug along the gulf shoreline at the western end of the island

(Photos 2 and 3) offers an excellent comparison point to compare shoreline changes. These photos highlight the shoreline recession that occurred within one year.

Photo 2. View of Canal Plug at East End in June 2009. Note the shoreline to the East (left) and tide level.

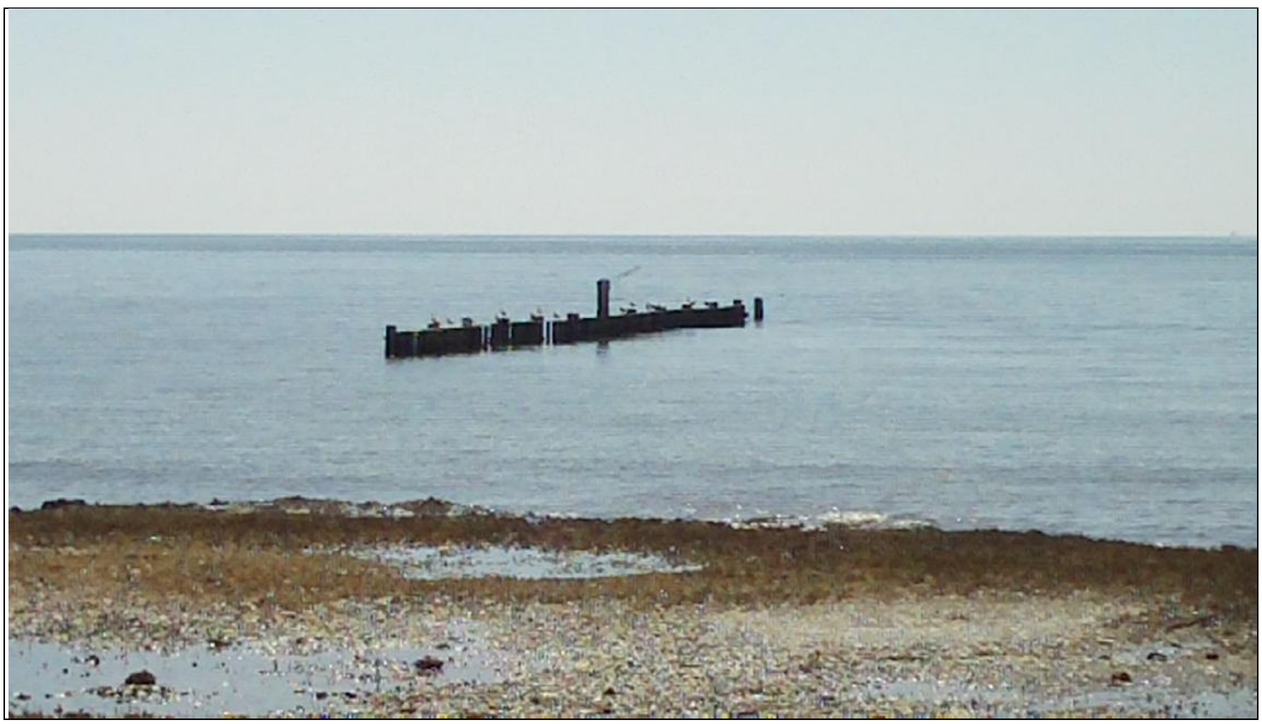


Photo 3. View of Canal Plug at East End in September 2010. Note that the water level is lower in this photo than in Photo 2 but that there is no shoreline close to the canal plug.

Three breaches were observed along the shoreline during the June 2009 site visit. The westernmost breach had closed by September 2010. The center breached remained open (Photo 4) though the breach was relatively shallow and had multiple meandering channels.

Photo 4. View looking north at Sta 55+00 showing the breach in the center of the island.

The breach at the east end of the island was the largest of the three in June 2009 and was over 1,000 feet wide and over 5 feet deep in some sections (Photo 5). By 2010, the throat of the channel was much narrower and had shallowed considerably so that it could be traversed by walking (Photo 6). The spit features within the interior lagoon appeared to have increased in both elevation and expanse.

Photo 5. View East Across the Breach at the East End of Chenier Ronquille in June 2009

Photo 6. View East Across the Breach at the East End of Chenier Ronquille in September 2010.

The shoreline to the east of the large breach is sandier than the beaches on the western side of the island, which exhibited more of discontinuous shoreline with marsh outcrops and oyster shells. During the September 2010 site visit, the Pass La Mer ebb shoal and bypassing bar could be observed due to the breaking waves (Photo 7). The bypassing bar appeared to reconnect with the shoreline east of the large breach, and could explain the sandier eastern end and spit features within the breach.

Photo 7. Sandy Shoreline on the west end of Chenier Ronquille in September 2010. Note the line of breakers locating the bypassing bar.

Offshore of the east end of the project is a single row of piles (Photo 8). The piles are widely spaced but extend for approximately 700 feet alongshore and are approximately 700 feet offshore. Two oil infrastructure platforms are located on the west side of Pass La Mer and several additional pile clusters (oil infrastructure remnants) are located seaward of those. Shoreline features in response to the piles and oil infrastructure were not apparent.

Photo 8. Row of Piles at the East End of Chenier Ronquille.

5.3 Aerial Photography

This section presents an aerial history of the project area from 1998 through 2010 (Photo 9 through Photo 12). This series of aerial photos shows that the west end of the island is undergoing greater shoreline retreat than the east end. The Columbia Gulf Transmission pipeline canal remains closed due to longshore transport even while Point Chenier Ronquille retreats north and east.

The center of the island is characterized by overwash fans which eventually morphed into breaches. The aerials show that large breaches formed after July 2007. Moreover, after the June 2009 site visit, the two western breaches have combined to form a 2,000 foot stretch of discontinuous shoreline backed by segmented marsh outcrops and a single large breach approximately 500 feet wide, though with multiple channels.

The relatively stable east end of the island has experienced less retreat than the west end. The aerials suggest that there may be some sediment transport across Pass La Mer via the ebb shoal. This is suggested by the bulge in the shoreline approximately 1,400 feet from the east end of the island, which is an example of a bypassing bar reattaching to the shoreline. Breaking waves show an outline of the ebb shoal in the 1998 aerial and was visible during the site visits. The ebb shoal appears to have been collapsing due to a lack of sediment being provided from Chaland Headland in subsequent aerials, which is suggested by the decrease in prominence of the shoreline bulge between 1998 and 2007. However, the Chaland Headland Restoration Project (BA-38-2), constructed in 2006, has reintroduced sediment into the coastal system and the Pass La Mer ebb shoal appears to be regrowing and increasing sediment transport to Chenier Ronquille, as seen in the 2008 and 2010 aerials. Closure of Pass La Mer in the near future is considered unlikely because it remained open naturally during the Chaland Headland Restoration Project despite near closure of the pass due to difficulty controlling the beach fill.

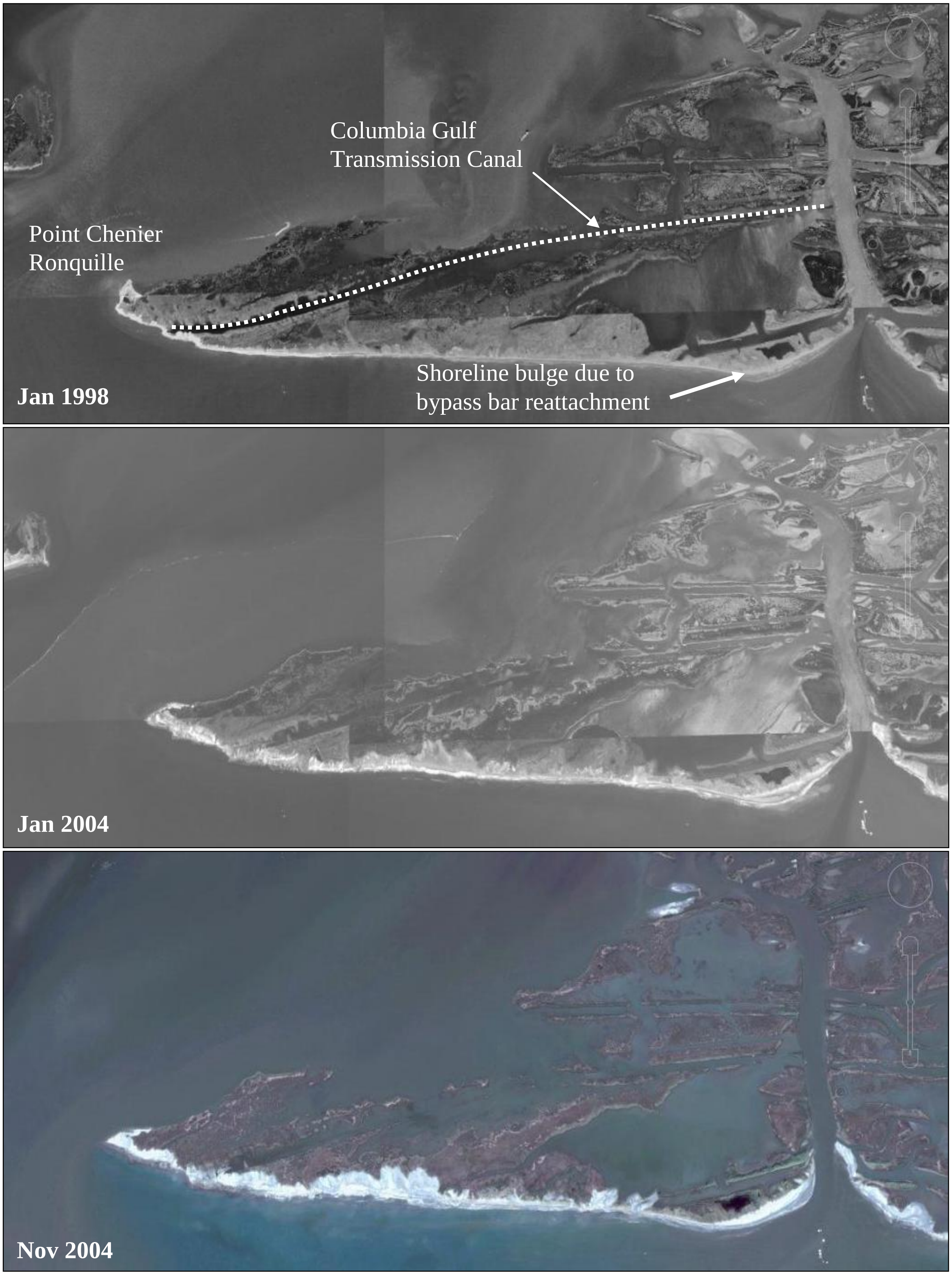


Photo 9. Series of Aerial Photographs showing Jan 1998 (top), Jan 2004 and Nov 2004 (bottom)

Photo 10. Series of Aerial Photographs showing Oct 2005 (top), May 2006 and Oct 2006 (bottom)

Photo 11. Series of Aerial Photographs showing July 2007 (top), Oct 2008, and Aug 2010 (bottom)

Photo 12. Oblique Aerials (May 2009) showing the West End (top), Center, and East End (bottom)

Oblique aerial images, collected by the USFWS, are also available that show the deteriorated state of Chenier Ronquille (Photo 13). These were collected in May 2009 and show the continued disintegration of the island following the October 2008 aerial.

6 PROJECT DATA COLLECTION

6.1 Oceanographic Data

Coastal Planning & Engineering, Inc. (CPE) deployed two Acoustic Doppler Current Profilers (ADCPs) that measured wave height, wave period, wave direction, water level, and current velocities between August 8, 2010 and October 12, 2010. These data were used to calibrate and verify the modeling effort. A complete discussion of the ADCP deployment is included in Appendix B.

The offshore AWAC (Acoustic Wave and Current) ADCP was located 6 miles south of the project site in approximately 35 feet of water while the nearshore ADCP (an Aquadopp) was located immediately west of the project site in Pass Ronquille, where Bay Long and the Gulf of Mexico meet, in approximately 17 feet of water (Figure 2). The offshore ADCP sampled the current profile, water surface elevation, and wave parameters every hour, while the nearshore ADCP sampled every half hour.

Figure 2. ADCP Location Map

The average significant wave height measured during the two month deployment at the offshore gauge location was 1.7 feet arriving from the south-southeast (158°) with a corresponding period of 5.1 seconds.

One significant storm event and four smaller storm events occurred during the two month deployment. A significant wave height of 6.8 feet arriving from the south-southeast (166°) was measured on August 27, 2010.

Tidal currents in Pass Ronquille were measured using the Aquadopp ADCP during the two month deployment. The maximum flood and ebb current velocities measured were 1.9 feet/second (1.1 knots) on August 17, 2010 and 3.0 feet/second (1.8 knots) on August 9, 2010, respectively. The average peak flood and ebb current velocities calculated from the ADCP measurements were 1.3 feet/second (0.8 knots) and 1.9 feet/second (1.1 knots), respectively. The average current speed was 1.0 feet/second (0.6 knots) while the maximum current speed was observed to be in phase with the tide and had a value of 3.0 feet/second (1.8 knots). These values are summarized in Table 2.

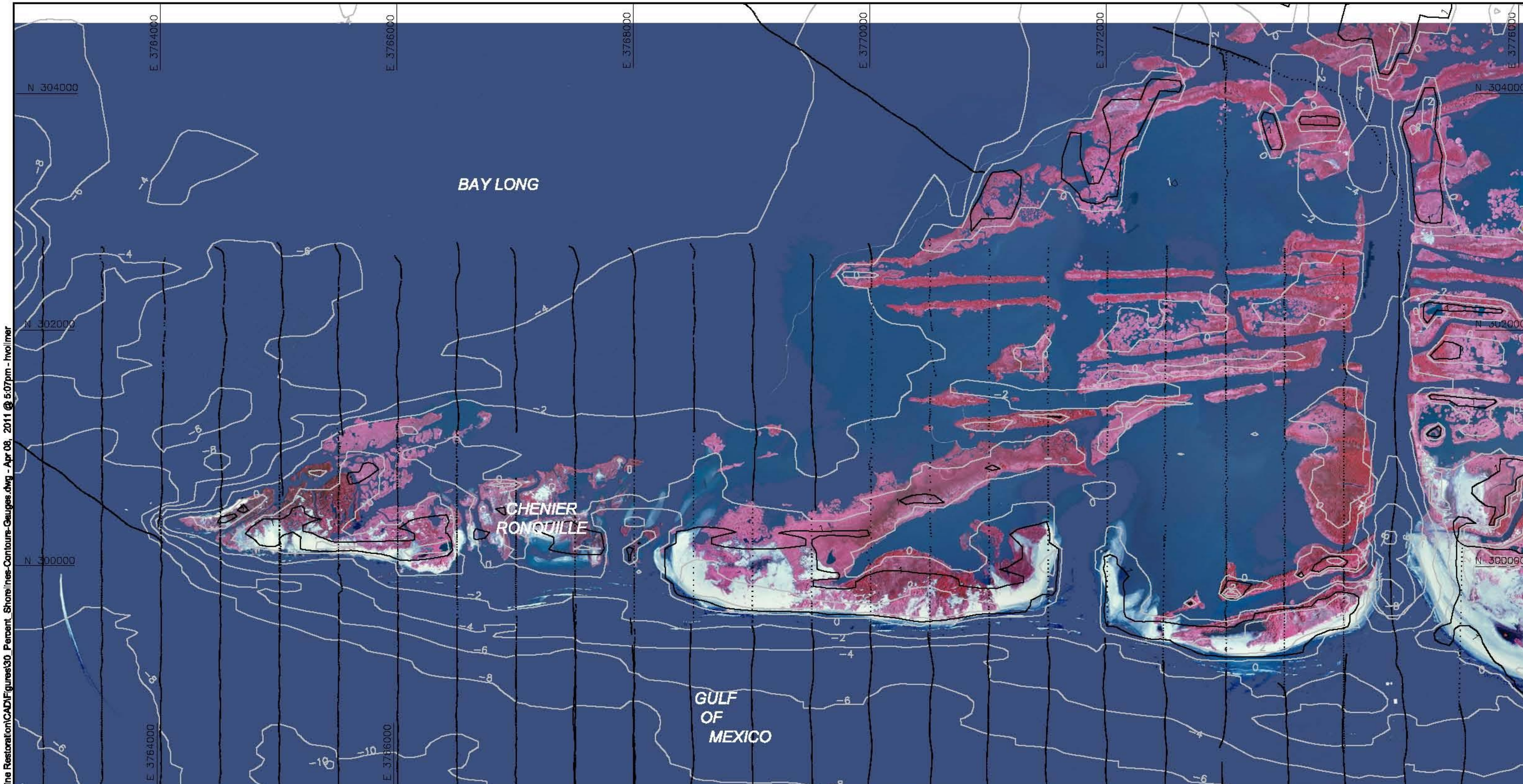
Table 2. Summary of ADCP Tidal Current Measurements

Current Stage	Current Velocity	
	(feet/sec)	(knots)
Maximum	3.0	1.8
Maximum Flood (8/17/2010)	1.9	1.1
Maximum Ebb (8/9/2010)	3.0	1.8
Average	1.0	0.6
Averaged Peak Flood	1.3	0.8
Averaged Peak Ebb	1.9	1.1

6.2 Survey Data

John Chance Land Surveyors (JCLS) conducted hydrographic and topographic surveys of the project area between August 24, 2010 and October 8, 2010 (JCLS, 2010). This data was used in design work, as the input surveys for the modeling efforts, and as a starting basis for WVA acreage analyses. The mean high water shoreline derived from this survey was also used in shoreline change analyses from which shoreline based volumetric comparisons were made. The survey report provided by JCLS is shown in Appendix C.

A total of 32 lines were surveyed: 24 lines in the project area and 8 lines in the adjacent pass and bay. The onshore sections were surveyed using RTK GPS while the offshore data were collected using hydrographic surveying methods. The project area survey lines were spaced at 500 feet and extended approximately 2,600 feet Gulfward and bayward of the project baseline. One of the project profiles extended approximately 4,200 feet bayward to capture the extent of the backing marsh near the eastern end of the project. A map depicting the locations of the points surveyed along with the bathymetry developed from the survey data is shown in Figure 3.



LEGEND:

SEPTEMBER 2010 CONTOURS (FT, NAVD)
SEPTEMBER 2010 COLLECTED SURVEY POINTS

NOTES:

1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
2. DATE OF PROJECT AREA PHOTOGRAPH: AUGUST 2010.

0 400 800
GRAPHIC SCALE IN FT

Figure 3. Contour Plot of 2010 Hydrographic and Topographic Survey

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Other recent surveys of the project site include BICM, FEMA, NOAA, USGS, Coast 2050, and Chaland Headland Restoration. Barrier Island Comprehensive Monitoring (BICM) hydrographic data was collected in 2006. The Coast 2050 and the Chaland Headland Restoration survey (conducted in 2000) are the only data sets that contained both topographic and bathymetric data collected concurrently; however, topographic data was only available for two lines that differed during both surveys. LIDAR data (collected in 1998, 2003, and 2006) provides the most comprehensive data set for topography; however, the lack of water clarity restricts LIDAR from providing bathymetric data. Table 3 summarizes the available survey and shoreline data.

Table 3. Summary of Available Survey and Shoreline Data

Date	Data Type	Source/Description	Reference
September 2010	Bathymetry / Topography	Project Survey	JCLS, 2010
December 2006	Bathymetry / Topography	BICM LIDAR Survey	LDNR, 2008
March 2003	Topography	FEMA LIDAR Survey	USACE, 2003
November 2000	Bathymetry / Topography	Coast 2050 Survey	Morris P. Hebert, 2001
October 2000	Bathymetry / Topography	Chaland Profile Survey	CPE, 2003
November 1998	Topography	NOAA LIDAR Survey	NOAA, 1998
~1989	Bathymetry	USGS DEM	USGS, 1998
1988, 1973, 1956, 1932, 1884	Shoreline	USGS Atlas of Shoreline Changes	Williams et al, 1992
1973-1978, 1922-1934, 1855-1877	Shoreline	USGS Vector Shorelines	Miller et al, 2004
Compilation of 1930's to present	Bathymetry	NGDC GEODAS	NOAA, 2009

6.3 Geotechnical Data

Fugro Consultants collected nine borings in the project fill area between September 10, 2010 and September 13, 2010 (Fugro, 2010). This work was performed to assess pertinent engineering soil properties of the existing soils, estimate the settlement of the natural underlying soils due to the placement of marsh and beach fill material, and to determine the stability of the primary containment dike. The geotechnical report submitted by Fugro Consultants, discussing their field work methodology, analyses, and conclusions, is included in Appendix D. A summary of the data collection effort is provided in the following paragraphs.

The borings were collected via marsh buggy-mounted drilling rig using wet-rotary drilling techniques. Four of the soil borings (B2, B5, B7, and B9) were drilled to a depth of 60 feet below the mudline, and five borings (B1, B3, B4, B6, and B8) were drilled to a depth of 40 feet below the mudline. The boring locations are shown in Figure 4.

Figure 4. Location of Project Area Borings

Based on review of the field and laboratory tests performed, the subsurface soils generally consist of alternating layers of granular and cohesive materials from the mudline to the completion depth of the borings. Borings collected on the beach face suggest that the sand lens thickness varies between 6 and 18 feet, with the sand lens thickening towards the west. The borings also suggest that the beach face is composed of 39% silt and 61% sand between 1 foot, NAVD and -9 feet, NAVD, while the island as a whole is composed of 59% silt and 41% sand between 1 foot, NAVD and -9 feet, NAVD.

6.4 Borrow Area Geotechnical Data

No additional data was collected within the borrow area in order to develop this report. Borrow area designs are based on geotechnical work previously conducted for the Chaland Headland Restoration Project (BA-38-2) and the East Grand Terre Island Restoration Project (BA-30). As-built surveys of the dredged borrow areas have been used to estimate the volume of fill remaining in the borrow areas.

6.5 Magnetometer Data

John Chance Land Surveyors (JCLS) conducted a magnetometer survey of the project area between September 24, 2010 and October 15, 2010. The magnetometer survey confirmed the locations of existing pipelines. Other anomalies range in amplitude from 18 to 4,433 gammas and durations from 9.5 to 207.1 feet. The contacts are presumed to represent articles of ferrous debris that are either buried below the mudline or are too small to be acoustically detected. A table of the recorded magnetic anomalies, identified and unidentified, are listed on the base map shown in the survey report provided in Appendix C (JCLS, 2010). None of the magnetic anomalies indicated pipelines within the project area that were not previously recorded in the SONRIS database.

6.6 Oil Infrastructure

Chenier Ronquille has a significant amount of oil and gas infrastructure in the project area. Figure 5 shows the location of these known and/or suspected pipelines. Table 4 summarizes the owners of the various pipelines in the project area. The data was taken from the SONRIS website.

Table 4. Summary of Pipeline Owners in Project Area.

Owner	Permit Number	Contact	Phone
Amoco Production Co.	P19900096		225-654-5443
Plains Pipeline (previously BP Oil Pipeline Co.)	P19980106		281-366-0845
Plains Pipeline (previously BP Oil Pipeline Co.)	P19890612		281-366-0845
Plains Pipeline (previously BP Oil Pipeline Co.)	P19890870		281-366-0845
Plains Pipeline (previously BP Oil Pipeline Co.)	P19971154		281-366-0845
Plains Pipeline (previously BP Oil Pipeline Co.)	P19991198	Mr. J. Erceg	281-366-0845
Plains Pipeline (previously BP Oil Pipeline Co.)	P20031452	Mr. David Twaddle	281-366-0845
Chevron Pipeline Co.	P20080721	Mr. Jeffrey Downing	713-432-3336
Chevron Pipeline Co.	P20030800	Mr. Jeffrey Downing	713-432-3336
Chevron Pipeline Co.	P20031479	Mr. Jeffrey Downing	713-432-3336
Columbia Gulf Transmission	P20041396	Mr. Darren Duhon	337-266-4672
Louisiana Interstate Gas Co.	P19800218		
Plains Pipeline, LP.	P20080362	Mr. Rusty Cavalier	504-393-6282
Plains Pipeline, LP.	P20070460	Mr. Rusty Cavalier	504-393-6282
Raine Oil & Gas Co.	P19811015		337-235-6964
Swift Energy Operating, LLC.	P20030850		337-542-4440

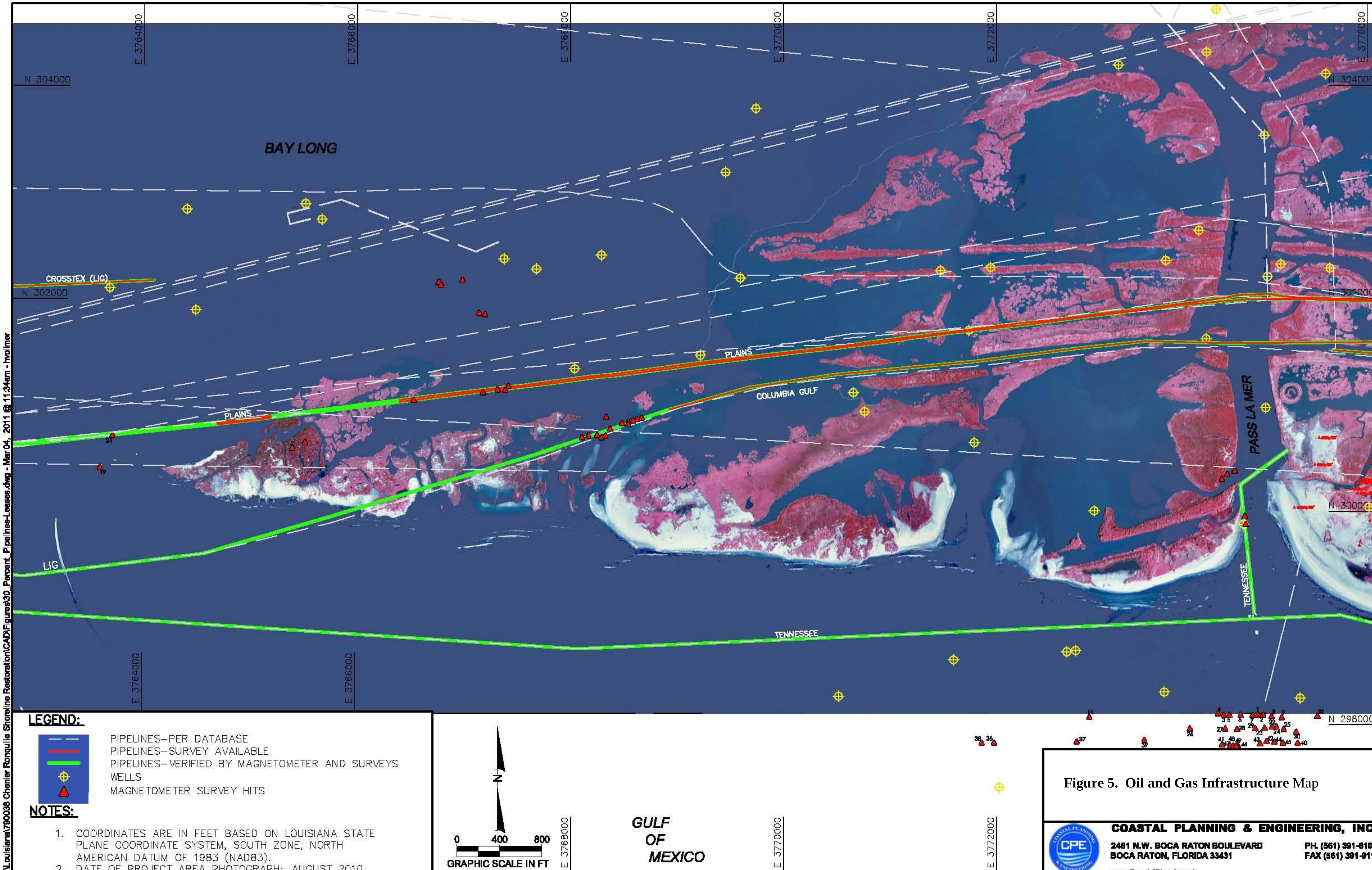


Figure 5. Oil and Gas Infrastructure Map

6.7 Oyster Leases

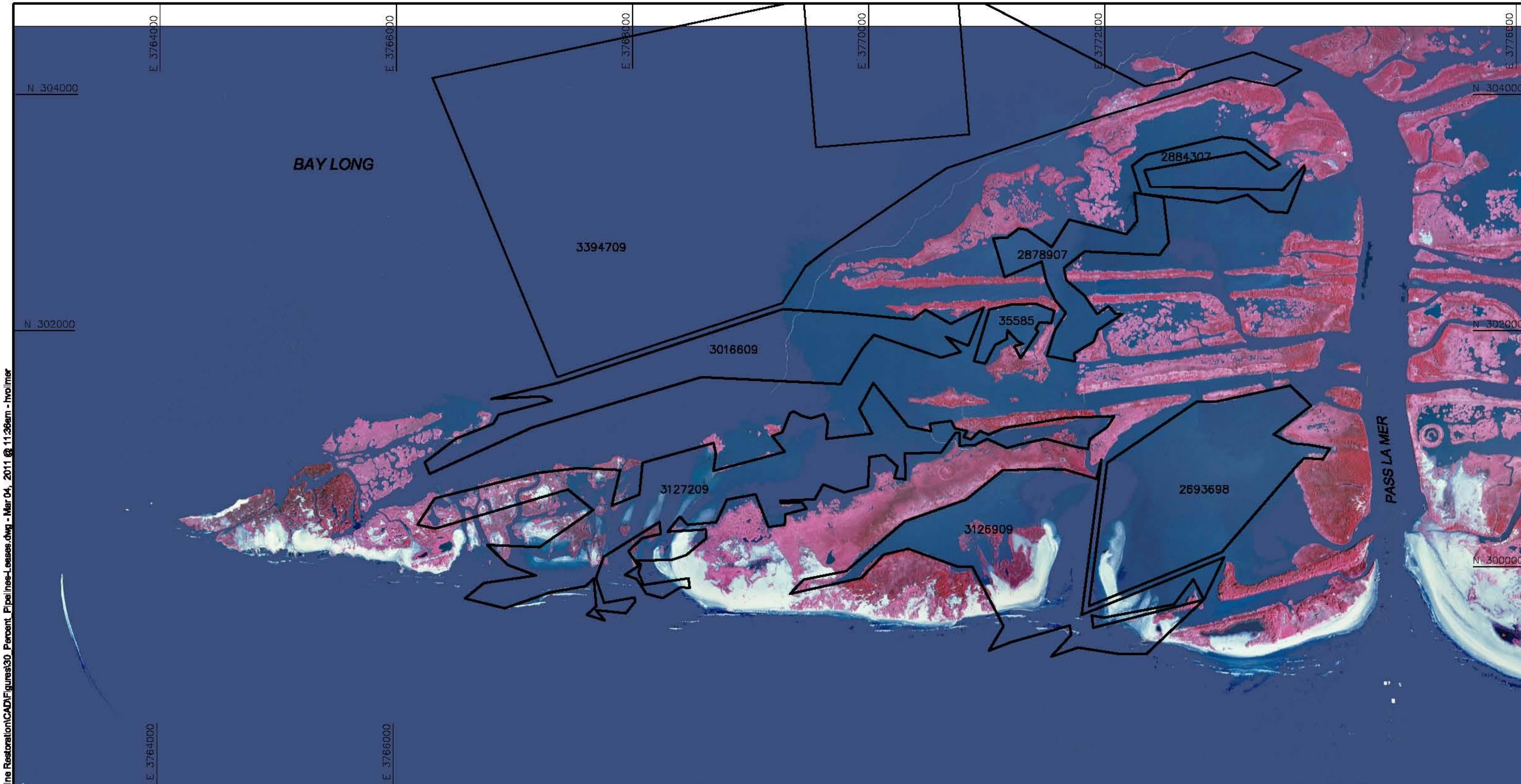
There are several oyster leases within the footprint of the project. A list of oyster leases and the respective owners' contact information is summarized in Table 5 and shown in Figure 6.

Table 5. Oyster Leases and Respective Owners

Lease Number	Lease Acreage	Lessee Name	Phone Number	Address	Parish
35585	5	Charles Atzenhoffer and Guy Adams	(504) 341-3764	850 Avenue C Westwego, LA 70094	Jefferson
2693698	52	Srecka Taliancich	(504) 912-1718	30583 Highway 11 Buras, LA 70041	Plaquemines
2878907	15	Guy Adams	(504) 341-9049	1307 Maple Street Westwego, LA 70094	Jefferson
2884307	8	Srecka Taliancich	(504) 912-1718	30583 Highway 11 Buras, LA 70041	Plaquemines
3016609	37	Eldon Frazier	(985) 632-6623	14640 W Main Street Cut Off, LA 70345	LaFourche
3126909	48	Valerie Garbin	(504) 391-1015	113 Avenue B Buras, LA 70041	Plaquemines
3127209	62	Vatroslav Garbin	(985) 657-9393	113 Avenue B Buras, LA 70041	Plaquemines
3394709	212	Charlotte Turner	(504) 756-1473	P.O. Box 175 Port Sulphur, LA 70083	Plaquemines

6.8 Land Owners

Louisiana Land and Exploration Co. (LL&E) is the only land owner in the project area.



N 298000

NOTES:

1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
2. DATE OF PROJECT AREA PHOTOGRAPH: AUGUST 2010.
3. OYSTER LEASE INFORMATION OBTAINED FROM USGS (2011).

N

0 400 800

GRAPHIC SCALE IN FT

E 3768000

GULF OF MEXICO

E 3770000

E 3772000

Figure 6. Oyster Lease Map

COASTAL PLANNING & ENGINEERING, INC.
2481 N.W. BOCA RATON BOULEVARD
BOCA RATON, FLORIDA 33431
PH. (561) 391-8102
FAX (561) 391-9116
www.CoastalPlanning.net

7 BORROW AREAS

It is proposed to use the sediment remaining within the borrow areas developed for the East Grand Terre Island Restoration project (BA-30) and Chaland Headland Restoration Project (BA-38-2) to construct the Chenier Ronquille project. Six beach and marsh fill borrow areas were identified for the East Grand Terre Island Restoration Project (CPE, 2004), which was completed in November 2010. Three of the six offshore borrow areas (WGT, M-1 and M-2) were exhausted during construction of the East Grand Terre Island project (CPE, 2011). A portion of the marsh and beach fill in the D-1 borrow area was excavated, but a majority of the borrow area remains available for construction of the Chenier Ronquille project. The Quatre Bayou borrow area was developed for the Chaland Headland Restoration Project (BA-38-2) (CPE, 2003b). Sand resources were depleted from this borrow area but it may still be used as a marsh fill source (CPE, 2008). No additional borrow area investigations are required in order to use the four remaining borrow areas for the Chenier Ronquille project. Table 6 summarizes the volumes available within the various borrow areas while Figure 7 shows the location of these borrow areas in reference to Chenier Ronquille.

Table 6. Summary of Borrow Areas and Volumes

Borrow Area	Mean Grain Size (mm)	Percent Silt (%)	Beach Fill Volume (cy)	Marsh Fill Volume (cy)
S-1	0.11	15	1,651,000	-
S-2	0.11	17	691,000	-
D-1 (sand deposit)	0.11	28	1,931,000	-
D-1 (overburden)	-	-	-	1,393,000
Quatre Bayou	-	-	-	5,088,000
Total			**4,273,000**	**6,481,000**

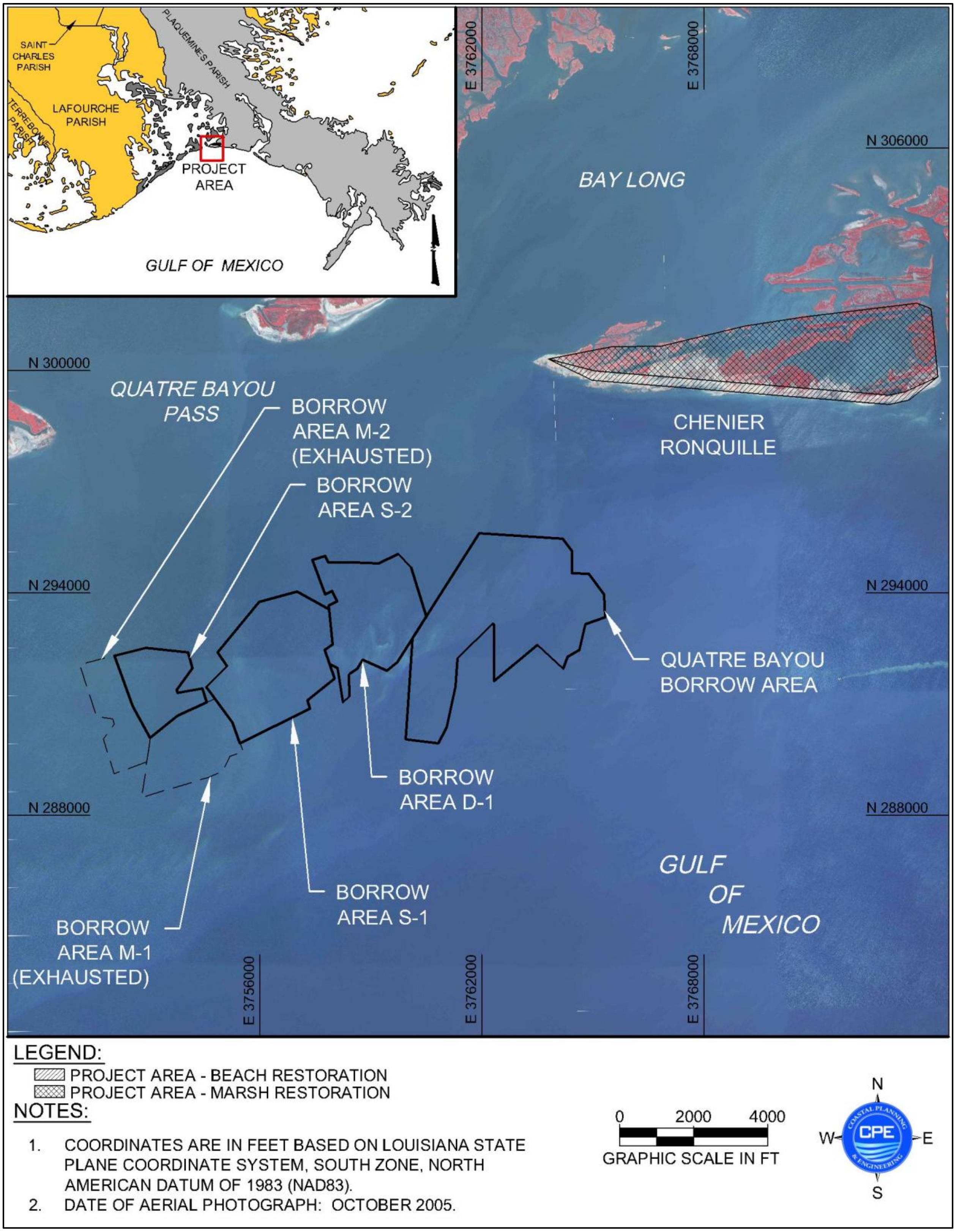


Figure 7. Borrow Area Location Map

7.1 Borrow Area S-1

Borrow area S-1 is located approximately 2.5 miles southwest of Chenier Ronquille. No overburden removal is necessary to access the sand. The S-1 borrow area contains approximately 1,651,000 cubic yards of material, characterized with a mean grain size of 0.11 mm sand with a silt content of 15%. The borrow area is located in water depths ranging from –10 to –11.5 feet, NAVD, with cut depths ranging from 4 to 6 feet. The borrow area has 2 cut elevations (-15 feet and -16 feet, NAVD). The relatively thin dredge cuts may increase dredge losses, lower productivity, and increase the unit price compared to the unit prices for East Grand Terre, which were based on cut thicknesses of 6 to 8 feet.

Note that the cut depth in the southern section of this borrow area has been increased from -14 feet, NAVD to -15 feet, NAVD in order to increase available volumes and make for a more efficient cut. This increased the silt content compared to the previous borrow area design but it was determined that the increased cut depth and volume offset this issue.

Within the borrow area limits, potential cultural resources have been identified that could impact the contractor’s dredging operations. These avoidance areas are located in the southern half of the borrow area and require 100-foot radius dredging buffers.

A 3-foot thick overdredge area has been added to all of the borrow area designs (labeled as maximum depth of equipment) to allow the Contractor to remove as much of the sand from the borrow area as possible. Allowing dredging below the sand lens may introduce additional silts into the beach fill, but this has been accounted for in project performance.

7.2 Borrow Area S-2

Borrow area S-2 is located west of borrow area S-1 and approximately 2.8 miles southwest of the project area. As with borrow area S-1, the sands are located on the surface and no overburden removal is necessary. The total dredgeable volume in this borrow area is approximately 691,000 cubic yards. The material is characterized with a mean grain size of 0.11 mm sand with a silt content of 17%. The borrow area is located in water depths ranging from -9.0 to -10.0 feet, NAVD, with cut depths ranging from 3 to 7 feet. The borrow area has 2 cut elevations (-14.0 feet and -16 feet, NAVD). The relatively thin dredge cuts may increase dredge losses, lower productivity, and increase the unit price.

Within the borrow area limits, potential cultural resources have been identified that could impact the contractor’s dredging operations. These avoidance areas are located near the northern and southern perimeter of the borrow area and require 100-foot radius dredging buffers.

7.3 Borrow Area D-1

Borrow area D-1 borders the eastern extent of borrow area S-1 and is located approximately 1.9 miles southwest of the project area. It is the closest sand borrow area to the project area requiring the shortest pumping distance. Unlike borrow areas S-1 and S-2, most of the sands in D-1 are located below a layer of overburden material though there is one section in the northeast section with surficial sand deposits. The overburden material is compatible for marsh construction but would have to be removed prior to excavating sand for beach construction. The beach fill material is characterized with a mean grain size of 0.11 mm sand with a silt content of 28%. The borrow area is located in water depths ranging from -10 to -14 feet, NAVD, with cut depths ranging from 2 to 15 feet. The borrow area has 5 beach fill cut elevations that range from -24 to -29 feet, NAVD.

During construction of East Grand Terre Island Restoration (BA-30) project, a portion of the southwest section of the borrow area was dredged. The overburden material was excavated for marsh construction exposing the underlying sand layer. Within this area, approximately 41,000 cubic yards of sand was mined and placed along the constructed dune (CPE, 2011). After construction of East Grand Terre Island, the total dredgeable sand volume in this borrow area is approximately 1,931,000 cubic yards.

Overlying the sandy beach fill material is an overburden layer adequate for marsh construction. The thickness of this layer is between 5 and 10 feet. The mixture of sand and fine sediments comprising the overburden is preferable as marsh fill compared to simply constructing the marsh with silts and clays. The Contractor will therefore be required to use borrow area D-1 for marsh fill material prior to dredging the underlying sand.

During construction of the East Grand Terre project, approximately 355,000 cubic yards of marsh fill was dredged from borrow area D-1 (CPE, 2011). Approximately 1,393,000 cubic yards of marsh fill is available within borrow area D-1 for construction of the Chenier Ronquille project.

Within the borrow area limits, a potential cultural resources has been identified that could impact the contractor’s dredging operations. This avoidance area is located near the eastern perimeter of the borrow area and requires 100-foot radius dredging buffer.

The contractor will be required to dredge marsh fill material from borrow area D-1 prior to excavating the Quatre Bayou borrow area (see Section 7.4), if necessary.

7.4 Quatre Bayou

The Quatre Bayou borrow area is the closest borrow area to Chenier Ronquille with the northern boundary being 0.9 miles south of the west end of the project. Quatre Bayou was dredged during the construction of Chaland Headland (BA-38-2) project. Most of the beach compatible sand and a portion of the marsh compatible material were removed at this time. Therefore, the remaining material is mostly muds, due to the side casting of overburden back into the borrow area, and is only suitable for marsh fill.

It is estimated that Quatre Bayou contains over 5 million cubic yards of marsh fill material. However, this volume is based on a survey conducted immediately after construction, and some consolidation of the side cast material has likely occurred, which would reduce the available volume. Infilling of the borrow area may also have occurred, which would increase the available fill volume.

Based on the as-built surveys following construction of the Chaland Headland project, cut thicknesses range from 7 to 20 feet.

Previous dredging of the Quatre Bayou borrow area has left a depression relative to the surrounding area. Therefore, Quatre Bayou could be used as a disposal site if the Contractor needed to sidecast any overburden from D-1 in order to access the sand resources. This option was included in the East Grand Terre Restoration project but not required.

7.5 Borrow Area Impact Analysis

A borrow area impact analysis was not performed for this report. However, a borrow area impact analysis was performed during the design of the East Grand Terre Island Restoration (CPE, 2005) and Chaland Headland Restoration (CPE, 2003b) projects. These reports showed that the complete dredging of the borrow areas would not result in adverse impacts to the adjacent shorelines. No additional borrow area impact analyses were performed as part of the Chenier Ronquille design.

8 PHYSICAL CHARACTERISTICS OF THE PROJECT AREA

8.1 Tides

The Grand Isle tide gauge is the only tide gauge located in the vicinity of Chenier Ronquille with published datums. The Grand Isle tide gauge is located at the eastern end of Grand Isle, on the bay side of the island, approximately 8.5 miles west of the project area. Tidal datums published for the Grand Isle tide gauge are shown in Table 7.

Table 7. Grand Isle Tidal Datums

Datum	Elevation (ft, MLLW)	Elevation (ft, NAVD)
Mean Higher-High Water	1.06	0.81
Mean High Water	1.05	0.80
Mean Diurnal Tide Level	0.53	0.28
Mean Tide Level	0.53	0.28
Mean Sea Level	0.54	0.29
Mean Low Water	0.01	-0.24
Mean Lower Low Water	0.00	-0.25

Note: NAVD '88 values referenced to Geoid '09

The tide gauge at Grand Isle is only referenced to Mean Lower Low Water because the vertical reference to NAVD has been removed from the NOAA website. The Polder Vertical Datum Report (USACE, 2010) suggests that the elevation of Mean Sea Level (MSL) is 0.29 feet, NAVD referenced to Geoid 2003.
(http://www.mvn.usace.army.mil/ed/edss/datumreports/Grand_Isle_PVDR.pdf).

In 2000, NOAA estimated that 0.00 feet, MLLW = -0.54 feet, NAVD. (NOAA, 2000, http://www.co-ops.nos.noaa.gov/benchmarks/8761724.html). During the Louisiana Emergency

Berm Project, CO-OPS estimated that 0.00 MLLW = -0.55 NAVD at Grand Isle (e-mail from Robert Burrows to Tim Osborne dated October 19, 2010) using V-Datum (v.2.3.0).

The tidal datums published by NOAA NOS CO-OPS were developed using five years of tide data, collected between January 2002 and December 2006, referenced to the 1983 to 2001 tidal epoch. However, tidal datums published for the Grand Isle tide gauge may not be representative of tidal datums measured at the project site as the Grand Isle tide gauge is located on the north side of the island within a bay and not on the open coast.

Other tide gauges in the vicinity of Chenier Ronquille include the tide gauge installed during the East Grand Terre Island Restoration Project (BA-30), ENCOS DCP W4A installed for the Emergency Berm Project, and the Coastwide Reference Monitoring System (CRMS) Site 0171 tide gauge. The tide gauge installed for the East Grand Terre Island Restoration Project was located on the east side of East Grand Terre adjacent to Quatre Bayou Pass, approximately 2.6 miles west of the project site, and was active between August 24, 2010 and September 30, 2010. The ENCOS tide gauge was located immediately north of Chenier Ronquille in the backing bay and data was analyzed between June 14, 2010 and September 28, 2010. The CRMS tide gauge is located in the marsh backing Chenier Ronquille and data was analyzed between May 22, 2007 and August 19, 2010. A map depicting the location of these tide gauges is shown in Figure 8.

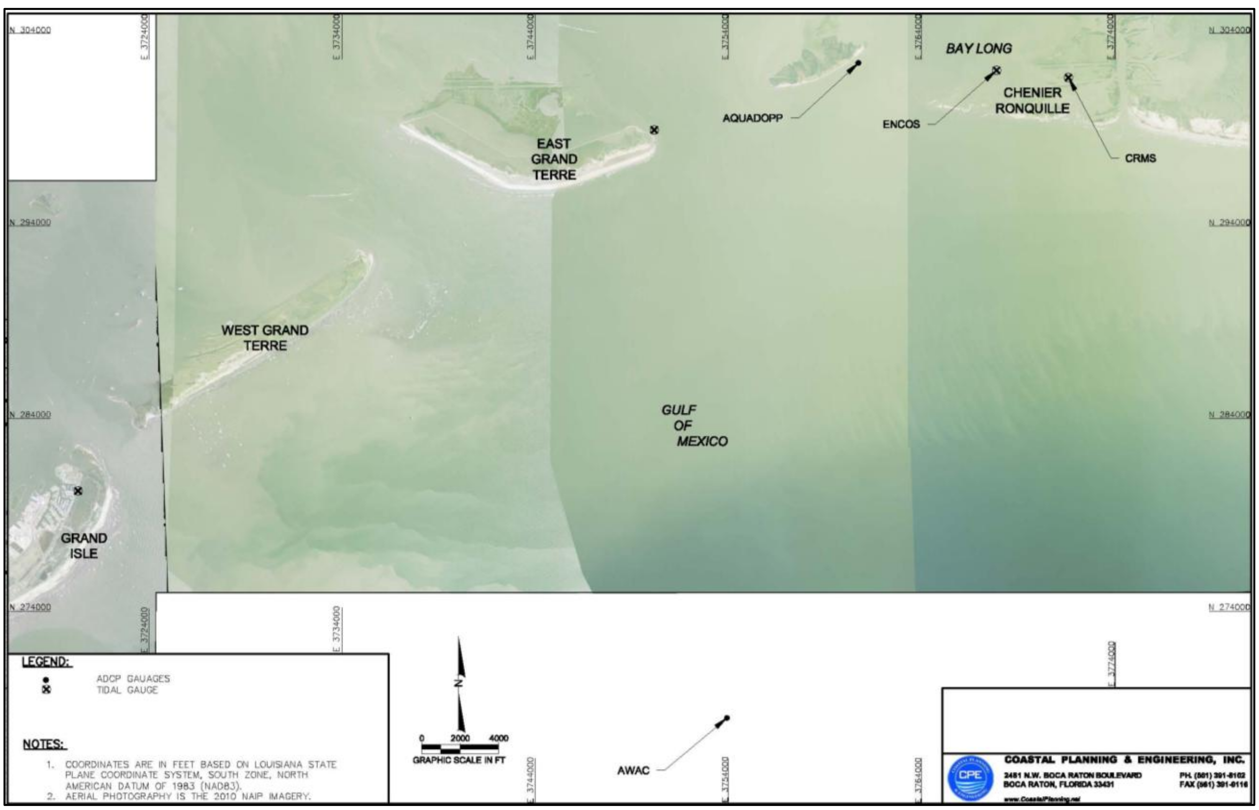


Figure 8. Tide Gauge Location Map

Project area tidal datums were calculated using ENCOS tide data and procedures outlined in the NOAA NOS CO-OPS Computational Techniques for Tidal Datums Handbook (NOAA, 2003). The ENCOS tide data was the data set chosen to use in tidal datum calculations as it is located in an open water area with direct access to the Gulf of Mexico and had over a month of continuous data. Although the East Grand Terre tide gauge was located directly along the open coast, the discontinuous data was of insufficient length and quality to use as a secondary source to calculate tidal datums.

Both the ENCOS (subordinate) and Grand Isle (control) tide data were inspected to determine a time period where both data sets provide continuous data. The tide data was considered continuous as long as gaps between adjacent points were less than a quarter of a tide cycle. It was determined that the ENCOS datum analysis could be performed between July 16, 2010 and September 28, 2010.

The tide data was then post-processed to calculate tidal datums over the analysis period for both the ENCOS and Grand Isle tide gauges. Hourly water levels were obtained from the measurements, using either the collected data or linear interpolation between data points. The hourly data were then analyzed to calculate, over the period of simultaneous comparison, mean higher-high water, mean high water, mean diurnal tide, mean tide, mean sea level, mean low water, mean lower-low water, great diurnal tide range, and mean tide range. These tidal parameters and ratios were then compared with published values, per procedures outlined in the Computational Techniques for Tidal Datums Handbook (NOAA, 2003), to determine tidal datums for the ENCOS tide gauge. Tidal datums for the project site, derived from the ENCOS tide gauge, are presented in Table 8.

Table 8. Chenier Ronquille Tidal Datums

Datum	Elevation (ft, NAVD)
Mean Higher-High Water	0.95
Mean High Water	0.94
Mean Diurnal Tide Level	0.33
Mean Tide Level	0.33
Mean Sea Level	0.33
Mean Low Water	-0.27
Mean Lower Low Water	-0.28

Comparison of the tides measured at Chenier Ronquille and Grand Isle suggest that a larger tide range is observed along the open coast than in the backing bay. The tide range observed at Chenier Ronquille is approximately 0.2 feet greater than that observed at Grand Isle. An example of this tide range difference is provided in Figure 9.

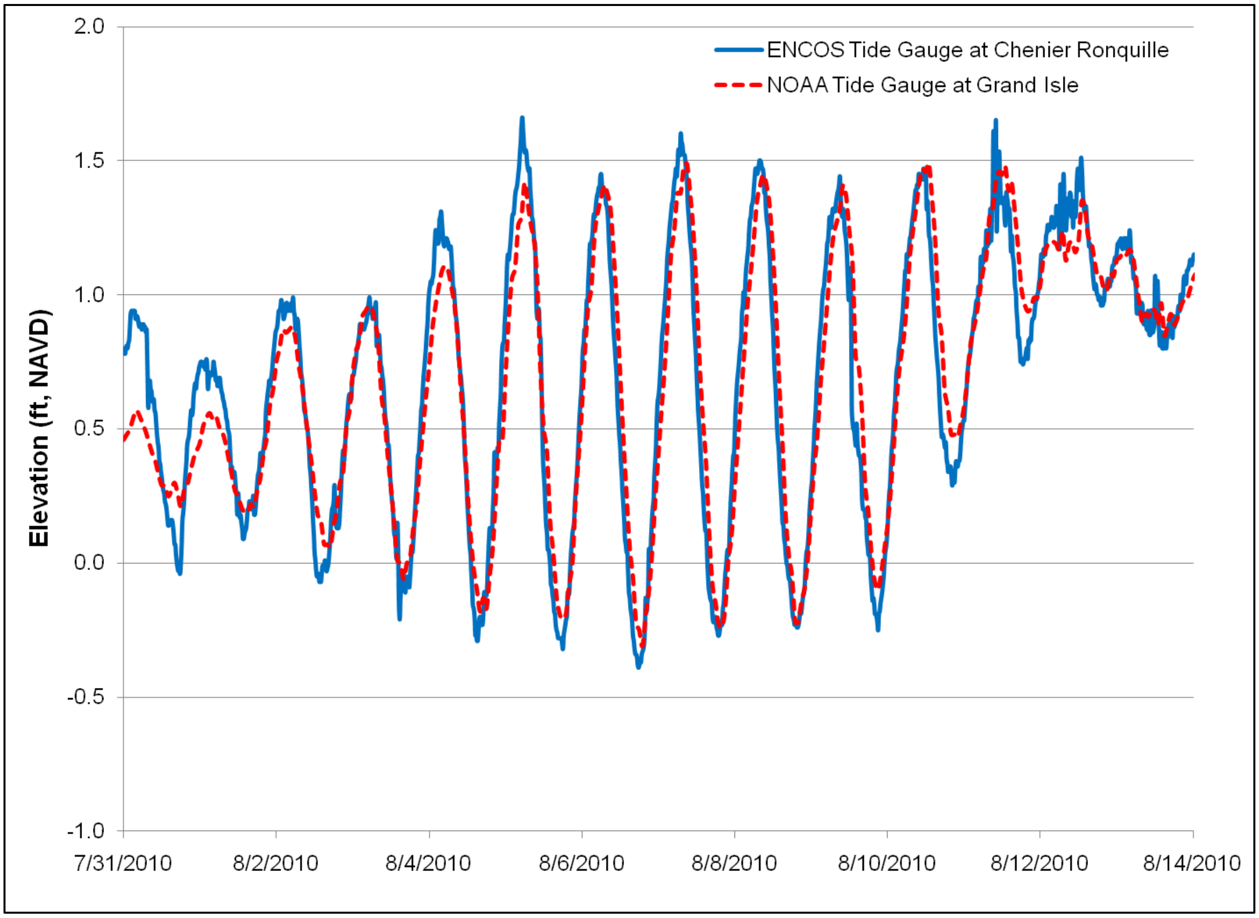


Figure 9. Grand Isle and Chenier Ronquille Tide Comparison

The CRMS tide gauge recorded data over a longer time period than the ENCOS gauge, but this gauge is located in the marsh backing Chenier Ronquille and may underestimate low water levels due to tide water entrapment in the marsh. Real time CRMS data is being collected in feet, NAVD but using Geoid '03. The data was updated to use Geoid'09. The MHW and MLW elevations reported at the CRMS station between June 2007 and June 2010 prior to correlation to the epoch are +1.26' NAVD and -0.04' NAVD, respectively. Following correlation to the epoch, the mean high and mean low water elevations are similar to those listed in Table 8. This indicates that we are in a period of slightly elevated tidal ranges and that tidal elevations are expected to be higher than the average shown in Table 8 in the earlier years of the project life.

8.2 Currents

Tidal currents in Pass Ronquille were measured using the Aquadopp ADCP during the two month deployment (Section 6.1). The average current speed was 1.0 feet/second (0.6 knots), while the maximum current speed was observed to be in phase with the tide and had a value of 3.0 feet/second (1.8 knots). Additional tidal current velocities are summarized in Table 2.

8.3 Winds

Wind statistics generated for the project site utilize wind data collected at the Grand Isle tide station from December 17, 1984 to December 22, 2009 (NDBC, 2010). The average measured wind speed is 11.0 miles/hour with a corresponding direction of 155° (SSE). With the exception of tropical storm events, the strongest winds occur between November and April, with the weakest winds occurring in July and August. The wind direction varies from 123° (SE) in October to 196° (SSW) in July. Monthly wind statistics appear in Table 9 while directional wind statistics are shown in Figure 10 and Figure 11.

Table 9. Monthly Wind Statistics

	Speed (mph)		Direction
Month	**Average**	**Maximum**	**(deg)**
January	12.7	42.5	153
February	12.3	38.0	152
March	12.1	51.7	159
April	12.0	43.6	156
May	10.5	32.9	156
June	9.4	49.2	168
July	8.4	61.1	196
August	8.3	75.4	176
September	10.6	49.9	131
October	11.9	48.3	123
November	12.3	39.6	136
December	12.3	41.4	148
Average	11.0	-	155

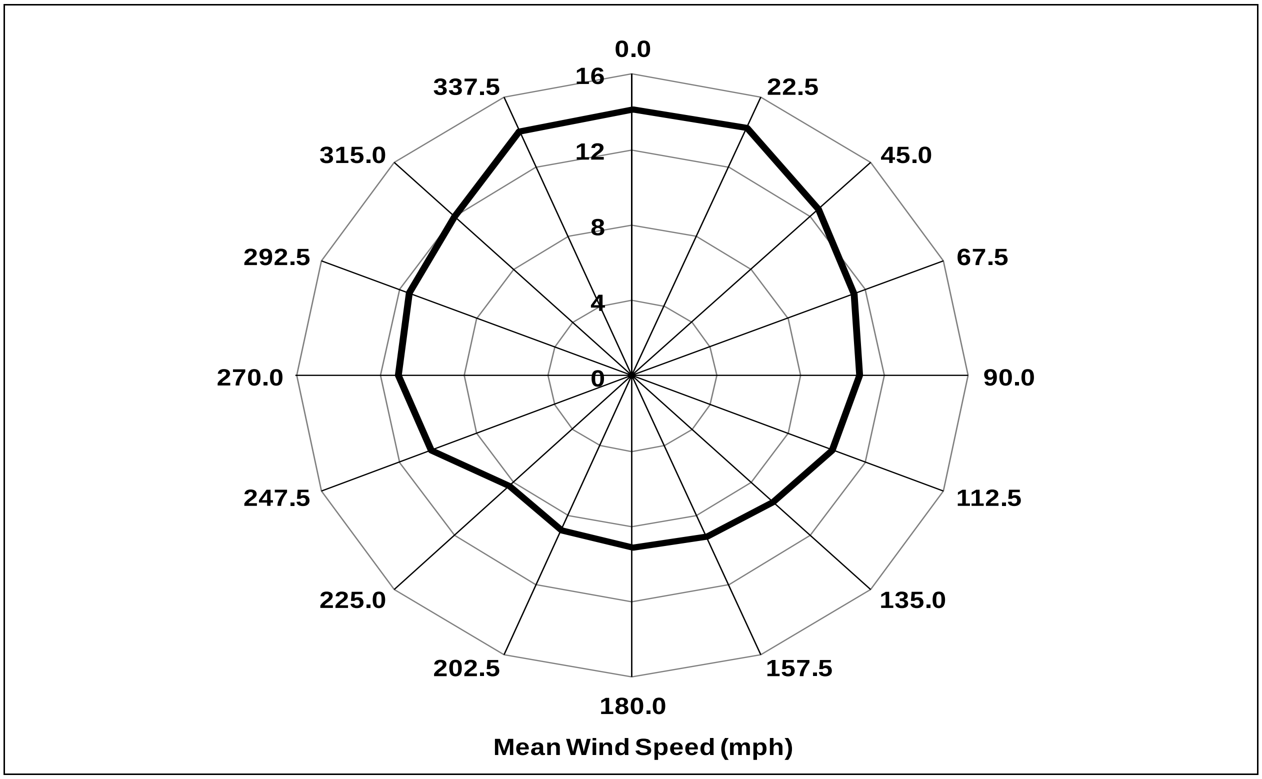


Figure 10. Directional Wind Statistics - Mean Wind Speed

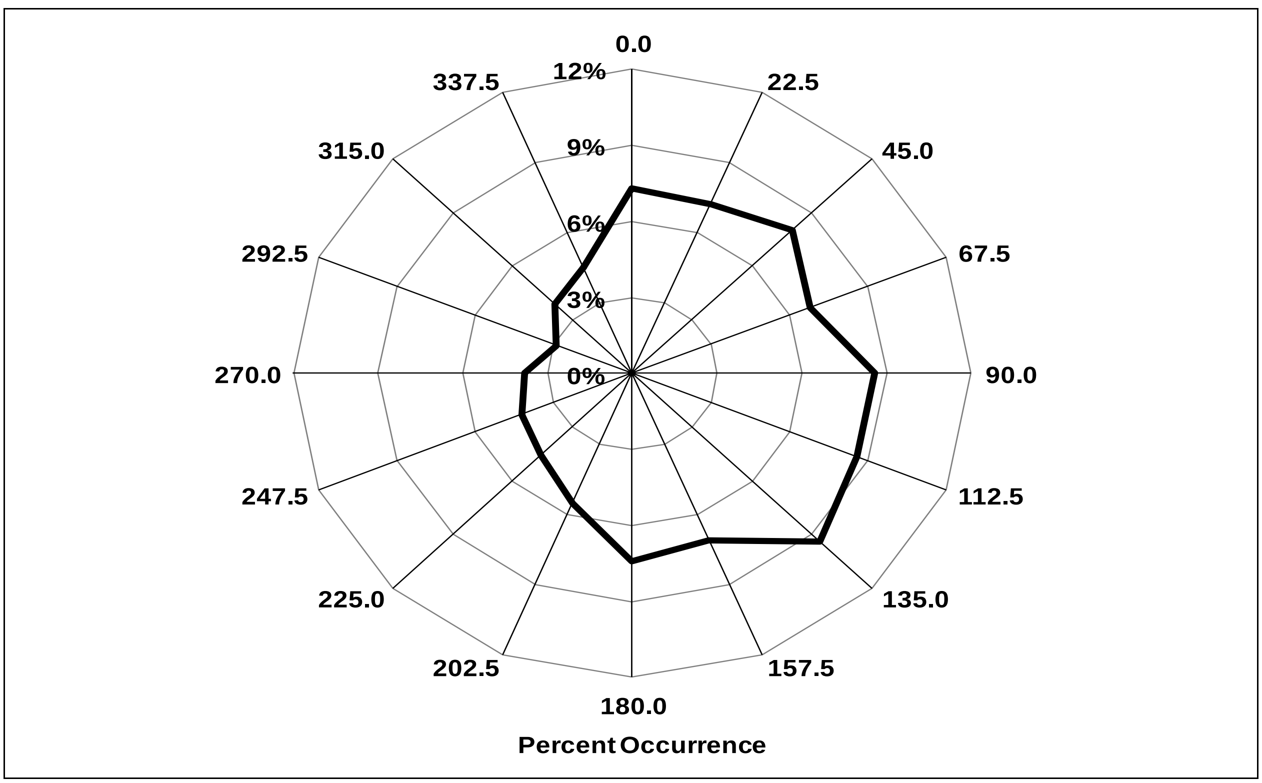


Figure 11. Directional Wind Statistics - Percent Occurrence

Winds under storm conditions appear in Table 10. These storm wind statistics account for extratropical storms and tropical storms but not hurricanes. Wind speed estimates from data collected at the Grand Isle tide station suggest that wind speed is expected to reach 36.5 miles/hour annually. Storm force winds can come from any direction but they typically come from the southeast.

Table 10. Grand Isle Storm Wind Statistics

Return Period	Wind Speed (mph)	
(yr)	Avg	Std Dev
1	36.5	3.0
2	42.0	4.5
3	45.2	5.5
4	47.6	6.2
5	49.5	6.8

The lack of hurricane wind speed data at the Grand Isle tide station disallowed the prediction of hurricane wind speed distributions using traditional extremal analysis theory. However, hurricane winds can be estimated indirectly from sources such as ship's logs, reports using the Beaufort wind scale (Jarvinen et al., 1988), and from statistical distributions of hurricane climatological characteristics using physical models of the hurricane wind speed field (USACE, 1985). Jagger and Elsner (2006) estimated extreme hurricane winds for the Gulf Coast of the United States using data from the best-track hurricane database (HURDAT), as shown in Table 11.

Table 11. Extreme Hurricane Winds for the Gulf Coast Region

Return Period	Wind Speed (mph)	
(yr)	Avg	Std Dev
5	120.8	6.5
10	141.5	7.0
50	172.6	7.0
100	181.8	7.6
500	195.6	9.4

8.4 Gulf Waves

Waves impacting the project area are generated primarily by local winds, though significant wave events may occur due to distant storms. However, the restricted fetch of the Gulf of Mexico basin limits the wave height and associated period during significant storm events compared to storms in the Atlantic or Pacific Oceans.

Wave statistics generated for the project area utilize the 1980-1999 hindcast at WIS Station 131 (USACE, 2003), the 2000-2010 Wavewatch III regional model data for the Western North Atlantic (NOAA, 2010), and the 2010 Wavewatch III global model data (NOAA, 2010). The resolution of the global Wavewatch III model is 30 minutes while the resolution of the regional Wavewatch III model is 10 minutes. Project area wave statistics describe the wave conditions at WIS Station 131, which is located 16.1 miles south of the project site in 66 feet of water.

The Wavewatch III data, interpolated to the location of WIS Station 131, were appended to the WIS hindcast such that the wave statistics generated for the project site include the major storms that occurred after 2000 (Gustav, Ike, Katrina, Rita, etc.). The average wave height at WIS Station 131 is 2.8 feet with a corresponding period and direction of 4.3 seconds and 151° (SSE) (Table 12). Approximately sixty-four percent (64%) of the waves propagate from the onshore direction band between 96° and 234°. Within this band, the average height is 2.9 feet with a corresponding period and direction of 4.7 seconds and 153° (SSE).

Table 12. Monthly Wave Statistics

Month	All Waves				Onshore Waves (96-234 deg)			
	Height (ft)		Period	Direction	Height (ft)		Period	Direction
	Avg	Max	(s)	(deg)	Avg	Max	(s)	(deg)
January	3.3	15.7	4.4	154	3.6	15.7	5.0	154
February	3.4	12.9	4.6	150	3.7	12.9	5.1	154
March	3.4	13.3	4.7	159	3.7	13.3	5.1	152
April	3.3	11.7	4.6	158	3.5	11.7	4.9	152
May	2.7	11.3	4.4	150	2.7	11.3	4.6	151
June	2.3	14.5	4.2	164	2.3	14.5	4.4	160
July	1.8	15.3	3.9	186	1.8	15.3	4.1	164
August	1.8	36.9	3.9	170	1.7	36.9	4.1	157
September	2.4	22.7	4.1	125	2.4	22.7	4.4	146
October	2.9	29.0	4.2	118	3.1	29.0	4.8	141
November	3.2	13.4	4.4	133	3.5	13.4	5.0	148
December	3.3	12.2	4.5	146	3.6	12.2	5.0	153
Average	2.8	36.9	4.3	151	2.9	36.9	4.7	153

The largest storm waves occur during hurricane season, between August and October. With the exception of tropical storm events, the highest waves under typical conditions occur between November and April, with the lowest waves occurring in July and August. The wave direction varies from 118° (ESE) in October to 186° (S) in July. Within the onshore direction band the wave direction varies from 141° (SE) in October to 164° (S) in July. The largest and longest waves under normal conditions come from the south-southeasterly direction band. Monthly wave statistics appear in Table 12, while directional wave statistics are shown in Figure 12 and Figure 13.

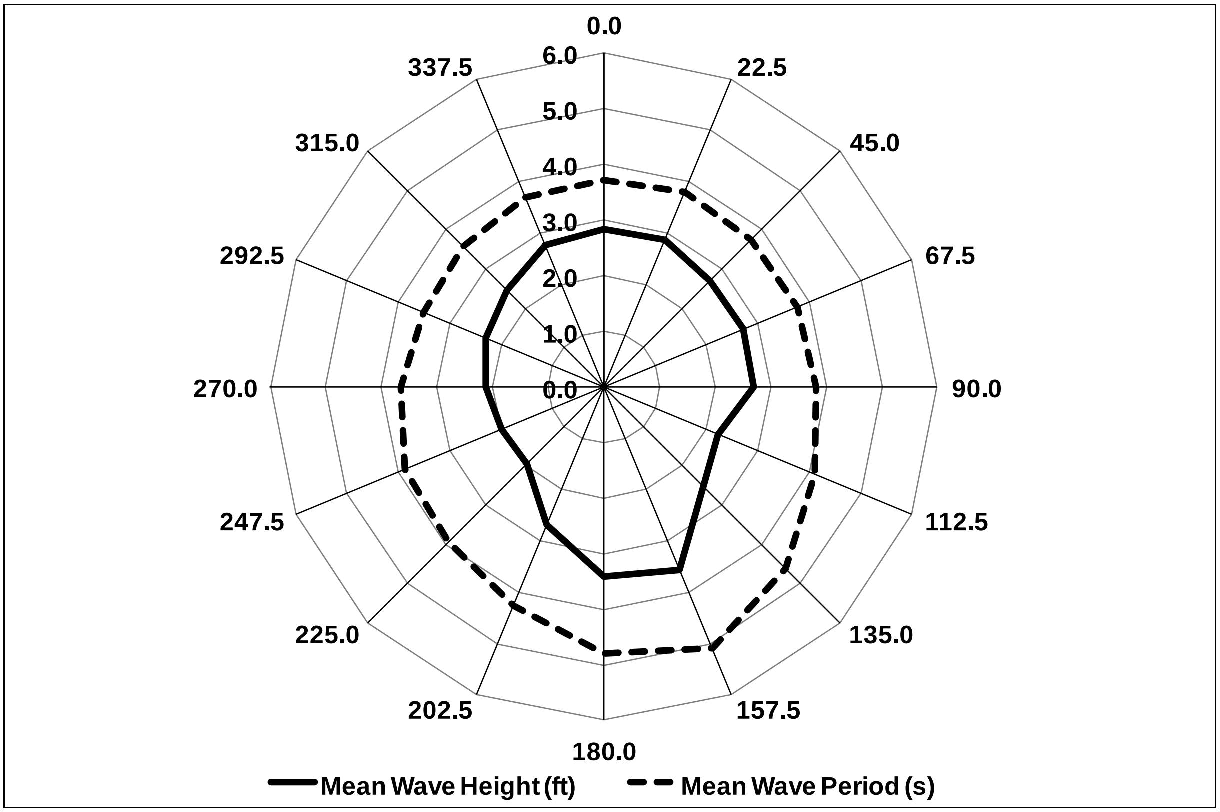


Figure 12. Directional Wave Statistics- Mean Wave Height and Period

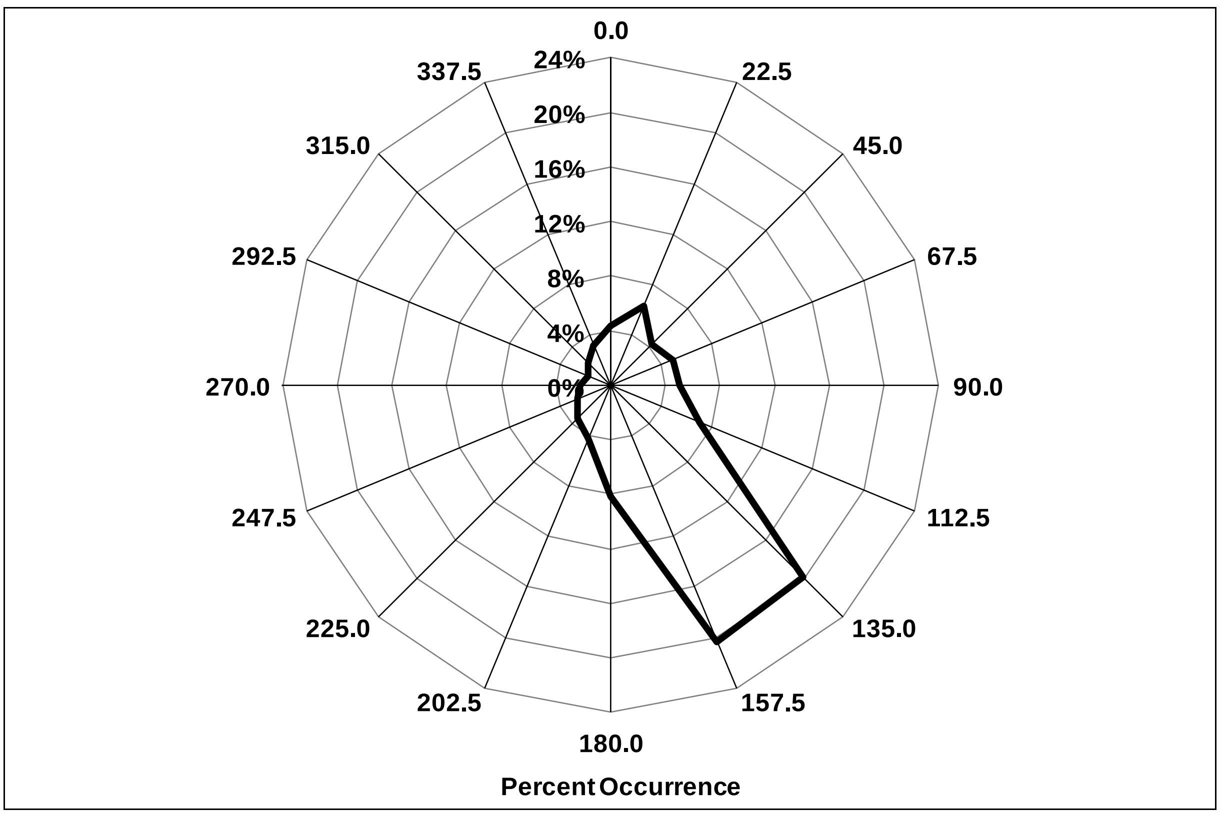


Figure 13. Directional Wave Statistics - Percent Occurrence

Waves under storm conditions are shown in Table 13 and Figure 14. These extremal wave statistics account for hurricanes, tropical storms, and extratropical storms. The wave heights are estimated based on combined WIS and Wavewatch III data. Offshore wave heights for the 1 and 25 year conditions range from 10.4 to 29.5 feet, with corresponding periods of 8.6 and 15.0 seconds. The top ten wave events used in the extremal wave analysis are shown in Table 14.

Table 13. Extremal Wave Statistics

Return Period	Wave Height (ft)		Wave Period (s)	
(yr)	Avg	Std Dev	Avg	Std Dev
1	10.4	1.9	8.6	0.36
2	14.2	2.9	10.0	0.56
3	16.5	3.6	10.8	0.74
4	18.2	4.1	11.3	0.87
5	19.5	4.5	11.8	0.98
10	23.7	5.7	13.2	1.33
15	26.2	6.5	14.0	1.53
20	28.0	7.0	14.5	1.68
25	29.5	7.5	15.0	1.80

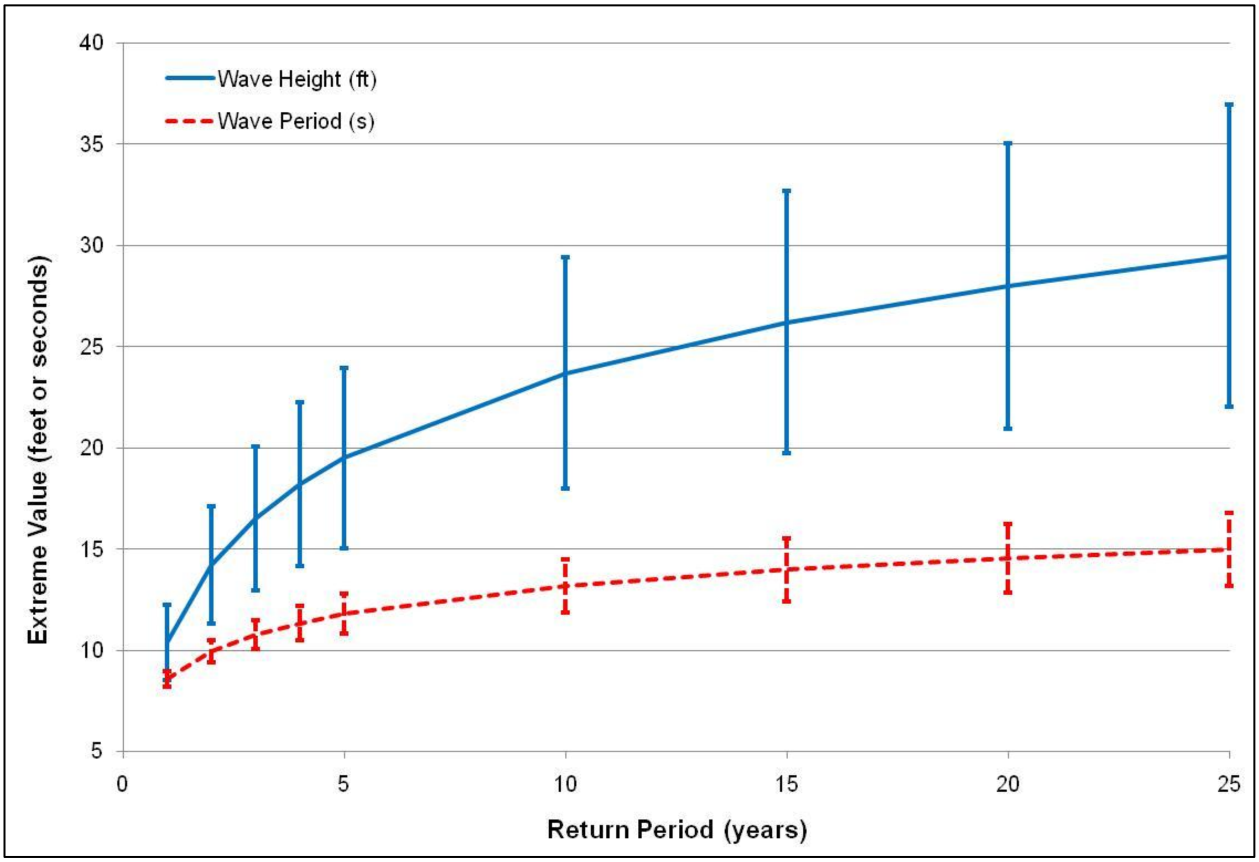


Figure 14. Extremal Wave Statistics

Table 14. Extreme Wave Events

Event Date	Event Name	Height (ft)	Period (s)
August 29, 2005	Katrina	36.9	15.0
October 28, 1985	Juan	29.0	12.5
October 3, 2002	Lili	25.1	11.4
September 23, 2005	Rita	22.7	11.6
September 12, 2008	Ike	20.4	13.3
August 26, 1992	Andrew	18.8	11.1
September 15, 2004	Ivan	18.6	15.4
September 25, 2002	Isidore	16.9	11.4
August 15, 1985	Danny	16.7	10.0
September 10, 1988	Florence	15.9	12.5

8.5 Bay Waves

A fetch limited wave analysis was used to estimate wave activity along the back-bay shoreline during average conditions and frequent storms. An average depth of -6.0 feet, NAVD was used for Bay Long based on data collected by JCLS in September 2010. Wave height and period were estimated using the fetch limited wave equations presented in the Coastal Engineering Manual (USACE, 2001). Under average conditions back-bay wave heights range from 0.3 to 1.0 feet with corresponding periods of 1.0 and 2.1 seconds; the average back-bay wave height is 0.6 feet with a corresponding period of 1.5 seconds. Estimated back-bay wave heights during typical conditions are shown in Table 15 and Figure 15.

Table 15. Fetch Limited Directional Wave Statistics

Wind Direction Band (deg)	Fetch (ft)	Average Wind Speed (mph)	Percent Occurrence	Wave Height (ft)	Wave Period (s)
0.0	31,111	14.2	7.3%	1.0	2.1
22.5	27,503	14.3	7.3%	0.9	2.0
45.0	3,972	12.5	8.0%	0.3	1.0
67.5	-	11.4	6.8%	-	-
90.0	-	10.8	8.6%	-	-
112.5	-	10.3	8.6%	-	-
135.0	-	9.4	9.4%	-	-
157.5	-	9.2	7.1%	-	-
180.0	-	9.1	7.4%	-	-
202.5	-	8.9	5.6%	-	-
225.0	-	8.3	4.6%	-	-
247.5	-	10.4	4.2%	-	-
270.0	7,853	11.1	3.8%	0.4	1.2
292.5	8,330	11.5	2.9%	0.4	1.3
315.0	7,571	11.9	3.9%	0.4	1.2
337.5	7,696	14.1	4.5%	0.5	1.3
Average	15,303	13.1	-	0.6	1.5

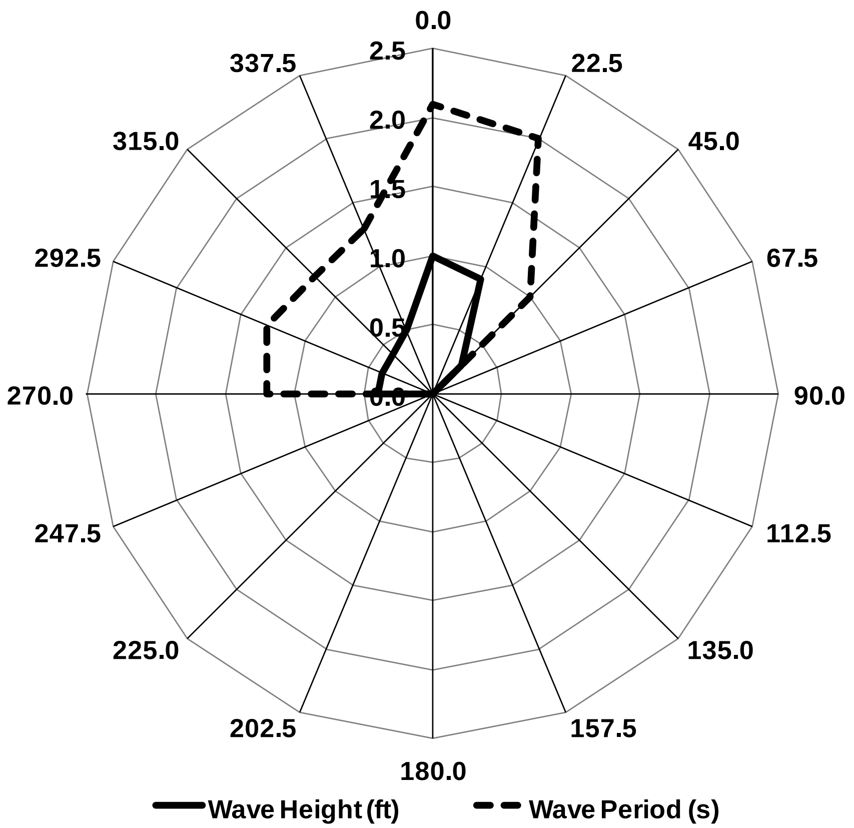


Figure 15. Fetch Limited Directional Wave Statistics - Mean Wave Height and Period

As the average wave energy in the back-bay area is small, wave induced erosion is attributed to storm events. A wave hindcast was performed for annual to 5-year return period storm events. A water level equal to the storm stage was used for the fetch limited waves during the 3, 4, and 5-year return period storm conditions. During the more frequent storms the water level was assumed to be equal to Mean Higher High Water (MHHW). The wind was assumed to blow from the North (0°) as this is the longest fetch. Under the annual to 5-year storm conditions wave heights range from 2.8 to 4.0 feet with corresponding periods of 3.0 and 3.4 seconds. A summary of the back-bay wave climate during storm events is provided in Table 16.

Table 16. Fetch Limited Extremal Wave Statistics

Return Period (yr)	Average Wind Speed (mph)	Wave Height (ft)	Wave Period (s)
1	36.5	2.8	3.0
2	42.0	3.3	3.2
3	45.2	3.6	3.3
4	47.6	3.9	3.4
5	49.5	4.0	3.4

8.6 Storm Surge

Storm surge is defined as the rise of the sea surface above its astronomical tide level due to storm forces. The elevation that the storm surge reaches is known as the storm stage. This increased elevation is attributable to a variety of factors, including wave set-up, wind shear stress, and atmospheric pressure. An estimate of these water level changes is essential in the development of a coastal restoration project. Increased water depth increases the potential for shoreline recession, long-term erosion, and overtopping from severe waves. Storm surge estimates for the project area are based on the observed water levels at Grand Isle between 1981 and 2010. Storm surge estimates, which include the effects of wind setup, barometric pressure, and astronomical tides, appear in Table 17 and Figure 16. Wave set-up is not included in this analysis as the Grand Isle tide gauge is located on the bay side of the island.

Table 17. Extremal Storm Surge Statistics

Return Period	Storm Surge (ft)	
(yr)	Avg	Std Dev
1	1.4	0.2
2	2.0	0.2
3	2.4	0.3
4	2.6	0.4
5	2.8	0.4
10	3.4	0.6
15	3.8	0.7
20	4.0	0.7
25	4.2	0.8

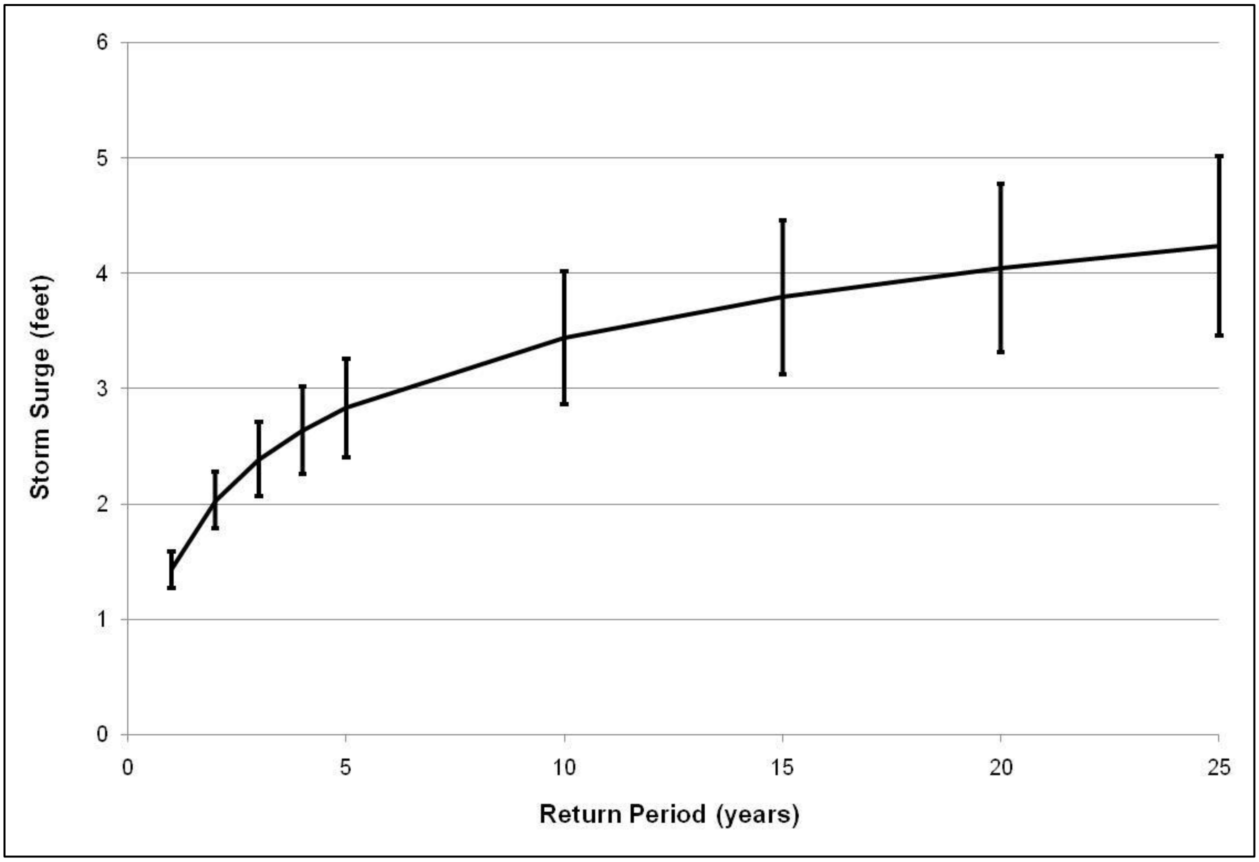


Figure 16. Extremal Storm Surge Statistics

An extremal surge analysis at Grand Isle was used to provide extreme surge values expected for the project area. Table 18 illustrates the top ten storm surge events measured at Grand Isle. The storm surge measured during these extreme events range from 4.4 feet, measured during Hurricane Gustav, to 2.5 feet, measured during Hurricane Cindy.

Table 18. Grand Isle Extremal Storm Surge Events

Event Date	Event Name	Storm Surge (ft)
September 1, 2008	Gustav	4.4
August 29, 2005	Katrina	4.2
September 12, 2008	Ike	3.7
September 23, 2005	Rita	3.3
October 28, 1985	Juan	3.2
August 26, 1992	Andrew	3.1
July 18, 1997	Danny	3.1
September 26, 2002	Isidore	3.0
October 3, 2002	Lili	2.7
July 6, 2005	Cindy	2.5

9 SUBSIDENCE AND SEA LEVEL RISE

A primary factor governing land loss along the Louisiana coast is relative sea level rise. Relative sea level rise consists of two components:

1. Eustatic Sea Level Change. Eustatic sea level change is defined as the global change in oceanic water level relative to a fixed vertical datum.

2. Subsidence. Subsidence is defined as the local change in land elevation relative to a fixed vertical datum.

Along the Louisiana coast the land elevation is decreasing while the mean sea level elevation is increasing, resulting in significant land loss. Estimates of eustatic sea level rise and subsidence appear in a number of sources, including NOAA (2010), Penland et al. (1989), the Intergovernmental Panel on Climate Change (2007), and the National Research Council (1987).

9.1 Relative Sea Level Rise

Tide data has been collected at the Grand Isle, LA tide gauge since 1947. The published data is a combination of data collected from two tide gauges, Bayou Rigaud and East Point, which are located about 0.9 miles apart along the northwest shore of Grand Isle. The zero-marks on these two gauges, and thus their data sets, were connected by leveling observations to form a continuous water level data set. NOAA calculated the rate of relative sea level rise at Grand Isle using monthly means of tide data collected between 1947 and 2006. According to NOAA, the sea level at Grand Isle is increasing at a rate of 0.0303 feet/year (9.24 mm/yr). Figure 17 shows this trend and the monthly mean sea level data.

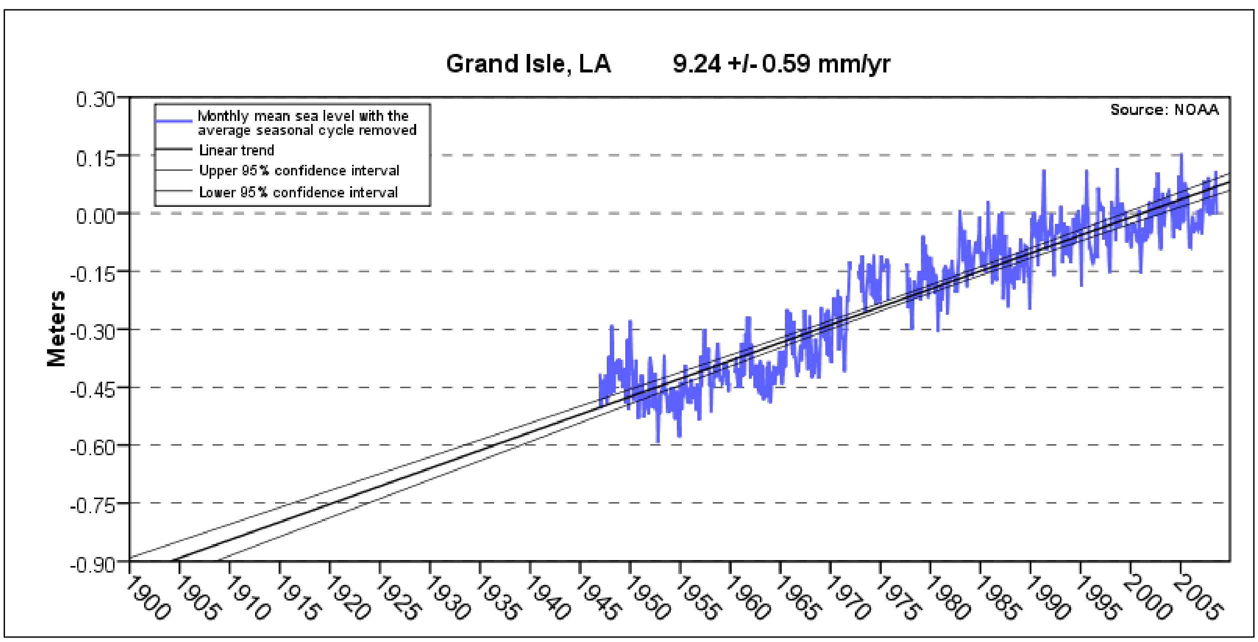


Figure 17. Sea Level Rise at Grand Isle

Penland et al. (1989) analyzed the rate of relative sea level rise using tide data collected at the Grand Isle tide gauge between 1947 and 1978. The trend observed throughout the entire record indicates that sea level is rising at a relative rate of approximately 0.0338 feet/year (10.3 mm/yr). However, the data shown in Figure 17 indicates that the rate of relative sea level rise changed during this time period. Between 1947 and 1962 the rate of relative sea level rise was approximately 0.0098 feet/year (3.0 mm/yr) while the rate of relative sea level rise was approximately 0.0630 feet/year (19.2 mm/yr) between 1962 and 1978 (Figure 18).

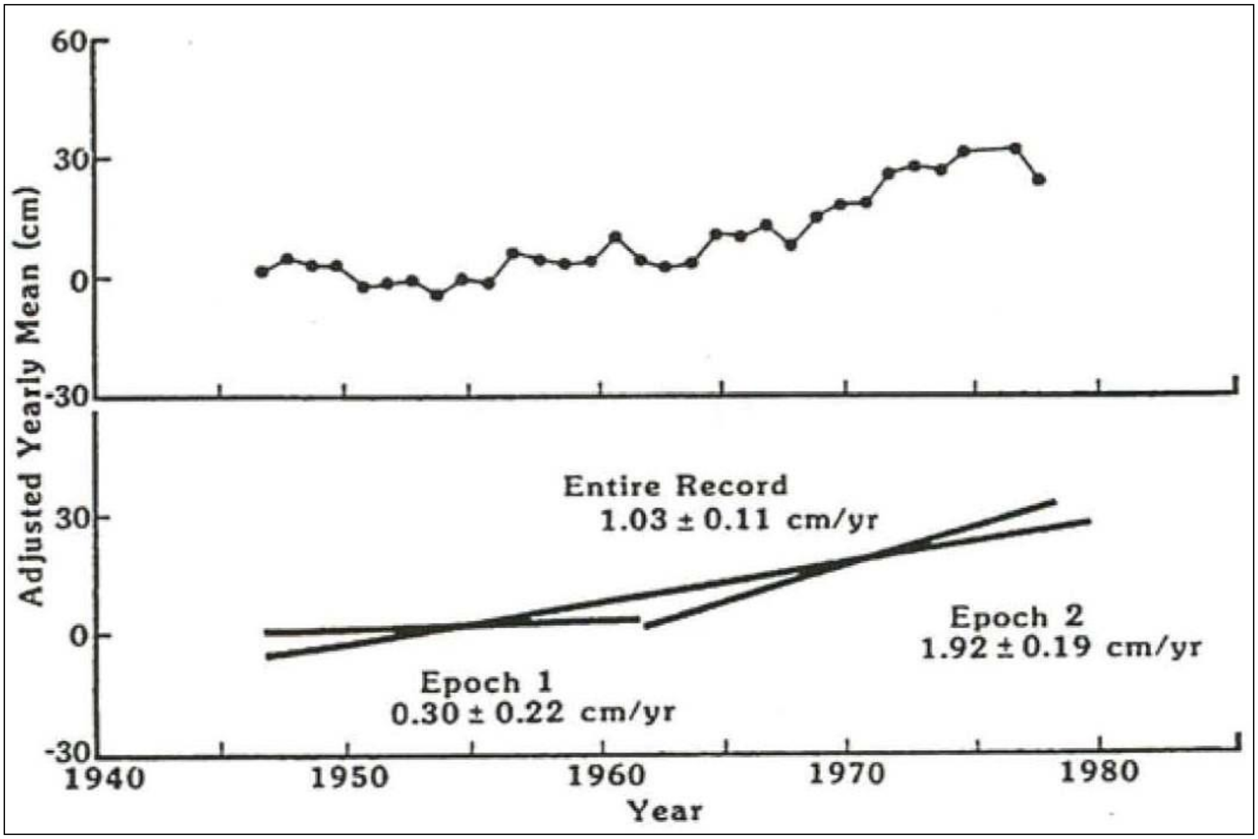


Figure 18. Analysis of Grand Isle Relative Sea Level Rise (Penland et al., 1989)

The rate of relative sea level rise was also calculated independently using tide data collected at the Grand Isle tide gauge between 1980 and 2010. The average rate of relative sea level rise was approximated by performing a first-order linear regression of the daily high/low tide data. This analysis suggests that between 1980 and 2010 the sea level increased at an average rate of approximately 0.0227 feet/year (6.9 mm/yr). Figure 19 shows the measured data and the results of the relative sea level rise analysis. This same analysis was completed using the monthly Mean Sea Level values provided by NOAA over the same analysis period and yielded a relative sea level rise rate of 0.0220 feet/year (6.2 mm/yr) shown in Figure 20, which differs by approximately 3.1% from the value calculated using daily high and low tide data.

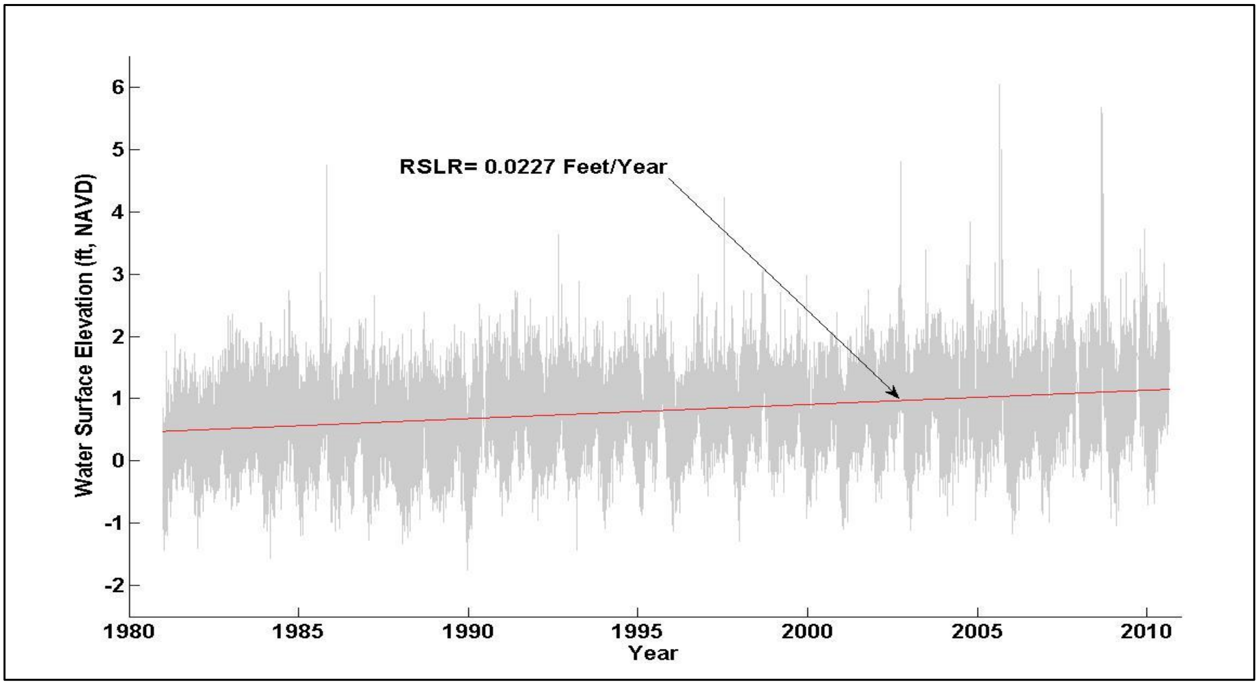


Figure 19. Grand Isle Relative Sea Level Rise (Daily High/Low Water Levels)

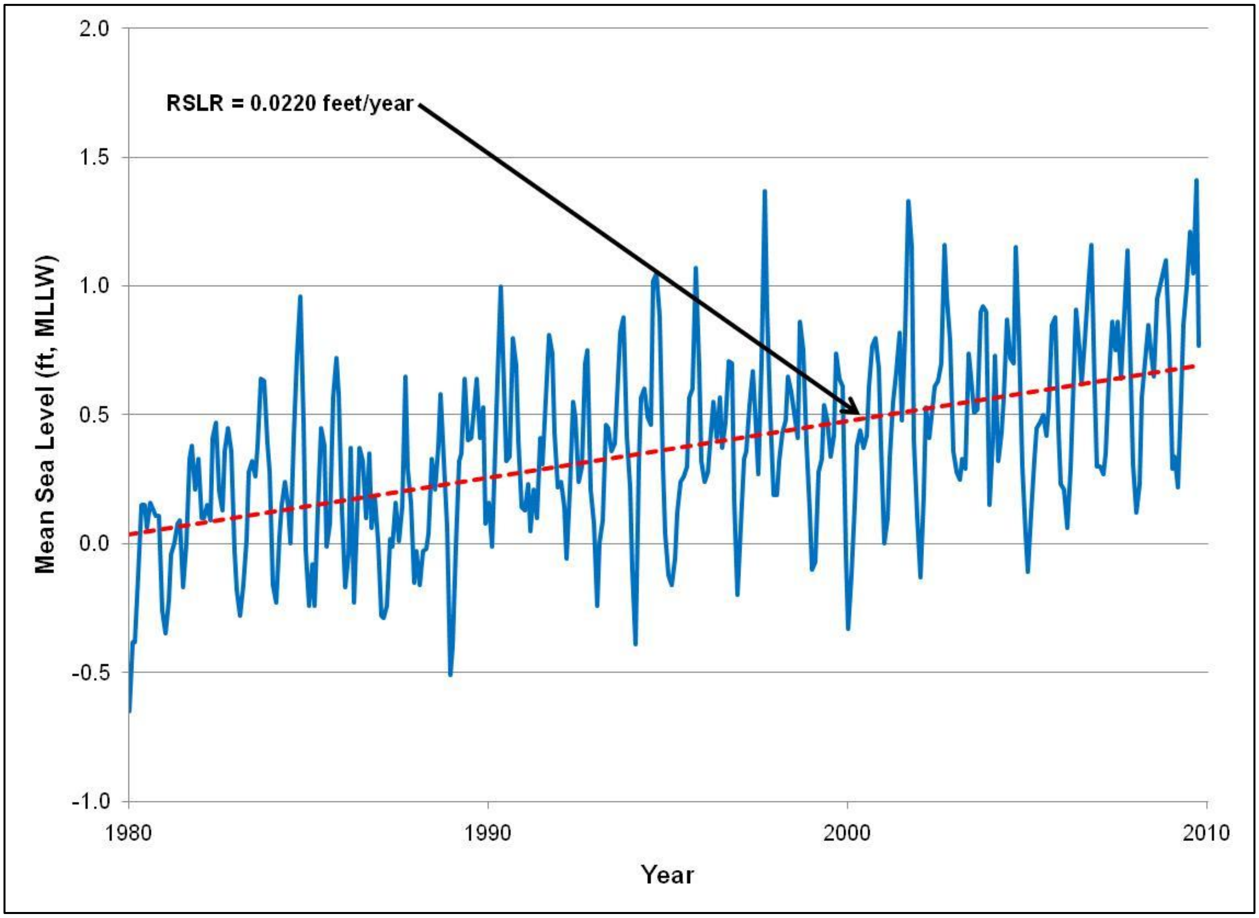


Figure 20. Grand Isle Relative Sea Level Rise (Monthly Mean Sea Level)

9.2 Eustatic Sea Level Rise

The eustatic sea level rise rate recommended by the USACE was reported by the Intergovernmental Panel on Climate Change (IPCC). The IPCC expressed the global average sea level rise, Δh, using Equation 1:

$$\Delta h(t) = X(t) + g(t) + G(t) + A(t) + I(t) + p(t) + s(t) \qquad \text{[Equation 1]}$$

where:

X = thermal expansion
g = loss of mass of glaciers and ice caps
G = loss of the Greenland ice sheet due to projected and recent climate change
A = loss of mass of the Antarctic ice sheet due to projected and recent climate change
I = loss of mass of the Greenland and Antarctic ice sheets due to the ongoing adjustment to past climate change
p = runoff from thawing of permafrost
s = deposition of sediment on the ocean floor
t = time

The IPCC used emissions scenarios and a range of values obtained from different numerical models to develop ranges of future sea level rise values. Seven Atmosphere-Ocean General Circulation Models (AOGCMs) were calibrated to project the components X, g, G, and A based on varying influencing factors such as climate sensitivity, heat uptake, and different emissions scenarios. The components I, p, and s were assumed to contribute to sea level rise at a constant rate independent of the emissions scenarios and AOGCMs. Currently, the IPCC (2007) concludes that the global mean sea level is rising at an average rate of about 0.0056 ± 0.0016 feet/year (1.7 ± 0.5 mm/year). Figure 21 depicts the range of global sea level rise projections.

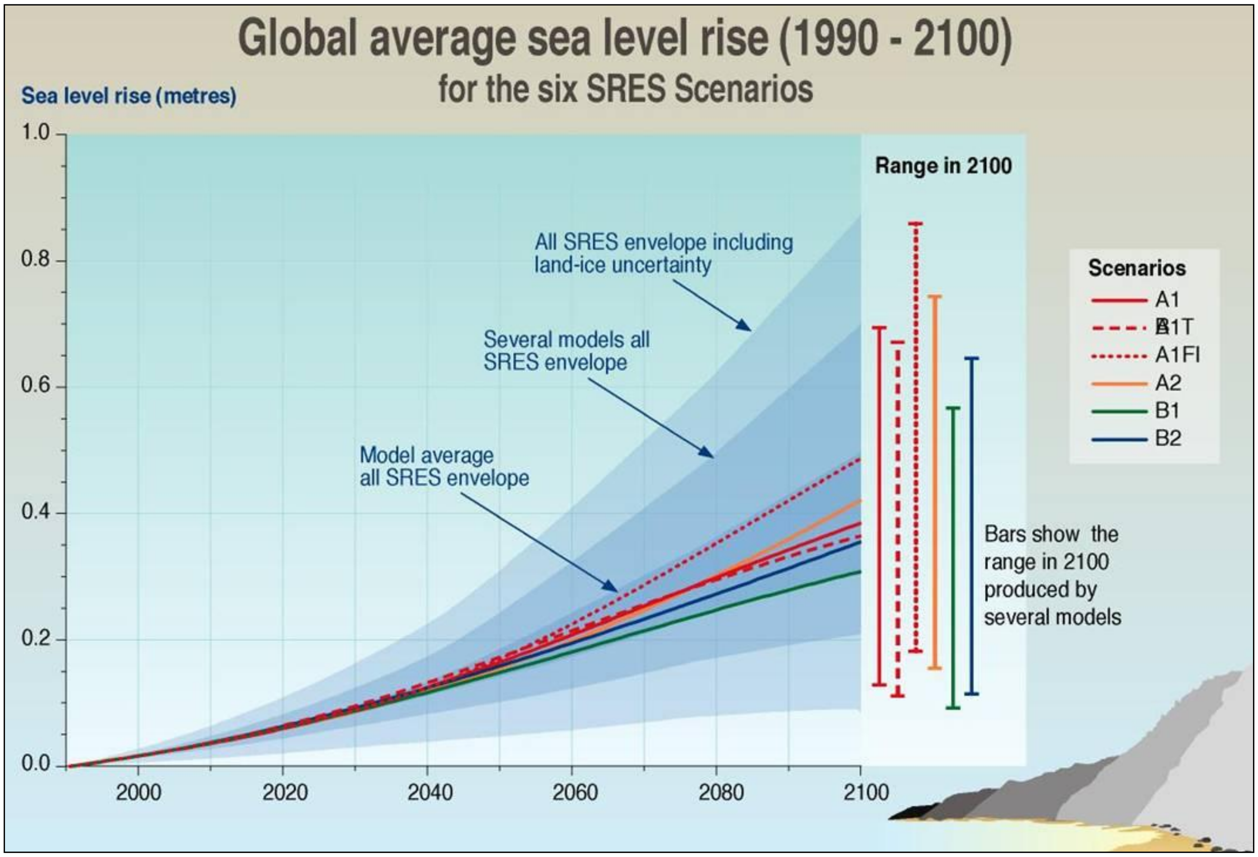


Figure 21. Global Average Sea Level Rise Projections

9.3 Sea Level Rise Projections

The USACE issued the circular EC 1165-2-211 *Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs* that provides guidance for incorporating the direct and indirect effects of projected future sea level change in USACE projects (USACE, 2009a). Per USACE guidance, the National Research Council (NRC) methodology should be used to predict future sea level changes. The NRC recommends that feasibility studies for coastal projects consider the high probability of accelerating global sea level rise and provided three different accelerating eustatic sea level rise scenarios that result in global eustatic sea level rise values, by the year 2100, of 1.64 feet (0.5 meters), 3.28 feet (1.0 meters), and 4.92 feet (1.5 meters).

Relative sea level rise projections can be estimated using tidal records and the NRC methodology. For a tidal record to be sufficient for this purpose it must be at least 37 years long, which is twice the length of the 18.6 year lunar nodal tide cycle. The numerical relationship employed for estimating total relative sea level rise is shown in Equation 2.

$$T(t) = (0.0056 + M)t + bt^2 \quad \text{[Equation 2]}$$

where:

T = relative sea level change between 1986 and year t in feet
M = subsidence in feet/year
b = acceleration in the rate of eustatic sea level change in feet/year^2

The NRC methodology was used to estimate relative sea level rise throughout the project life. Three scenarios (baseline, intermediate, and high) were analyzed to provide a range of future sea level rise projections for the project area. The baseline scenario assumes that the sea level will continue to rise at its current rate throughout the project life. The intermediate scenario employs the modified NRC Curve I, or the sea level rise acceleration factor that yields a sea level change of 1.6 feet by the year 2100 (b = 0.000077 feet/year^2), to project future sea level rise. The high scenario employs the modified NRC Curve III, or sea level rise acceleration factor that yields a sea level change of 4.9 feet by the year 2100 (b = 0.000330 feet/year^2), to project future sea level rise. The subsidence rate of 0.0247 feet/year (7.54 mm/year), calculated by taking the difference between the NOAA relative sea level rise rate of 0.0303 feet/year (9.24 mm/year) and the IPCC eustatic sea level rise rate of 0.0056 feet/year (1.70 mm/year). This was assumed to remain constant over the project life.

9.4 Subsidence

Subsidence, also known as vertical land movement, was estimated according to the USACE EC 1165-2-211 circular. The rate of eustatic mean sea level rise (approximately 0.00056 ft/yr according to IPCC) was subtracted from the historical rate of relative sea level rise (approximately 0.0303 ft/year according to NOAA) established at Grand Isle, Louisiana. Subsidence for the project area was estimated at 0.0247 ft/year (approximately 0.49 feet over the 20-year project life) as shown in Table 19. The rate of subsidence was held constant for each of the three relative sea level rise scenarios as it is the rate of the water level that is projected to increase and not the rate of vertical land movement.

9.5 Relative Sea Level Rise Summary

Over the 20-year project life, the rate of sea level rise is expected to increase. However, the low acceleration rate of sea level rise allows the rate of sea level rise to be treated as a constant. The average rate of sea level rise was calculated by dividing the total sea level rise experienced over the project life by the project life. A summary of the various sea level rise rates is provided in Table 19 and shown in Figure 22.

Table 19. Relative Sea Level Rise Summary

	Total Sea Level Change (feet)			Sea Level Change Rate (feet/year)		
Rate Type	**Eustatic**	**Subsidence**	**Relative**	**Eustatic**	**Subsidence**	**Relative**
Baseline	0.11	0.49	0.61	0.0056	0.0247	0.0303
Intermediate	0.23	0.49	0.72	0.0114	0.0247	0.0362
High	0.61	0.49	1.11	0.0306	0.0247	0.0554

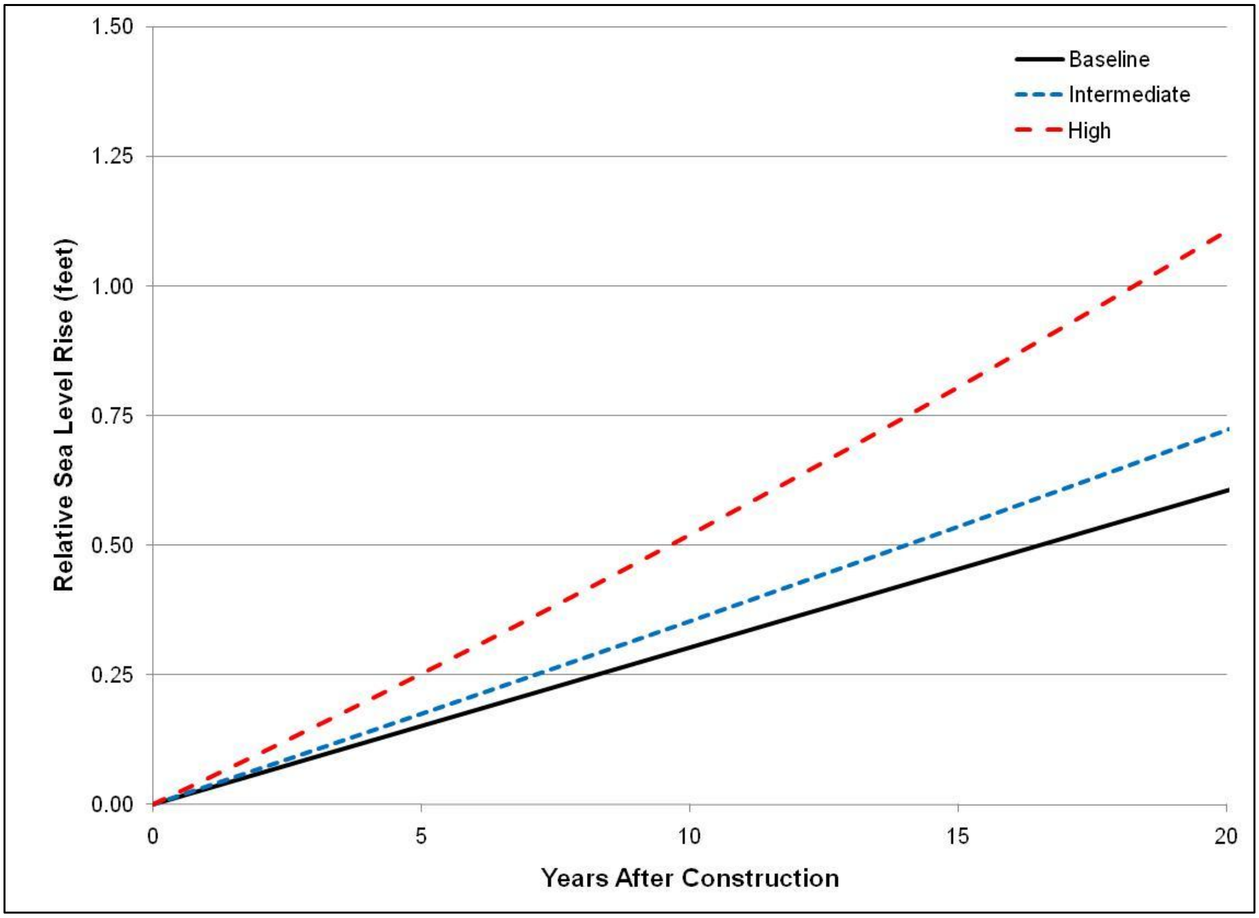


Figure 22. Local Relative Sea Level Rise Projections

10 SHORELINE CHANGES

Chenier Ronquille contains perched beaches of sand and shell with backshore washover terraces and flats located between vegetated dredge spoil banks. The spoil banks were created by dredging pipeline right-of-ways and access canals, which are found along the length of the island. Shoreline recession, erosion, and overwash have resulted in the disintegration of Chenier Ronquille. The long-term shoreline erosion rate, between 1884 and 2002, is 19.7 feet/year, while the short-term shoreline erosion rate, between 1988 and 2002, is 17.2 feet/year (Penland, Conor, and Beall, LCA 2004).

10.1 Historic Shoreline Changes

Historic shoreline changes were obtained from the *Louisiana Barrier Island Erosion Study – Atlas of Shoreline Changes in Louisiana From 1853 to 1989* (Williams et al., 1992). The locations of the USGS transects used in the historic shoreline change analysis are presented graphically in Figure 23. Gulfside shoreline changes presented in Louisiana Barrier Island Erosion Study were annualized for comparison purposes and are presented in Figure 24 and Table 20. A negative Gulf shoreline change indicates shoreline recession and movement to the north while a positive Gulf shoreline change indicates shoreline accretion and movement to the south.

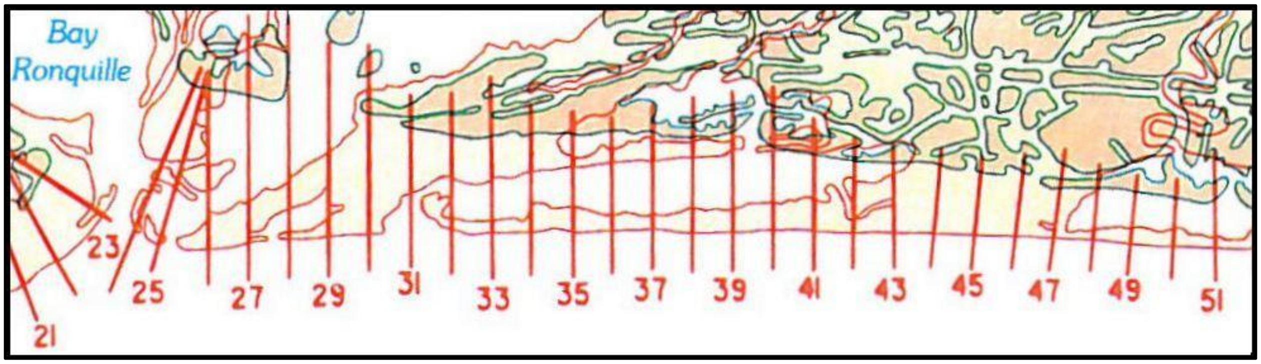


Figure 23. USGS Shoreline Change Transects

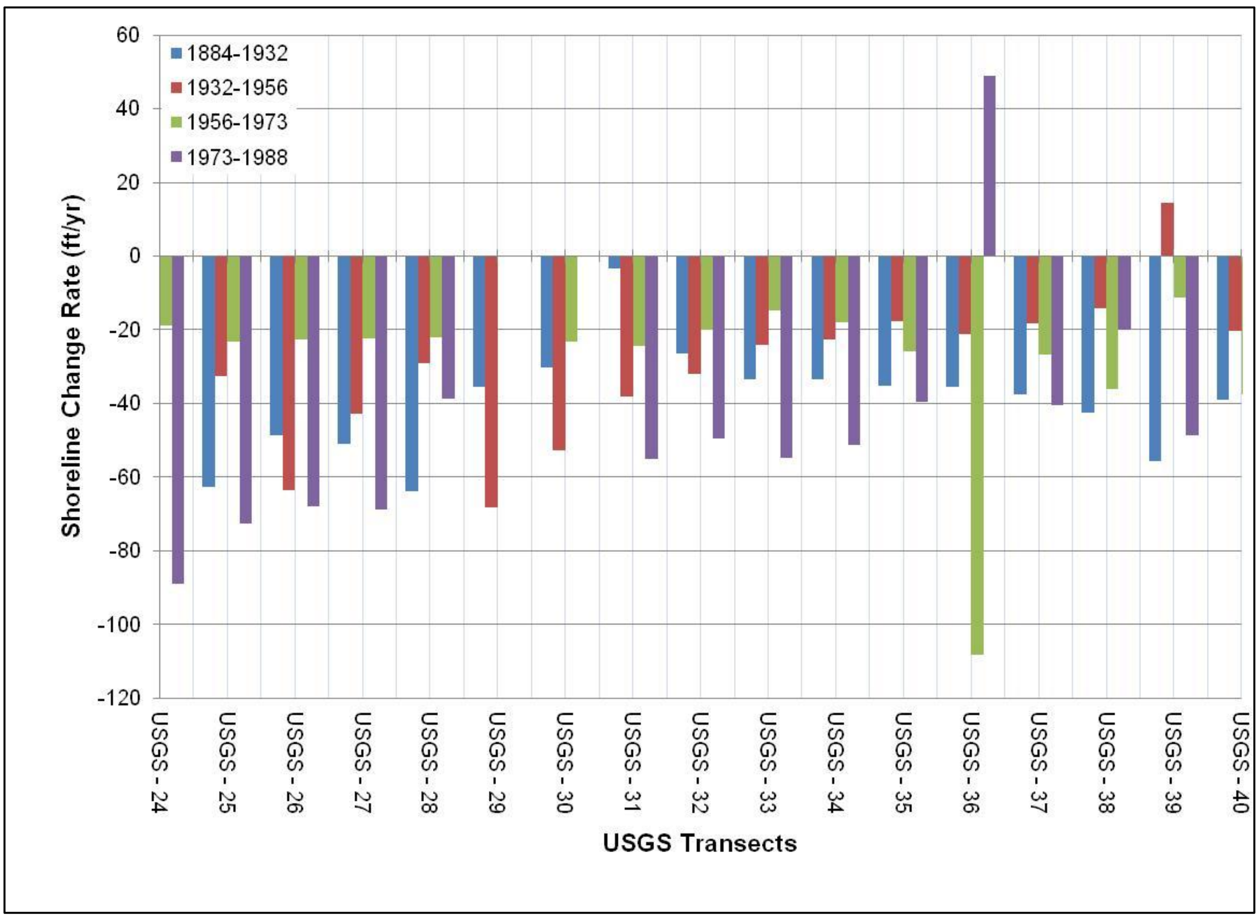


Figure 24. Historic Shoreline Change Rates

Table 20. Historic Shoreline Change Rates

	Annual Shoreline Change (ft/yr)				
Transect	**1884-1932**	**1932-1956**	**1956-1973**	**1973-1988**	**1884-1988**
USGS - 24	-62.0	-28.4	-18.9	-89.0	-51.1
USGS - 25	-62.7	-32.5	-23.2	-72.6	-50.7
USGS - 26	-48.7	-63.6	-22.8	-67.8	-50.6
USGS - 27	-51.1	-42.7	-22.4	-68.9	-47.0
USGS - 28	-63.8	-29.1	-22.0	-38.7	-45.3
USGS - 29	-35.5	-68.1	-	-	-
USGS - 30	-30.3	-52.8	-23.2	-	-
USGS - 31	-3.3	-38.0	-24.5	-55.1	-22.2
USGS - 32	-26.5	-32.0	-19.9	-49.4	-30.0
USGS - 33	-33.4	-24.2	-14.9	-54.9	-31.4
USGS - 34	-33.4	-22.7	-17.9	-51.2	-30.9
USGS - 35	-35.1	-17.6	-25.9	-39.6	-30.2
USGS - 36	-35.4	-21.1	-108.3	49.0	-31.8
USGS - 37	-37.5	-18.3	-26.6	-40.5	-31.7
USGS - 38	-42.6	-14.1	-36.1	-20.1	-31.7
USGS - 39	-55.5	14.4	-11.2	-48.8	-31.2
USGS - 40	-39.1	-20.2	-37.6	-5.2	-29.6
Average	-40.9	-30.1	-28.5	-43.5	-36.4

10.1.1 Shoreline Change between 1884 and 1932

Over the recorded life of Chenier Ronquille, the greatest shoreline recession rates occurred between 1884 and 1932. Rapid recession rates during this time period are attributed to the emergence of Pass La Mer and the widening of Pass Ronquille. The recession rate along the project area was moderately uniform and averaged 40.9 feet/year. The largest recession rates occurred near the opening of Pass Ronquille, between USGS transects 24 and 28, which receded at an average rate of 57.6 feet/year. Figure 25 shows the land loss/gain between 1884 and 1932.

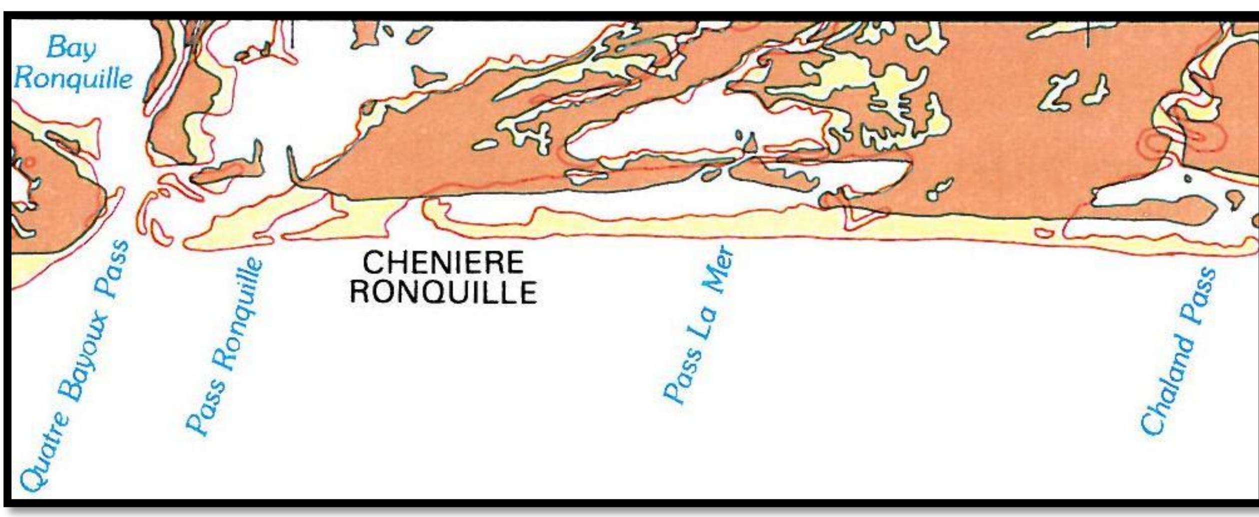


Figure 25. Shoreline Change between 1884 (yellow) and 1932 (brown)

10.1.2 Shoreline Change between 1932 and 1956

Gulf shoreline recession rates reduced to 30.1 feet/year between 1932 and 1956 compared to the 40.9 feet/year recession rate from the 1884 to 1932 analysis period (Figure 26). The largest recession rate occurred near USGS transect 29, which receded at a rate of 68.1 feet/year, while the shoreline accreted 14.4 feet/year along USGS transect 39. The accretion was a result of the eastward migration of Pass La Mer, which is no longer located at USGS transect 39.

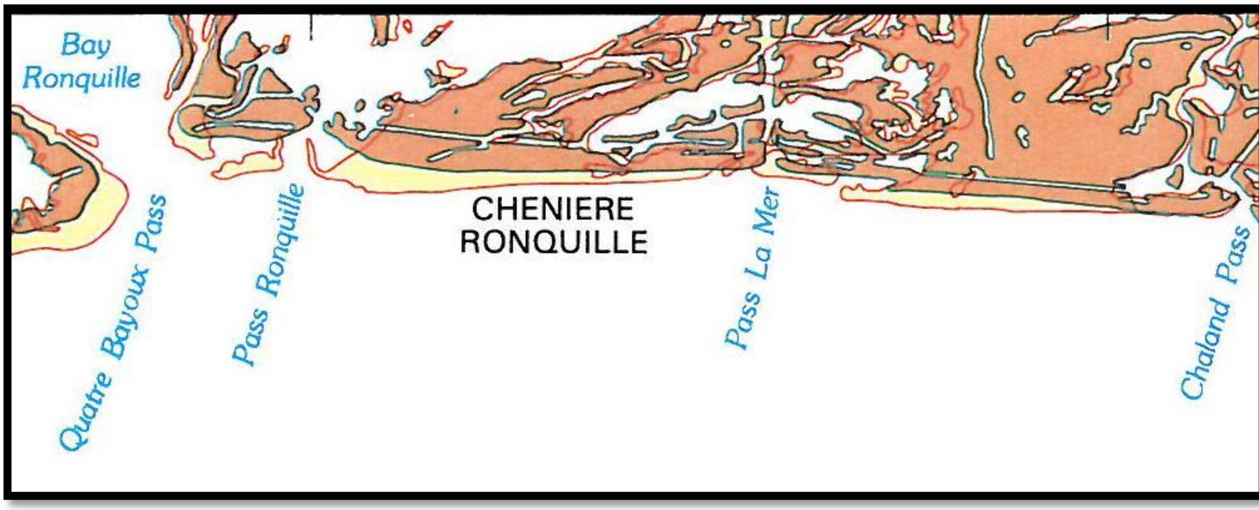


Figure 26. Shoreline Change between 1932 (yellow) and 1956 (brown)

10.1.3 Shoreline Change between 1956 and 1973

The smallest shoreline recession rates experienced over the recorded life of Chenier Ronquille occurred between 1956 and 1973 (Figure 27). The shoreline receded at a rate of 108.3 feet/year along USGS transect 36, which was the largest recession rate over the recorded life of Chenier Ronquille. This extreme recession along USGS transect 36 was a result of the shoreline breaching at this location. However, the rest of the barrier island receded at a relatively uniform rate of 23.1 feet/year.

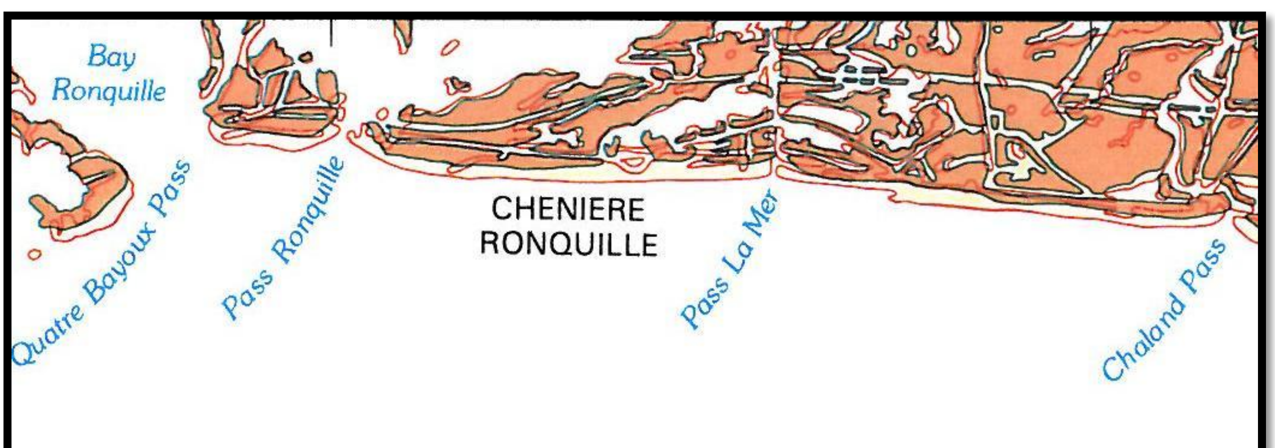


Figure 27. Shoreline Change between 1956 (yellow) and 1973 (brown)

10.1.4 Shoreline Change between 1973 and 1988

The average Gulf shoreline recession rate increased to 43.5 feet/year between 1973 and 1988 compared to the 23.1 feet/year recession rate from the 1956 to 1973 analysis period (Figure 28). The breach along USGS transect 36 closed, which accounts for the 49.0 feet/year accretion rate recorded at USGS transect 36. Furthermore, the loss of tidal flow through the breach at USGS transect 36 resulted in Pass Ronquille widening from 800 feet to 3,900 feet.

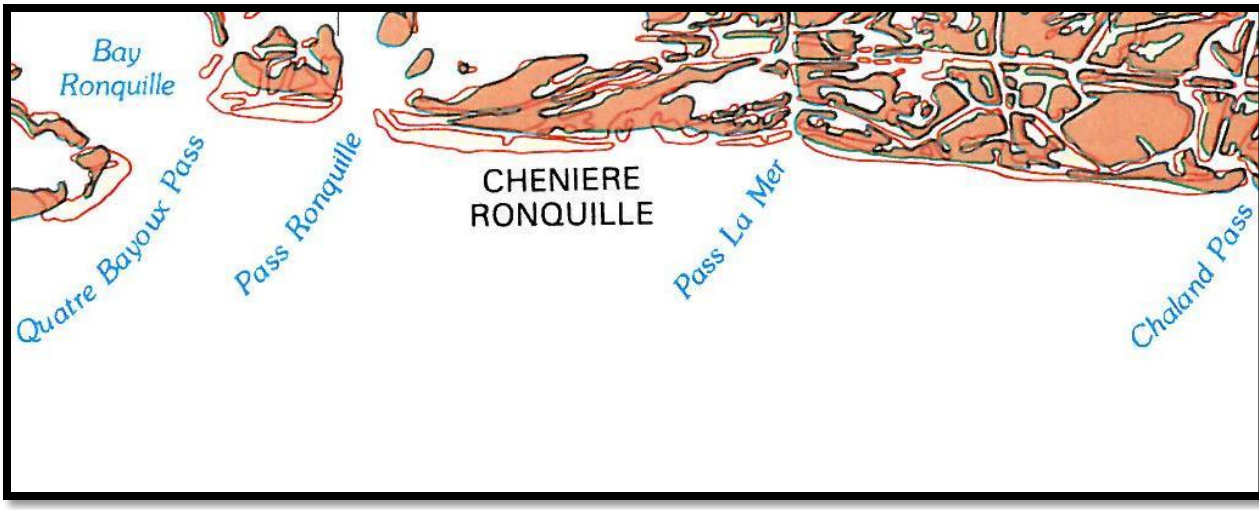


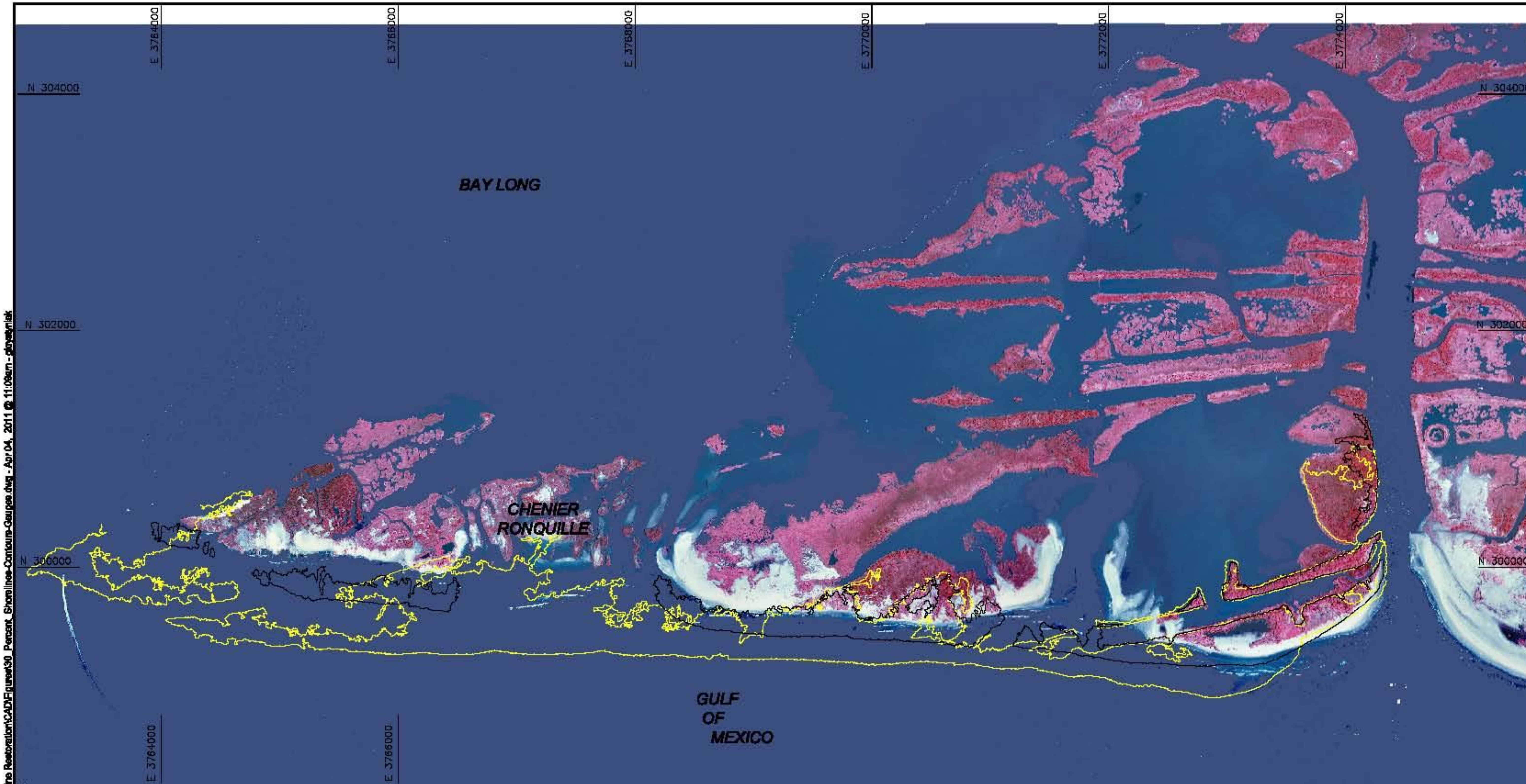
Figure 28. Shoreline Change between 1973 (yellow) and 1988 (brown)

10.2 Recent Shoreline Changes

John Chance Land Surveyors performed a survey of the project area in September 2010 (Section 6.2). This survey coupled with the rectified aerial photograph collected on August 25, 2010 allowed for the digitization of the 2010 mean high water shoreline. Previous shoreline locations were developed by contouring LIDAR data collected in 1998 and 2006. Given the orientation of the shoreline between 1998 and 2010, the orientation of transects along which shoreline changes were measured was set to a north-south azimuth (180º); transects were spaced at 500-foot intervals starting at the western extent of the island, as shown in Figure 29. Shoreline discontinuities represent breaches in the island where shoreline changes were not calculated.

10.2.1 Gulf Shoreline Changes

Gulf shoreline changes were calculated along project transects by measuring the distance between the mean high water Gulf shorelines. Analysis of Gulf shoreline changes suggests that the west end of the island is receding faster than the east end of the island. Table 21 summarizes the Gulf shoreline changes measured along each transect line. It should be noted that positive Gulf shoreline changes indicate shoreline advance while negative Gulf shoreline changes indicate shoreline retreat. The average shoreline retreat rate measured over the length of the island varies between 32.0 feet/year, as measured between 1998 and 2006, and 58.4 feet/year, as measured between 2006 and 2010.



N 298000

LEGEND:

DECEMBER 2006 MHW (1.3 FT, NAVD)
NOVEMBER 1998 MHW (1.3 FT, NAVD)

NOTES:

1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
2. DATE OF PROJECT AREA PHOTOGRAPH: AUGUST 2010.

N

0 400 800
GRAPHIC SCALE IN FT

E 3770000

Figure 29. MHW Shoreline Location Map

COASTAL PLANNING & ENGINEERING, INC.
2481 N.W. BOCA RATON BOULEVARD
BOCA RATON, FLORIDA 33431
PH. (561) 391-8102
FAX (561) 391-9116
www.CoastalPlanning.net

Table 21. Gulf Shoreline Change Rates Along Chenier Ronquille

	Total Shoreline Change (ft)			Annual Shoreline Change (ft/yr)		
Station	**1998-2006**	**2006-2010**	**1998-2010**	**1998-2006**	**2006-2010**	**1998-2010**
25+00	-325.7	-180.2	-505.9	-40.3	-48.0	-42.8
30+00	-498.3	-312.5	-810.8	-61.6	-83.3	-68.5
35+00	-403.9	-399.2	-803.1	-50.0	-106.5	-67.9
40+00	-413.9	-359.5	-773.4	-51.2	-95.9	-65.4
45+00	-453.9	-362.5	-816.3	-56.1	-96.7	-69.0
50+00	-	-	-1015.7	-	-	-85.8
55+00	-447.7	-346.4	-794.1	-55.4	-92.4	-67.1
60+00	-	-	-760.9	-	-	-64.3
65+00	-243.9	-224.5	-468.4	-30.2	-59.9	-39.6
70+00	-196.0	-190.9	-386.9	-24.2	-50.9	-32.7
75+00	-196.2	-195.4	-391.6	-24.3	-52.1	-33.1
80+00	-204.9	-194.2	-399.1	-25.3	-51.8	-33.7
85+00	-150.2	-171.3	-321.6	-18.6	-45.7	-27.2
90+00	-298.2	-145.9	-444.1	-36.9	-38.9	-37.5
95+00	-139.9	-485.4	-625.3	-17.3	-129.4	-52.8
100+00	-164.4	-269.7	-434.1	-20.3	-71.9	-36.7
105+00	-212.4	-170.7	-383.1	-26.3	-45.5	-32.4
110+00	-234.9	-68.7	-303.6	-29.1	-18.3	-25.7
115+00	-75.6	16.2	-59.4	-9.3	4.3	-5.0
120+00	-3.2	121.2	117.9	-0.4	32.3	10.0
Average	**-259.1**	**-218.9**	**-519.0**	**-32.0**	**-58.4**	**-43.9**

10.2.2 Overwash Rates

The rate of overwash is important when developing the sediment budget because overwash is an indication of conservation of sand in the system. Overwash rates were estimated along project transects by measuring the distance between the northern extents of the overwash fans between two time periods. The extent of the overwash fans was delineated by the +2-foot contour to the north of the fronting dune. Analysis of overwash rates suggests that overwash is occurring more quickly in the center of the island than at the extremities. Table 22 summarizes the overwash rates measured along each transect line. It should be noted that any southward movement of the +2-foot contour was set to zero as it was assumed that this was not a function of overwash. The average overwash rate measured over the length of the island varies between 2.7 feet/year, as measured between 1998 and 2006, and 41.4 feet/year, as measured between 2006 and 2010.

Table 22. Overwash Rates Along Chenier Ronquille

	Overwash Distance (ft)			Annual Overwash Rate (ft/yr)		
Station	**1998-2006**	**2006-2010**	**1998-2010**	**1998-2006**	**2006-2010**	**1998-2010**
25+00	0.0	0.9	0.0	0.0	0.2	0.0
30+00	0.0	57.3	0.0	0.0	15.3	0.0
35+00	95.5	18.5	14.8	11.8	4.9	1.2
40+00	2.0	271.0	269.5	0.2	72.3	22.8
45+00	7.9	234.3	242.2	1.0	62.5	20.5
50+00	-	-	326.0	-	-	27.5
55+00	0.0	17.5	327.2	0.0	4.7	27.7
60+00	-	-	207.5	-	-	17.5
65+00	67.2	260.6	232.8	8.3	69.5	19.7
70+00	0.0	68.9	262.4	0.0	18.4	22.2
75+00	2.3	66.2	68.5	0.3	17.7	5.8
80+00	0.0	0.0	0.0	0.0	0.0	0.0
85+00	0.0	33.3	0.0	0.0	8.9	0.0
90+00	0.0	47.6	0.0	0.0	12.7	0.0
95+00	149.1	850.2	999.3	18.4	226.7	84.4
100+00	32.1	618.9	596.2	4.0	165.0	50.4
105+00	6.7	164.1	170.8	0.8	43.7	14.4
110+00	0.0	83.2	77.3	0.0	22.2	6.5
115+00	13.4	0.0	137.8	1.7	0.0	11.6
120+00	10.9	1.8	92.1	1.3	0.5	7.8
Average	**21.5**	**155.2**	**201.2**	**2.7**	**41.4**	**17.0**

10.2.3 Bay Shoreline Changes

Bay shoreline changes were calculated along project transects by measuring the distance between the mean high water bay shorelines. Bay shoreline changes were calculated using the 2006 and 2010 surveys near the western extent of the island. The 1998 survey and the 2006 and 2010 surveys along the eastern extent of the island were not used as they do not extend across the island to capture the location of the bay shoreline. Analysis of bay shoreline changes suggests that the bay shoreline is moving south at a constant rate of 1.7 feet/year.

10.3 Shoreline Response to Relative Sea Level Rise

Bruun (1982) showed that beach profiles should adjust to increased water elevation with a recession of the shoreline and a deposition of sand in the offshore area (Figure 30). Shoreline recession due to relative sea level rise for a sand only system can be estimated using Bruun's (1982) rule (Equation 3).

$$x = rb/(h-d) \qquad \text{[Equation 3]}$$

where:

x = shoreline recession (feet)

r = relative sea level rise (feet)
b = distance from the berm crest to the break in profile slope (feet)
h = berm elevation (feet, NAVD)
d = break in slope elevation (feet, NAVD)

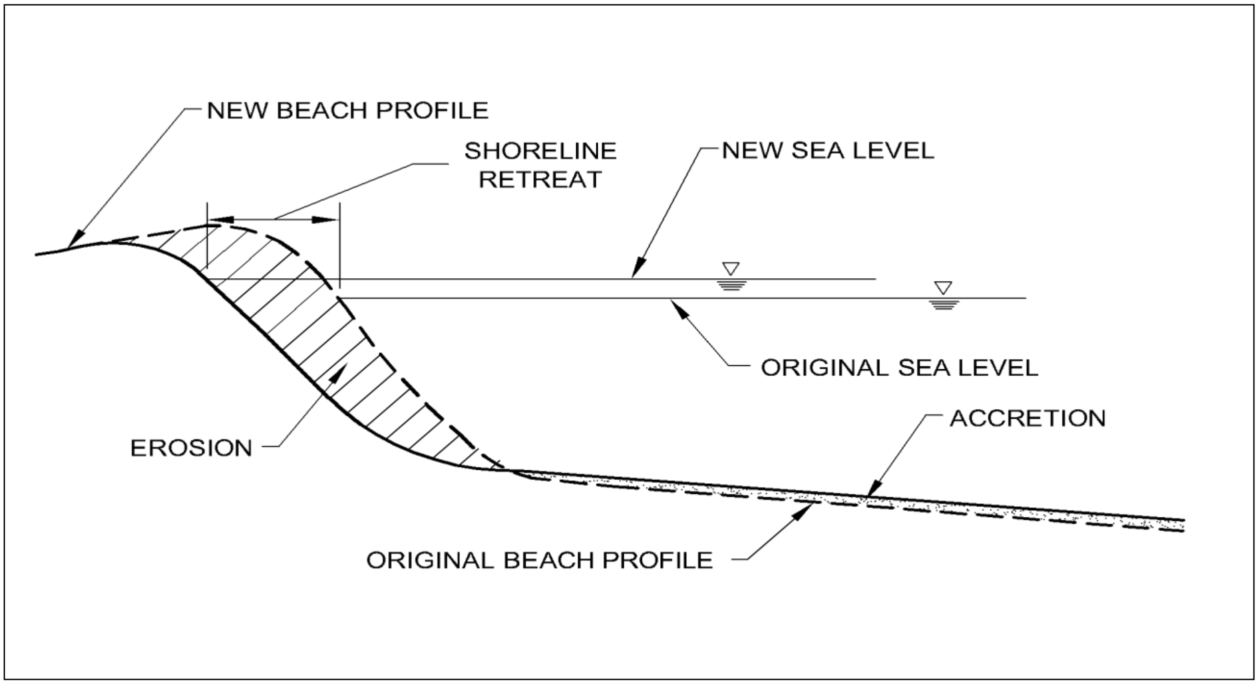


Figure 30. Impact of Sea Level Rise on Shoreline Recession

List et al. (1997) demonstrate a lack of correlation between observed and predicted shoreline retreat rates in Louisiana using Bruun's rule. However, CPE (2005) showed that in Louisiana Bruun's rule is better applied using the break in profile slope rather than the depth of closure. Therefore, the term b/(h-d) is the reciprocal of the nearshore slope observed along the profile. Based on the September 2010 beach profile surveys, the average nearshore slope is 1V:53H. This suggests that the Chenier Ronquille barrier shoreline recedes 53 feet for every 1 foot increase in relative sea level. Employing the current rate of relative sea level rise of 0.0303 feet/year, the Chenier Ronquille barrier shoreline is receding at a rate of 1.6 feet/year due to relative sea level rise.

11 ACTIVE PROFILE HEIGHT

Volume changes can be approximated by multiplying the shoreline change by the active profile height and the alongshore distance between profiles (USACE, 2001). Typically the active profile height extends from the berm crest to the depth of closure, where the depth of closure is defined as the depth at which there is no net cross-shore movement of sediment. In Louisiana, the nearshore sediment is composed of fine sand and silt. Therefore, a distinction must be made between the cross-shore movement of sand and the cross-shore movement of silt. As silt may remain in suspension over significantly longer time and distance scales than sand, the silt will have a deeper depth of closure that is much further offshore than fine sand.

Hallermeier (1978) and Birkemeier (1985) developed empirical equations to estimate the depth of closure based on wave parameters of an extreme wave event. The depth of closure can be estimated using the maximum significant wave height and associated period that is exceeded for 12 consecutive hours within the record employed. Since the depth of closure is event dependent, an average depth of closure was calculated. The maximum significant wave height and associated period exceeded for 12 consecutive hours were calculated for each yearly wave record. These wave heights and periods were then averaged to determine the wave height and period employed when calculating the depth of closure.

Using hindcast data for WIS Station 131, the average wave height exceeded 12 hours per year in the vicinity of the project area is 8.6 feet with a corresponding period of 8.0 seconds. The maximum yearly 12-hour wave events are provided in Table 23.

Table 23. Yearly Maximum Wave Height Exceeded 12 Consecutive Hours

Year	Event Date	Height (ft)	Period (s)
1980	October 27	7.8	7.7
1981	December 21	9.9	9.1
1982	December 25	9.6	8.3
1983	November 19	9.5	8.3
1984	February 27	9.5	9.1
1985	October 28	18.1	12.5
1986	March 18	7.9	7.7
1987	November 16	8.1	6.3
1988	December 27	6.4	6.3
1989	August 01	9.0	7.1
1990	November 09	7.1	5.0
1991	April 29	8.7	8.3
1992	August 27	7.0	5.0
1993	December 14	10.4	9.1
1994	November 27	9.4	8.3
1995	November 11	8.3	7.7
1996	November 30	9.7	9.1
1997	April 05	9.5	9.1
1998	September 11	7.8	5.0
1999	January 22	11.4	9.1
2000	November 08	5.5	6.4
2001	April 03	6.5	6.8
2002	September 26	11.4	10.7
2003	May 07	5.8	7.1
2004	October 09	7.6	7.7
2005	October 23	6.0	12.2
2006	October 16	8.7	7.5
2007	January 13	5.2	7.7
2008	May 03	6.7	7.6
2009	March 27	7.6	7.6
2010	May 02	10.3	8.8
Average		8.6	8.0

Using the average values, the depth of closure along the Chenier Ronquille Barrier Shoreline is estimated to be -17 feet (NAVD) using Hallermeier's equation and -13 feet (NAVD) using Birkemeier's equation. However, Mann and Thomson (2003) suggested that the sand depth of closure in Louisiana is much shallower. Based on the cross-shore sediment distribution, they suggested using -4 feet, NAVD as the depth of closure for sand volume calculations at Holly Beach.

Comparisons of beach profiles in Louisiana suggest that the submerged profile is comprised of two distinct shapes, as shown in Figure 31. The upper portion of the profile, extending several feet below mean high water, follows a profile shape described by equilibrium beach profile while the lower section of the profile is relatively flat. The elevation at which the two sections meet will be called the break in slope for the remainder of this report. The flat offshore profile suggests that as the island recedes there is little volumetric change Gulfward of the break in slope compared to the erosion occurring along the island face. The break in slope elevation can therefore be considered to be the bottom of the active profile when calculating sand volume changes along the Gulf shoreline.

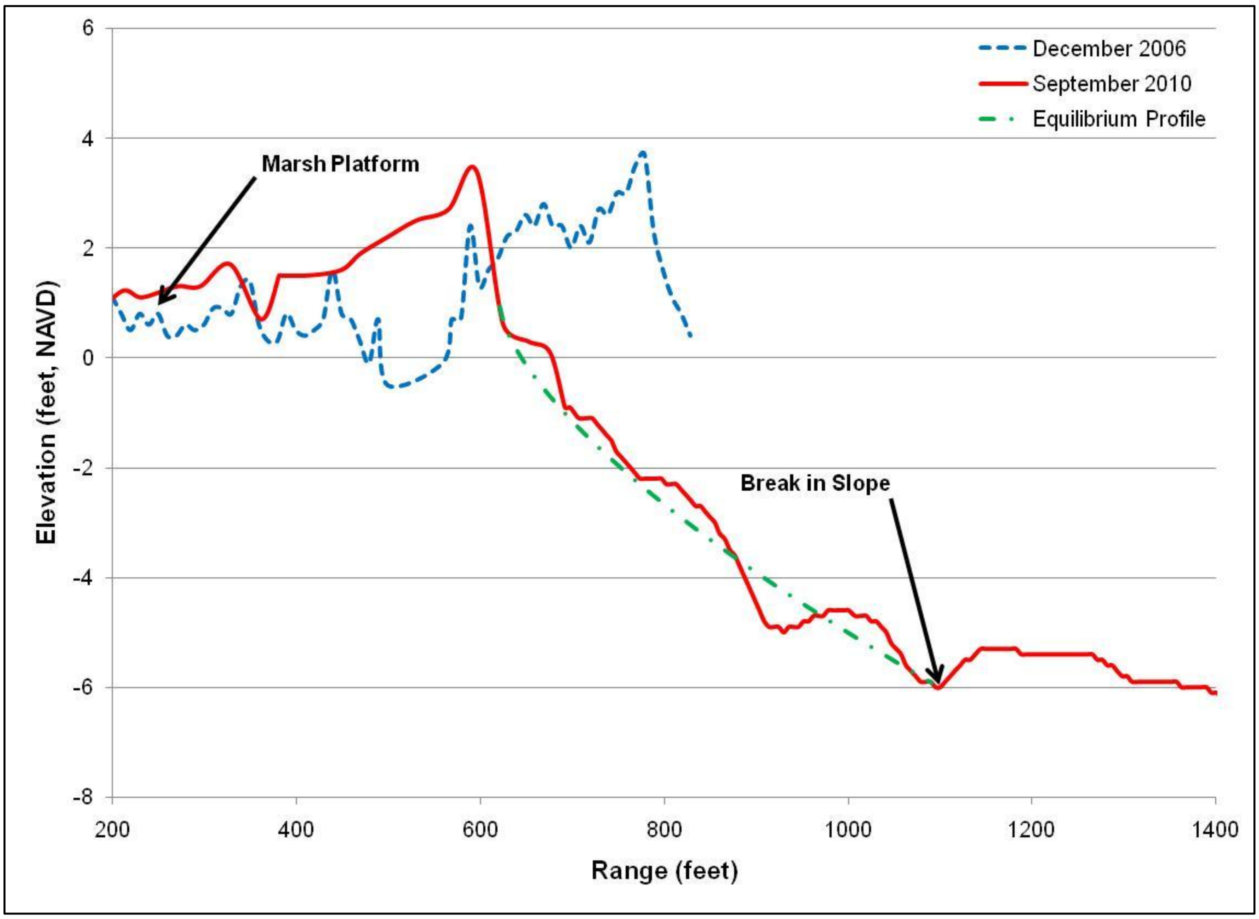


Figure 31. Nearshore Profile Comparison

The low-crested barrier islands in Louisiana are frequently overtopped. The resultant overwash of sediment deposited on the backing marsh platform is what maintains island elevation and helps rebuild the migrating dune (Campbell, 2005). The volume above the marsh remains relatively constant due to overwash, whereas the true volume change along the Gulf shoreline occurs between the top of the marsh and the break in slope (Campbell, 2005). Therefore, the marsh elevation can be considered to be the top of the active profile when calculating volume changes along the Gulf shoreline. An example of dune migration due to overwash is shown in Figure 31, where the LIDAR survey conducted in December 2006 is compared with the profile survey conducted in September 2010.

Inspection of profiles along the Chenier Ronquille reveals that not only is the dune migrating, but the marsh platform onto which the dune rolls over is also migrating. This landward migration of the marsh occurs when sediment is washed over the dune and is deposited in the backing bay or in the pipeline canals. Over time sediment deposited in the backing bay and pipeline canals builds up and eventually emerges to become new marsh. The volume above the marsh remains relatively constant due to overwash, whereas the true volume change along the edge of the marsh platform occurs between the top of the marsh and the bottom of the backing bay or pipeline canal. Therefore, the marsh elevation can be considered to be the top of the active profile and the bottom of the backing bay or pipeline canal can be considered to be the bottom of the active profile when calculating overwash volumes from bayside shoreline changes. An example of overwash filling a pipeline canal is shown in Figure 31.

Examination of the cross-shore profiles along Chenier Ronquille, as surveyed in September 2010, showed a break in slope elevation between -9.0 and -1.5 feet, NAVD, a marsh platform elevation between 1.1 and 2.9 feet, NAVD, and a bottom of bay/pipeline canal elevation between -6.0 and -0.5 feet, NAVD. Above the break in slope, the profile is a concave curve of the approximate form $y = ax^{2/3}$, while the profile is relatively flat with an offshore slope of approximately 1V:400H below the break in slope. The break in slope, marsh platform, and bottom of bay/pipeline canal elevations for all project area profiles are provided in Table 24, along with the Gulf and overwash active profile heights.

Table 24. Project Area Active Profile Height and Limits

Station	Marsh	Elevation (ft, NAVD) Break in Slope	Bay/Canal	Active Profile Height (ft) Gulf	Overwash
25+00	1.1	-9.0	-6.0	10.1	7.1
30+00	1.1	-9.0	-6.0	10.1	7.1
35+00	1.1	-9.0	-1.2	10.1	2.3
40+00	1.1	-9.0	-1.2	10.1	2.3
45+00	1.1	-9.0	-1.2	10.1	2.3
50+00	1.1	-9.0	-1.2	10.1	2.3
55+00	1.1	-9.0	-1.2	10.1	2.3
60+00	1.1	-9.0	-1.2	10.1	2.3
65+00	1.1	-5.0	-1.2	6.1	2.3
70+00	1.1	-5.0	-1.2	6.1	2.3
75+00	1.1	-5.0	-1.2	6.1	2.3
80+00	1.1	-5.0	-1.2	6.1	2.3
85+00	1.1	-5.0	-1.2	6.1	2.3
90+00	1.1	-3.5	-0.5	4.6	1.6
95+00	1.1	-3.5	-0.5	4.6	1.6
100+00	1.1	-3.5	-0.5	4.6	1.6
105+00	1.1	-3.5	-0.5	4.6	1.6
110+00	2.9	-3.5	-0.5	6.4	3.4
115+00	2.9	-1.5	-0.5	4.4	3.4
120+00	2.9	-1.5	-3.5	4.4	6.4

12 VOLUME CHANGES

Five data sets were available to estimate volume changes along Chenier Ronquille. The first survey data set is the November 1998 LIDAR survey. The second was an October 2000 survey, collected as part of the design work for Chaland Headland. The third was a survey collected in November 2000, as part of the Coast 2050 study. Given that the October and November 2000 surveys were collected only one month apart they can be considered to be the same survey. The fourth survey data set is the BICM 2006 bathymetric survey joined with the 2006 LIDAR survey. The fifth survey data set is the September 2010 survey conducted by JCLS as a part of this design work.

Profile data are normally used to estimate volume changes in project areas. The temporal proximity of the surveys collected in 2000 and the dissimilar lines surveyed does not facilitate the computation of project area volume change estimates. LIDAR data is also available, but it only captures subaerial changes in the profile. Combined LIDAR and bathymetric surveys describe the subaerial and offshore profiles, but fail to overlap in the nearshore or active part of the profile. Therefore, it was not possible to directly calculate volume changes based on this survey data. However, shoreline data, extracted from LIDAR and profile survey data sets, are available and allowed for a greater number of comparison points along the barrier island. Therefore, it was decided to base volumetric change estimates on the shoreline change data.

Shoreline based volumetric changes can be approximated by multiplying the shoreline change by the active profile height and the alongshore distance between profiles (USACE, 2001). Therefore, the volume lost from the Gulf face was estimated using Gulf shoreline changes, as shown in Figure 29, and the Gulf active profile, which extends from the marsh platform to the break in slope (sand depth of closure), as discussed in Section 10 and shown in Table 24. Overwash volumes were estimated in a similar fashion using marsh shoreline changes, as shown in Table 22, and the overwash active profile, which extends from the marsh platform to the bottom of the bay/pipeline canal, as discussed in Section 10 and shown in Table 24. Volumetric losses are shown as a negative volume change while volumetric gains are shown as a positive volume change.

12.1 Gulf Volume Changes

Gulf volume changes were calculated along project transects by multiplying the Gulf shoreline change by the Gulf active profile height. Analysis of Gulf volume changes suggests that the west end of the island is eroding more than the east end of the island. Table 25 summarizes the Gulf volume changes calculated along each transect line. It should be noted that negative Gulf volume changes denote erosion while positive volume changes denote accretion. The total Gulf erosion measured over the length of the island varies between 85,527 cubic yards/year, as measured between 1998 and 2006, and 150,714 cubic yards/year, as measured between 2006 and 2010.

Table 25. Gulf Volume Changes Along Chenier Ronquille

	Total Volume Change (cy)			Annual Volume Change (cy/yr)		
Station	**1998-2006**	**2006-2010**	**1998-2010**	**1998-2006**	**2006-2010**	**1998-2010**
25+00	-60,925	-33,698	-94,626	-7,537	-8,986	-7,997
30+00	-93,202	-58,442	-151,644	-11,530	-15,584	-12,815
35+00	-75,539	-74,671	-150,209	-9,345	-19,912	-12,694
40+00	-77,409	-67,247	-144,656	-9,576	-17,933	-12,224
45+00	-84,891	-67,795	-152,684	-10,502	-18,079	-12,903
50+00	-	-	-189,972	-	-	-16,054
55+00	-83,736	-64,790	-148,526	-10,359	-17,277	-12,552
60+00	-	-	-142,313	-	-	-12,026
65+00	-27,548	-25,360	-52,908	-3,408	-6,763	-4,471
70+00	-22,135	-21,569	-43,705	-2,738	-5,752	-3,693
75+00	-22,163	-22,076	-44,240	-2,742	-5,887	-3,739
80+00	-23,146	-21,935	-45,081	-2,863	-5,849	-3,810
85+00	-16,970	-19,352	-36,323	-2,099	-5,160	-3,070
90+00	-25,405	-12,427	-37,832	-3,143	-3,314	-3,197
95+00	-11,917	-41,351	-53,268	-1,474	-11,027	-4,502
100+00	-14,004	-22,978	-36,982	-1,733	-6,127	-3,125
105+00	-18,093	-14,537	-32,630	-2,238	-3,876	-2,757
110+00	-27,840	-8,142	-35,982	-3,444	-2,171	-3,041
115+00	-6,158	1,320	-4,838	-762	352	-409
120+00	-263	9,871	9,608	-33	2,632	812
Total	**-691,346**	**-565,178**	**-1,588,812**	**-85,527**	**-150,714**	**-134,266**

12.2 Overwash Volumes

Overwash volumes were calculated along project transects by multiplying the marsh shoreline change by the overwash active profile height. Analysis of overwash volumes suggests that overwash is greater in the center of the island than at the extremities. Table 26 summarizes the overwash volumes calculated along each transect line. It should be noted that negative overwash volumes were set to 0 as it was assumed that the dune cannot be overwashed from the landward to seaward side. The total overwash measured over the length of the island varies between 1,875 cubic yards/year, as measured between 1998 and 2006, and 27,791 cubic yards/year, as measured between 2006 and 2010.

Table 26. Overwash Volumes Along Chenier Ronquille

	Total Overwash (cy)			Annual Overwash (cy/yr)		
Station	**1998-2006**	**2006-2010**	**1998-2010**	**1998-2006**	**2006-2010**	**1998-2010**
25+00	0	112	0	0	30	0
30+00	0	7,531	0	0	2,008	0
35+00	4,069	788	630	503	210	53
40+00	84	11,542	11,480	10	3,078	970
45+00	337	9,980	10,317	42	2,661	872
50+00	-	-	13,883	-	-	1,173
55+00	0	747	13,938	0	199	1,178
60+00	-	-	8,839	-	-	747
65+00	2,862	11,098	9,914	354	2,960	838
70+00	0	2,935	11,174	0	783	944
75+00	100	2,820	2,919	12	752	247
80+00	0	0	0	0	0	0
85+00	0	1,417	0	0	378	0
90+00	0	1,409	0	0	376	0
95+00	4,419	25,190	29,609	547	6,717	2,502
100+00	952	18,337	17,666	118	4,890	1,493
105+00	198	4,861	5,059	25	1,296	428
110+00	0	5,240	4,867	0	1,397	411
115+00	844	0	8,678	104	0	733
120+00	1,292	207	10,919	160	55	923
Total	**15,156**	**104,216**	**159,892**	**1,875**	**27,791**	**13,512**

13 SEDIMENT BUDGET

A sediment budget was developed for Chenier Ronquille to describe the movement of beach and marsh sediment into, out of, and within the project area. This allows project performance to be predicted even though the beach and dune system following construction is fundamentally different from the pre-construction conditions due to reduced silt content. The volumes of sand required for the various beach options to meet project objectives can thus be assessed.

The sediment budget was developed by decomposing the volumetric change, described in the previous section, into component parts. This section discusses the separation of the volume change into offshore losses associated with island composition, losses due to relative sea level rise, overwash, and longshore transport.

13.1 Island Composition

The Chenier Ronquille barrier island is composed of sand, silt, and clay. A distinction must be made within a sediment budget to account for the difference in material. From a coastal engineering perspective, it is the volume of sand within the system that is important because the sand provides protection from wave attack. When silt and clay are exposed they are suspended in the water column and can be transported offshore.

The sediment budget assumes that silt and clay exposed along the Gulf shoreline face are placed in suspension and transported offshore, while the sand fraction remains behind. The sand component is then transported alongshore or overwashes the island. Therefore, volume changes must be reduced by the percent silt and clay to obtain the sand volume changes and longshore transport rates. Campbell (2005) suggested that marsh samples should be taken to estimate the silt content in the island as the beach face is transient and does not represent the composition of the island.

Nine borings were collected in the project area to determine the composition of the island. As discussed in Section 6.3, the subsurface soils generally consist of alternating layers of granular and cohesive materials from the mudline to the completion depth of the borings. The borings suggest that the island is composed of 59% silt and 41% sand between 1 foot, NAVD and -9 feet, NAVD. Therefore, the total volumetric loss was reduced by 59%. The volume lost offshore was estimated at 89,780 cubic yards/year, as shown in Table 27. Note that where the shoreline advanced (Station 120+00) there is no volumetric gain of silt.

Table 27. Sediment Budget for Chenier Ronquille (1998 to 2010)

Station	Total Volume Lost from Gulf Face (cy/yr)	Volume Lost Offshore (Silt Loss) (cy/yr)	Volume Lost to RSLR (cy/yr)	Overwash Volume (cy/yr)	Longshore Sand Volume Change (cy/yr)	Longshore Transport Rate (cy/yr)
25+00	8,788	5,194	748	0	2,847	39,027
30+00	14,084	8,324	748	0	5,013	36,180
35+00	13,951	8,245	748	53	4,905	31,167
40+00	13,435	7,940	748	970	3,777	26,263
45+00	14,180	8,381	748	872	4,180	22,486
50+00	17,643	10,427	748	1,173	5,295	18,305
55+00	13,794	8,152	748	1,178	3,716	13,010
60+00	13,217	7,811	748	747	3,911	9,294
65+00	4,838	2,859	445	838	696	5,383
70+00	3,996	2,362	445	944	246	4,687
75+00	4,045	2,391	445	247	963	4,441
80+00	4,122	2,436	445	0	1,241	3,478
85+00	3,321	1,963	445	0	914	2,237
90+00	4,240	2,506	411	0	1,323	1,323
95+00	5,969	3,528	411	2,502	-472	0
100+00	4,144	2,449	411	1,493	-209	472
105+00	3,657	2,161	411	428	657	681
110+00	3,753	2,218	532	411	592	24
115+00	734	434	532	733	-965	-568
120+00	-1,458	0	532	923	-2,913	397
Total	**150,454**	**89,780**	**11,446**	**13,512**	**35,717**	

13.2 Relative Sea Level Rise

A portion of the shoreline recession is due to the island maintaining its elevation while relative sea level rise (RSLR) processes are occurring. Shoreline recession due to RSLR does not result in a net volume change in the cross-shore profile but simply a redistribution of the sediment across the profile (Figure 30). The volume change calculated from shoreline recession must therefore be reduced to account for RSLR prior to calculating the volume change used to develop the sand longshore transport rate. The shoreline retreat due to relative sea level rise was estimated to be 1.6 feet/year (Section 9.3). This was multiplied by the cell width and the Gulf active profile height to determine the volumetric loss due to relative sea level rise. Therefore, relative sea level rise accounts for a volumetric loss of approximately 11,446 cubic yards/year along the length of the island, as shown in Table 27.

13.3 Overwash

The average dune crest elevation along Chenier Ronquille is low at approximately 3.3 feet, NAVD. As a result, sediment is transported over the top of the dune by waves during storm events. This is the primary mechanism for island rollover. The landward movement of sediment conserves sediment within the system despite the landward shift of the shoreline. Therefore, the volume change, calculated from shoreline change, must be reduced to account for the conservation of the sediment within the system. The overwash volume was estimated using marsh migration rates, as shown in Table 22, and the overwash active profile, which extends from the marsh platform to the bottom of the backing bay/pipeline canal, as discussed in Section 10 and shown in Table 24. The annual overwash volume was estimated at 13,512 cubic yards/year, as shown in Table 27.

13.4 Longshore Transport

This section discusses the longshore transport rate for Chenier Ronquille. An annualized sediment budget was developed using shoreline changes, active profile heights, island composition percentages, and relative sea level rise rates.

The conservation of sand principle was used to estimate the volume of sand transported in a longshore direction. The conservation of sand equation allows for the longshore transport to be estimated using Equation 4.

$$LT_{out} = V_{total} - V_{offshore} - V_{RSLR} - V_{overwash} + LT_{in} \qquad \text{[Equation 4]}$$

where:

LT_{out}	= Longshore transport out of the reach
V_{total}	= Volume change calculated based on shoreline change
$V_{offshore}$	= Offshore (silt) volume loss
V_{RSLR}	= Volume change associated with relative sea level rise
$V_{overwash}$	= Volume of overwash
LT_{in}	= Longshore transport into the reach from an adjacent cell

Table 27 summarizes the various components of volume change along Chenier Ronquille between 1998 and 2010. From this a volume change within each cell due to longshore transport was calculated. If the longshore sand volume change is greater than zero then net sediment transport was into the cell, while a negative value indicates that net sediment transport was out of the cell. However, a positive or negative value for volume change due to longshore transport does not indicate the direction of longshore transport. The net longshore transport rate along the barrier island is determined by integrating these volumes in a longshore direction (Equation 4).

A starting point for the longshore transport integration must be identified. An area of zero net sediment transport (a nodal point) is the typical point at which to start such a summation because it is easier to identify an area of no net sediment transport than estimate the longshore transport at a given point. Given that the shoreline appears to be rotating slightly about the eastern end of the island, the nodal point should be located at the eastern end of the island. The nodal point is often located at the location with the highest shoreline retreat. The highest retreat rate along the eastern end of the island occurs at Station 95+00, located 9,500 feet east of the western project extent. This also coincides with the location where the island has formed its largest breach. Due to tidal currents and wave refraction, there is often a longshore transport reversal adjacent to an inlet or major breach. Therefore, Station 95+00 was chosen as the nodal point and the start of the longshore transport integration (Table 27). A plot of the longshore sediment transport curves is shown in Figure 32.

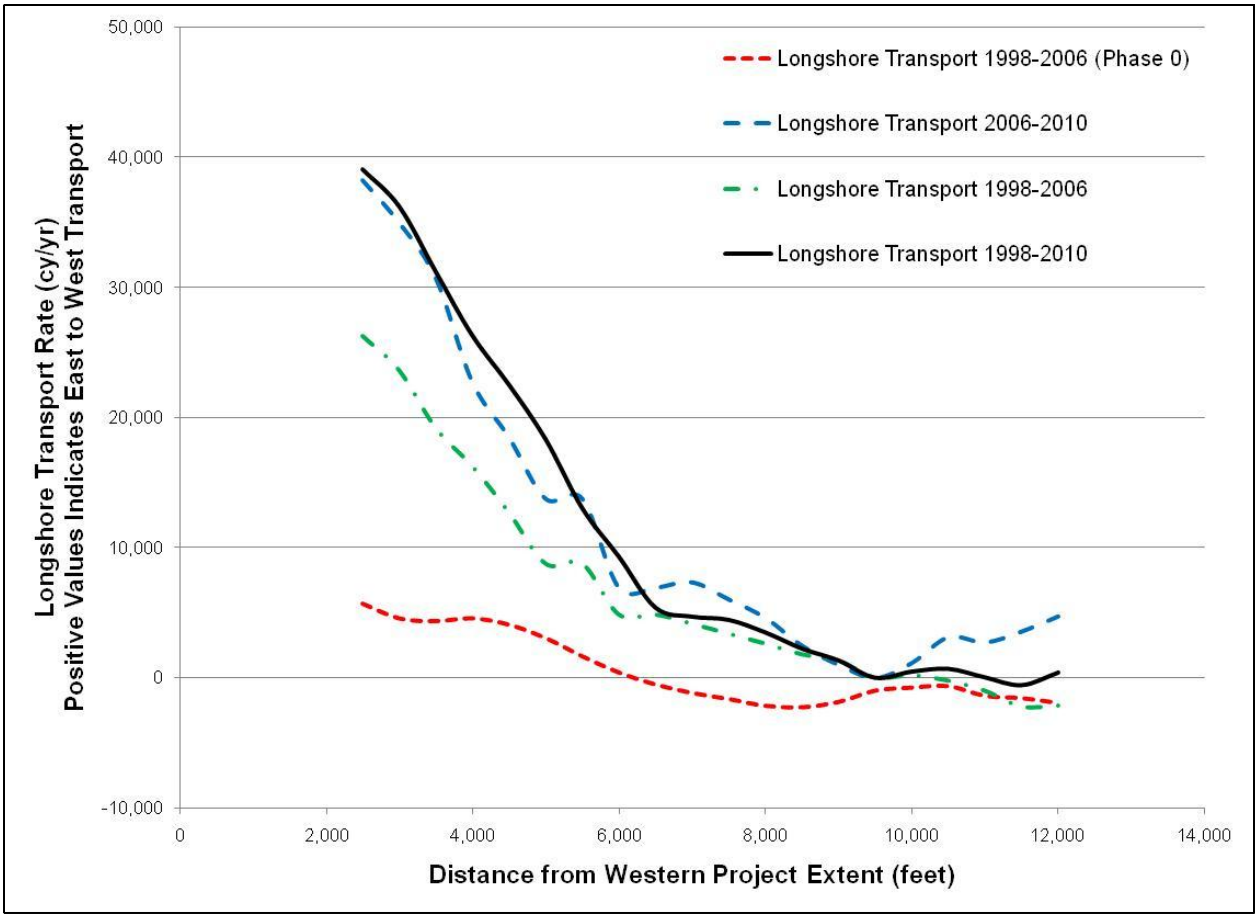


Figure 32. Longshore Sediment Transport Curve for Chenier Ronquille

The slope of the longshore transport curve indicates whether erosion or accretion is occurring and the severity of this erosion or accretion. Areas of higher erosion (or accretion) will result in a steeper longshore transport curve. Stable areas will result in a flatter longshore transport curve. Therefore, the longshore transport curve suggests that erosion is occurring along most of the project length though higher at the western end.

Figure 33 shows that the net longshore transport rate estimated for Chenier Ronquille increased to a maximum of approximately 39,000 cubic yards/year at the western extent of the barrier island. Approximately 3,300 cubic yards/year are being transported across Pass la Mer from Chaland Headland to Chenier Ronquille. Of this 3,300 cubic yards/year, only 400 cubic yards/year continue to be transported west as the remaining 2,900 cubic yards/year are forming a bulge in the shoreline which is causing a transport reversal. Therefore, the net loss due to longshore transport along the Chenier Ronquille barrier shoreline is 38,600 cubic yards/year.

LEGEND:

- LONGSHORE TRANSPORT (CY/YR)
- OVERWASH (CY/YR)
- OFFSHORE LOSS (CY/YR)
- RSLR LOSS (CY/YR)

NOTES:

1. THIS SEDIMENT BUDGET WAS DEVELOPED USING SHORELINE BASED VOLUMETRIC CHANGES BETWEEN 1998 AND 2010.
2. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
3. DATE OF PROJECT AREA PHOTOGRAPH: AUGUST 2010.

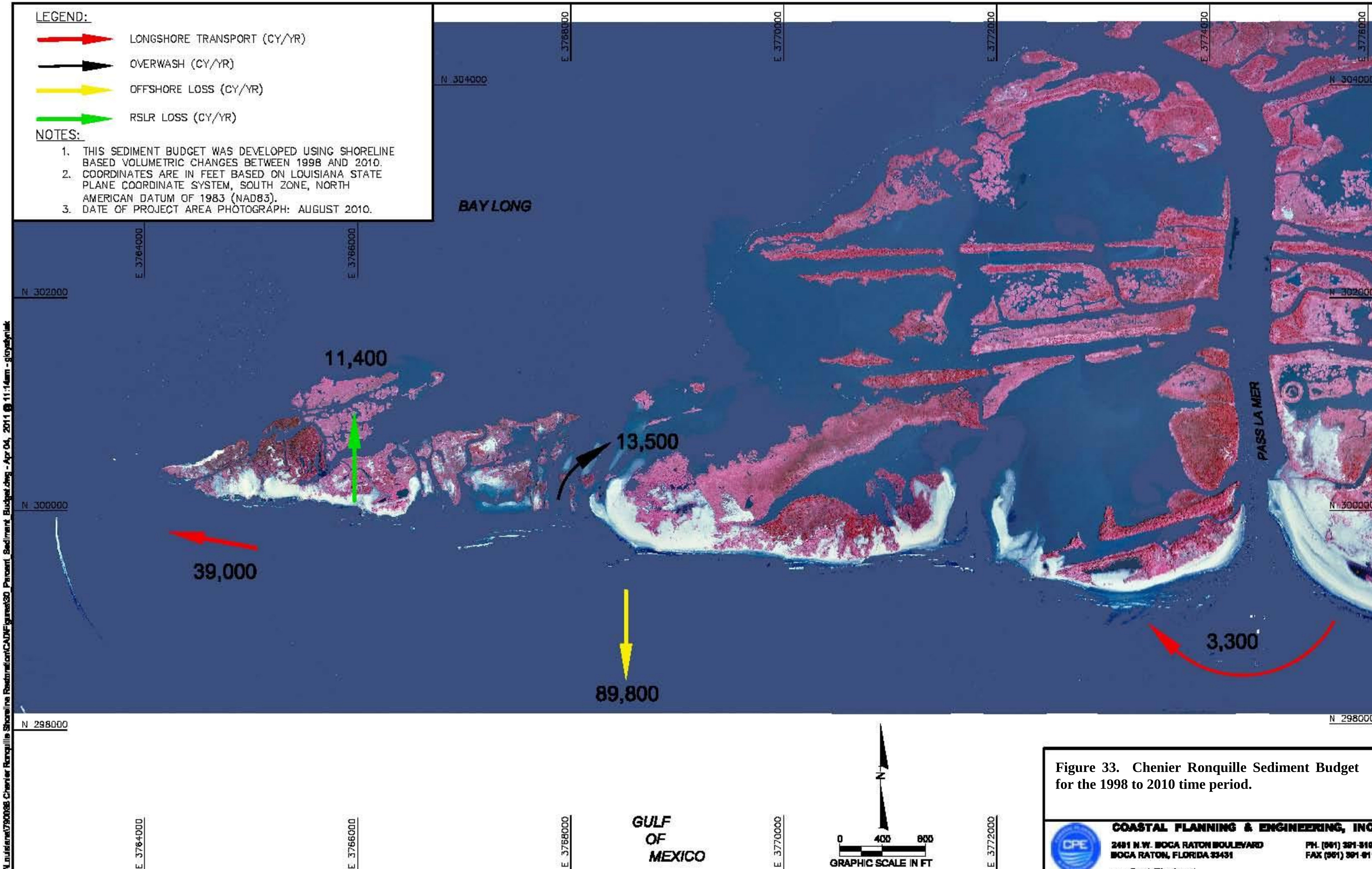


Figure 33. Chenier Ronquille Sediment Budget for the 1998 to 2010 time period.

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The development of the longshore transport curve along Chenier Ronquille allows for the prediction of sediment transport under a nourished, with project condition. The longshore transport rate can be applied to alternatives that are similar to the island conditions between 1998 and 2010. It should be recognized that with the nourishment of Chaland Headland, the transport of sand onto Chenier Ronquille increased due to newly available sediment. However, this was ignored in order to provide a conservative benefit and design. Figure 33 provides a summary view of the various components of the sediment budget.

14 PLANFORM DESIGN ALTERNATIVES

The planform of a project is driven by several variables including existing and historic island footprint, maximizing project effectiveness, existing island features that could assist constructability (such as spoil banks that can be used for marsh containment), oil and gas infrastructure, assessment of future impacts to island features, and project cost.

The original scope of work proposed a beach fill that extended the complete length of Chenier Ronquille with a backing marsh of variable width. It was proposed to have the northern limit of the marsh fill follow the southern edge of the Columbia Gulf Transmission pipeline canal (Figure 34). Several other alternatives to the original concept were also considered during the Phase 0 report development. These are outlined in Figure 34 and discussed below.

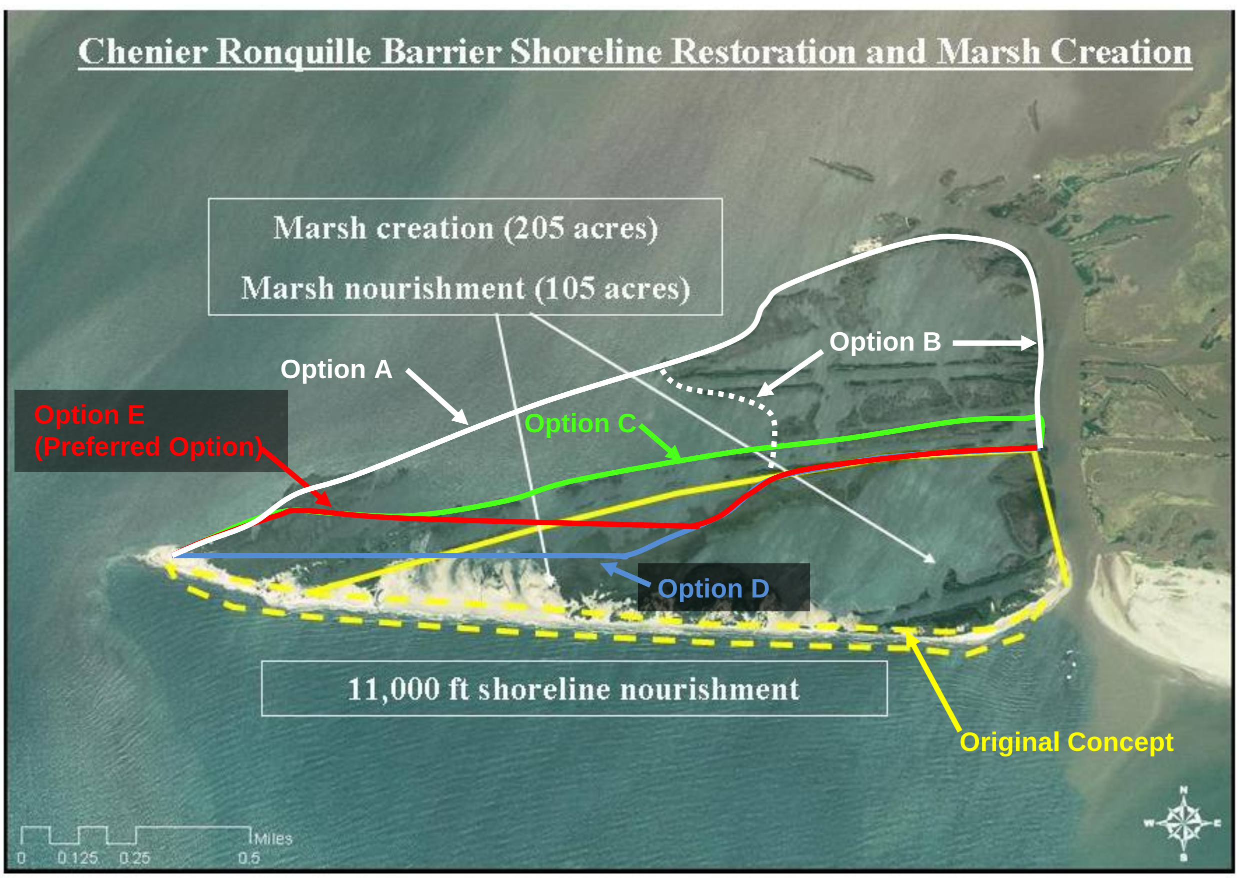


Figure 34. Project Footprints Considered During Phase 0 Development

Option A investigated whether a large marsh encompassing all of the remnants of Chenier Ronquille would be feasible. It was estimated that the cost would exceed the budget given the large area to fill and the difficulty associated with constructing and maintaining the primary dikes across the northwest opening.

Option B eliminated the large marsh section on the western side, while restoring an existing marsh section. Again budget considerations eliminated this option along with a cursory benefit analysis that suggested the benefits of constructing a beach would not extend this far north.

Option C shifted the marsh limit from the original concept to the northern side of the Columbia Gulf Transmission canal and extended the marsh further west. The increase in marsh fill volume resulted in the expected cost exceeding a budget limit of $35M. However, the 2010 survey showed the average water depths to be shallower than expected and this option became viable once more. This is similar to the footprint of marsh option 1 (Section 15.4.1).

Option D extended the marsh to the western end of the island but moved the marsh boundary south. This option was developed based on allowing marsh buggies to construct the entire primary dike. However, the width of the marsh on the west end was very narrow (~400 feet). This was deemed to be too narrow to be effective based on the overwash observed on Chaland Headland following Hurricanes Gustav and Ike.

Option E was chosen as the preferred layout for the Phase 0 Report because it provided a wider marsh on the west end of the island, minimized oil and gas infrastructure concerns and was within the budget limit.

15 BEACH FILL DESIGN

This section discusses the development of the beach fill design. The beach fill design extends along 8,000 feet of Gulf shoreline between Quatre Bayou Pass and Pass La Mer. The design was reduced from 11,000 feet in the Phase 0 design to save volume and cost. The general concepts that affect the design are discussed in the following sections, such as the construction template, design cross-section, profile equilibration, and advanced fill.

15.1 Construction Template

The construction cross-section was based on borrow sediment properties and experience from construction templates used for similar restoration projects in Louisiana. The following design details were incorporated:

1. Gulfward Slope of Dune: 1V:30H above +1.0 feet, NAVD
   1V:60H below +1.0 feet, NAVD

2. Bayward Slope of Dune: 1V:30H

These slopes and break points in the slopes were partially based on the constructed profiles at Holly Beach and East Grand Terre.

At Holly Beach a 1V:30H slope was used above +1.3 feet, NAVD while a 1V:45H slope was used below this elevation (CPE, 2003a). The borrow source at Holly Beach had a grain size of 0.13 mm (CPE, 2003a) and a silt content of 13%. The Contractor incurred non-pay losses above the offshore template at Holly Beach, which suggests a flatter slope could be used. The break in slope elevation was based on the approximate location of the mean high water line. Below mean high water, the contractor has limited control over the construction slope.

The East Grand Terre Island Restoration Project started with a 1V:90H offshore construction slope that was altered during construction to a 1V:75H offshore slope (CPE, 2011). The subaerial (above +1.0 feet, NAVD) slope remained at 1V:30H throughout the project. The borrow area used for East Grand Terre had a grain size of 0.10 mm and a silt content of 10% (CPE, 2003b). The non-pay losses following the slope adjustment were minimal as the contractor constructed an offshore slope that was steeper than the adjusted slope.

The borrow areas used for Holly Beach and East Grand Terre have characteristics similar to those proposed for Chenier Ronquille. The borrow areas proposed for Chenier Ronquille have a grain size of approximately 0.11 mm and a silt content ranging from 8.4% to 11.7%. The similar grain size and silt content suggest that the offshore slope should be steeper than that used for East Grand Terre but flatter than that used for Holly Beach. Therefore, a 1V:60H offshore slope is proposed for Chenier Ronquille. Furthermore, a 1V:60H offshore slope approximates the existing nearshore profile.

No steps (berm platforms) are proposed for the construction template. Additional working of the fill can increase the cost of construction and does not provide any added benefits compared to a single slope. Regardless of the construction template, the material will be reworked into a natural profile by wave and wind action.

15.2 Dune Settlement

The soil underlying the fill material will consolidate due to the addition of beach fill. Consolidation of the underlying soils is a function of the consolidation properties, the area of the applied load, and the thickness of fill above the water table (surcharge load). It is also expected that consolidation of the underlying soils will vary due to differences in the lift thickness, variations in the properties of the marsh fill material, and differences in the project footprint. A settlement analysis of the consolidation of the underlying soils was completed by Fugro Consultants, Inc. and is included in Appendix D. This analysis ignored secondary compression, desiccation, and evaporation of water from the fill material because these factors have limited effect on fill consolidation when compared to other variables.

Fugro Consultants, Inc. used boring samples, collected throughout the project area, to develop settlement estimates. The borings were not consistent in composition throughout the project area and it was decided to estimate the settlement using all of the borings and then the boring samples collected only along the beach face (borings B1 and B2). The settlement curves developed for a dune constructed to +6.0 feet, NAVD were compared to data collected from settlement plates installed during the East Grand Terre Island Restoration Project (CPE, 2011) and the Pass Chaland to Grand Bayou Pass Barrier Shoreline Restoration Project (CEC, 2005). For both projects, the dune crests were constructed to an elevation of +6 feet, NAVD. Figure 35 shows the dune settlement with EGT denoting data collected during the East Grand Terre Island project and BA-35 denoting data collected during the Pass Chaland project. After review of all the settlement curves, Fugro's analysis using all of the borings in the project area was used for the settlement analysis because it more closely represented settlement observed in the field.

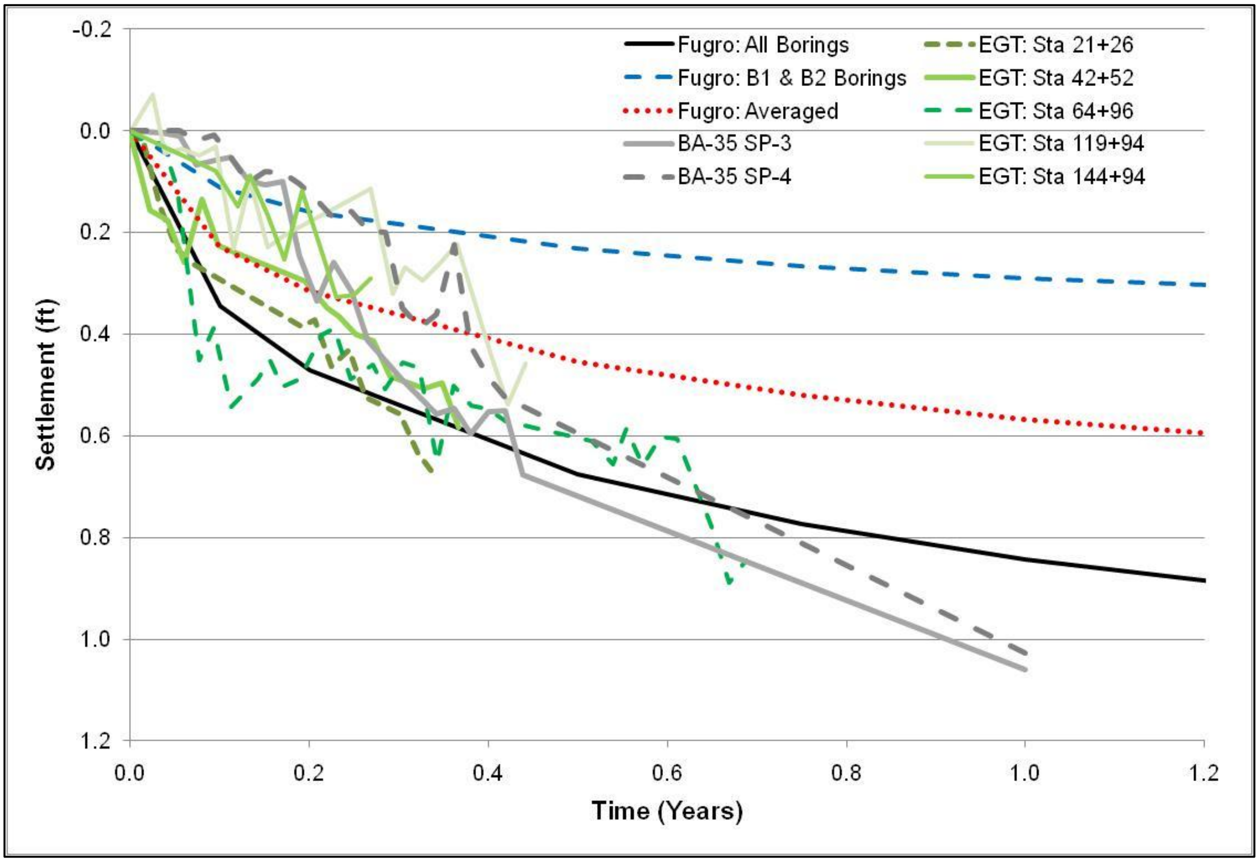


Figure 35. Settlement Curves of Dune Constructed to +6.0 feet, NAVD

15.3 Design Cross-Section

A standard beach nourishment cross-section consists of two primary components:

1. The design section, which is the cross-section required to meet project objectives. In this case, the design section is based on maintaining a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.

2. Advanced fill, which is the sacrificial portion of the fill required to protect the design section from anticipated sediment losses due to longshore losses, relative sea level rise losses, overwash, offshore losses, and settlement (see Section 15.5 for details).

This two-section design is in accordance with the National Research Council (1995) recommendations.

Cross-shore modeling (SBEACH) was used to evaluate the performance of the cross-section with respect to overtopping and post-storm dune elevation. The purpose of the analysis was to determine a cross-section that would resist breaching and maintain a sufficient dune elevation to prevent overtopping by more frequent storm events.

Previous Louisiana barrier island designs have been based upon maintaining a +4 feet, NAVD post-storm dune crest elevation in TY20. These designs typically applied a constructed dune elevation of +6.0 feet, NAVD. However, when the settlement rates estimated by Fugro are applied to TY20 conditions, the design objective of +4.0 feet, NAVD dune in TY20 cannot be met. With settlement and subsidence alone, a constructed dune with an elevation of +6.0 feet, NAVD will settle and subside to an elevation of +3.6 feet, NAVD. Therefore, it was decided to evaluate alternatives including a +8.0 feet, NAVD dune height in addition to a +6.0 feet dune.

To account for conditions in TY20, the entire profile was lowered vertically by offsetting it 0.495 feet due to subsidence throughout the project life (Table 19). The dune portion of the profile was settled by 2.5 feet, per guidance from Fugro (Appendix D). The water level was raised by 0.039 feet to account for eustatic sea level rise. Thus, a dune constructed at an elevation of +8.0 feet, NAVD will be +5.0 feet, NAVD in TY20, assuming that no other storms lower the dune crest prior to that time. The dune width was varied to determine the effect of dune width on the final profile elevation. Figure 36 shows the SBEACH output following a 20-year storm event (Hurricane Katrina) under TY20 conditions. This shows that a 200-foot wide dune crest at +8 feet, NAVD would be required to maintain a +4 feet, NAVD dune elevation in TY20. The design volume for this section is approximately 1.5M cubic yards of beach fill. The advanced fill volume requirement is approximately 1.3M cubic yards. Therefore, the total volume requirement for this design section is approximately 2.8M cubic yards.

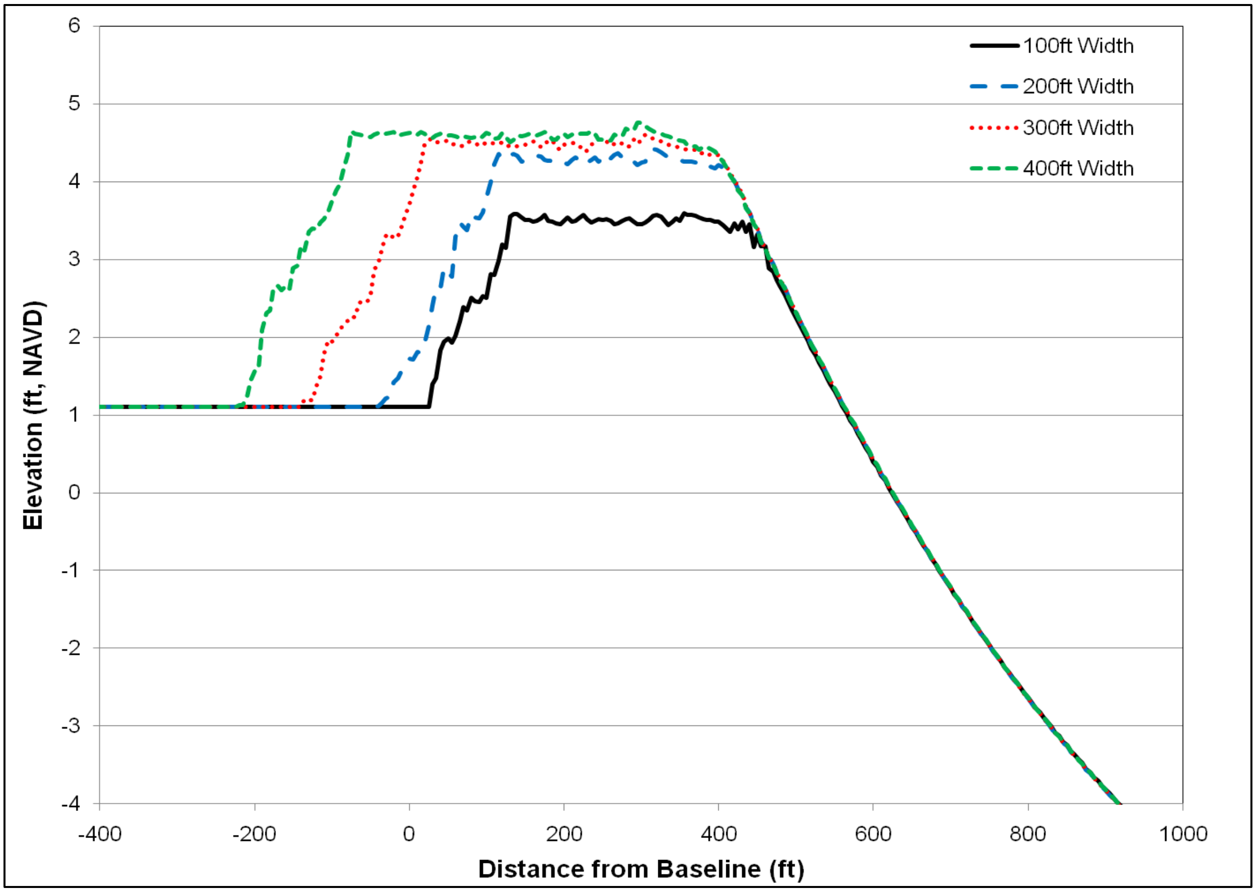


Figure 36. SBEACH Output for Various Dune widths for a +8 feet, NAVD Constructed Dune following a 20-year storm in TY20.

The cost of constructing a project with a beach fill volume of 2.8M cubic yards in order to meet the TY20 objective of +4 feet, NAVD exceeded the available budget. Therefore, the project objectives were refined to try and meet a more manageable target. Another design objective was developed to maintain dune elevation (+5.0 feet, NAVD) following the first 10-year return period storm event, while preventing breaching throughout the 20-year project life.

After evaluating probability curves, there is a 50% likelihood that a 10-year storm event will occur by TY7. To account for conditions in TY7, the entire profile was subsided by 0.15 feet, the dune portion of the profile was settled by 1.90 feet, and the water level was raised by 0.039 feet. Thus, the constructed dune elevation of +8.0 feet, NAVD was lowered to +5.95 feet, NAVD to simulate TY7 conditions.

As with the analysis for the 20-year conditions, the dune width was varied for the TY7 conditions in order to determine the width required to maintain +5 feet, NAVD following the 10-year storm event. Figure 37 shows that a 90-foot wide dune crest will meet this criterion. Therefore, a 90-foot dune crest at +5.95 feet, NAVD in TY7 is the design condition. This corresponds to a +8 feet, NAVD dune crest at construction. The volume required to construct this template (the design volume) is 1,300,000 cubic yards.

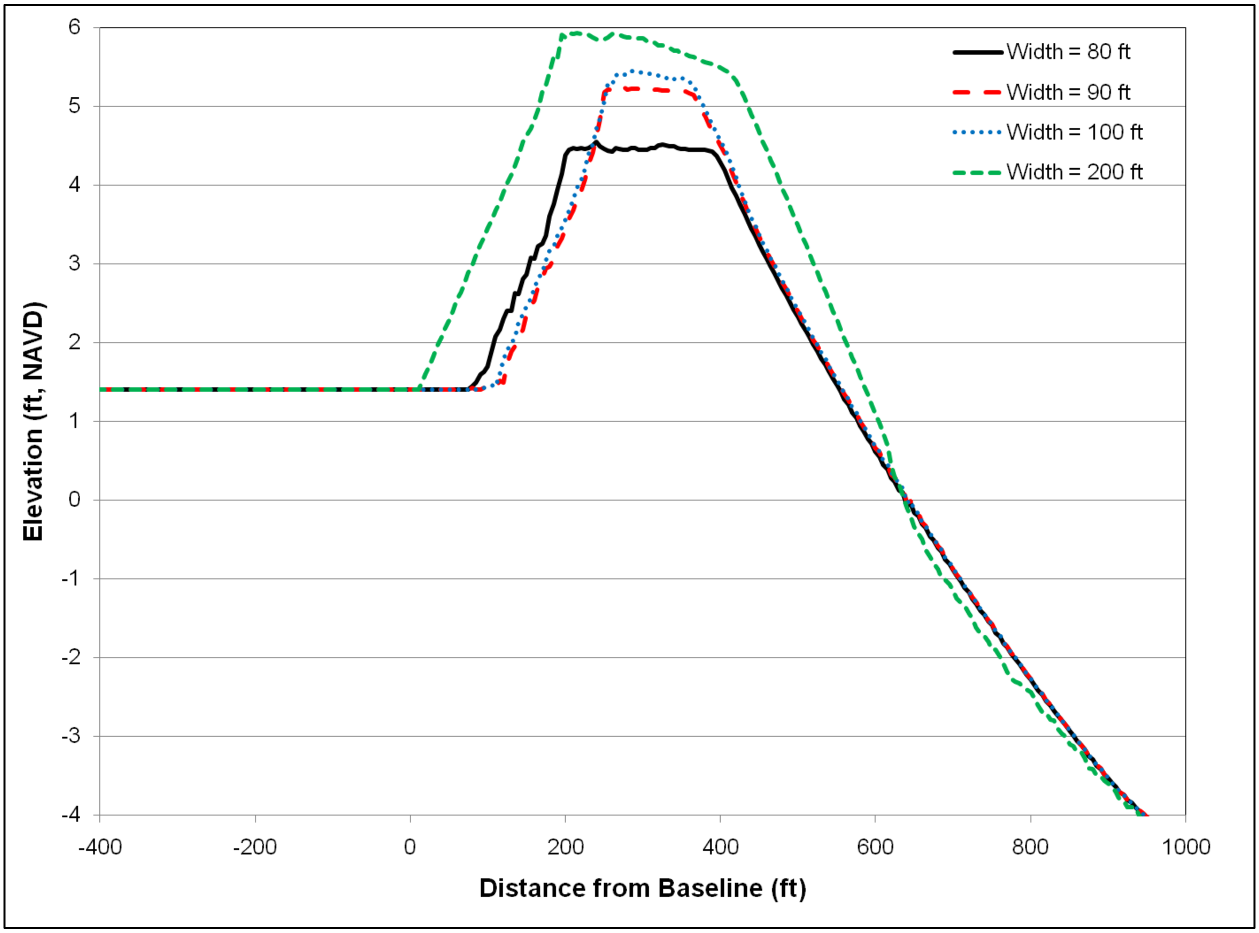


Figure 37. SBEACH Output for Various Dune widths for a +8 feet, NAVD Constructed Dune following a 10-year storm in TY7.

15.4 Post-Construction Profile Equilibration

It is expected that the constructed beach template will readjust to an equilibrium beach profile in the year following construction. The equilibration process assumes that there is only cross-shore redistribution of sediment and the sand volume is conserved. Below mean high water, the profile is anticipated to approximate Dean's (1987) equilibrium beach profile of the form described in Equation 5. Above mean high water the seaward dune crest is expected to translate such that the sand volume is conserved.

$$y = Ax^{2/3} \qquad \text{[Equation 5]}$$

where:

y = depth below mean high water (ft)
$A = 0.101\ (ft^{1/3})$ (Dean's empirically derived coefficient for 0.11 mm grain size)
x = cross-shore distance from mean high water (ft)

The constructed beach slope is 1V:30H from the dune crest to +1.0 feet, NAVD and 1V:60H from +1.0 feet, NAVD to the construction toe of fill. For a 1V:60H offshore slope, it is anticipated that the mean high water shoreline will retreat due to profile equilibration as the offshore slope of the construction template is slightly steeper than the slope of Dean's (1987) equilibrium beach profile for 0.11 mm sand. The September 2010 profile, construction template, and equilibrium beach profile at Station 50+00 for a +8.0 feet, NAVD constructed dune elevation are shown in Figure 38. Under the assumption that sand volume is conserved, the seaward dune crest is expected to translate approximately 38 feet landward during the equilibration process.

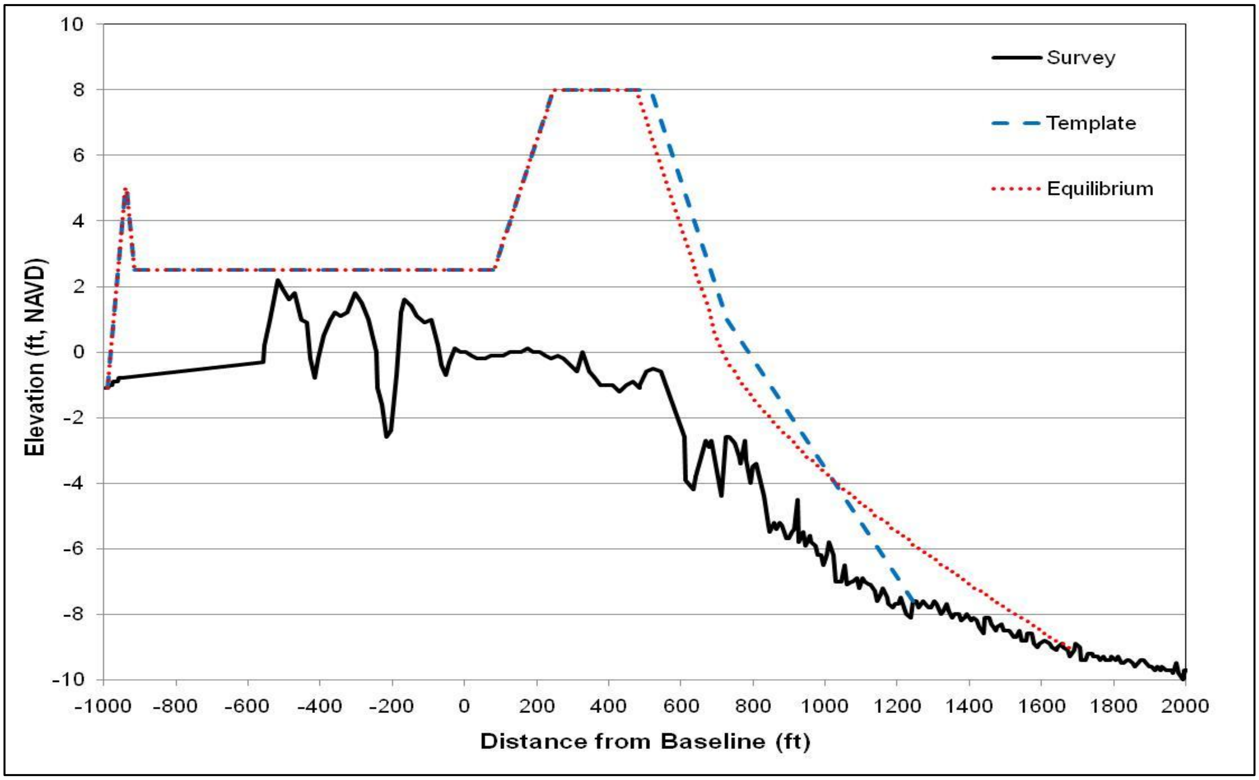


Figure 38. Beach Template Equilibration

15.5 Advanced Fill

Advanced fill is the sacrificial portion of the beach cross-section that protects the design cross-section. Advanced fill is placed during construction such that the design cross-section is protected for a pre-determined time period. The initially placed advanced fill does not necessarily have to protect the design cross-section for the project life as it can be replaced during nourishment events or the project can be designed to meet different objectives. CWPPRA projects require a one-time construction event, therefore the option of nourishment events will not be considered.

The advanced fill placed must account for all expected losses. Expected losses include longshore, relative sea level rise, overwash, offshore (silt), and settlement. These losses, or components of the advanced fill, are discussed in detail in the following sections.

15.5.1 Longshore Losses

The advanced fill volume required to counteract the effects of longshore transport was calculated using the longshore transport rates discussed in Section 13.4. Variations in the longshore transport rate are expected to be similar to those observed along the existing island as the constructed shoreline is parallel to the existing shoreline. Transport off the west end of the island was estimated to be approximately 39,000 cubic yards/year while transport entering the system is approximately 400 cubic yards/year. Therefore, the advanced fill volume required to offset losses due to longshore transport is approximately 38,600 cubic yards/year. Note that if the longshore transport onto the island is underestimated that the total loss value remains the same as the loss of the west end will simply increase accordingly.

15.5.2 Relative Sea Level Rise Losses

A portion of the shoreline recession is due to relative sea level rise. Shoreline recession due to relative sea level rise does not result in a net volume change in the cross-shore profile but simply a redistribution of the sediment across the profile (Figure 30). However, sand must still be added to the system to counteract the dry beach/dune loss caused by relative sea level rise.

The post-construction beach is expected to have a steeper beach profile than the pre-construction beach as there will be a higher percentage of sand in the fill. A steeper beach profile will reduce shoreline recession attributed to relative sea level rise (Equation 3). However, the existing shoreline recession rates due to relative sea level rise were used when calculating advanced fill requirements, as this gives a more conservative result. Therefore, the advanced fill volume required to counteract the effects of relative sea level rise is approximately 9,200 cubic yards/year.

15.5.3 Overwash

Analysis of recent shoreline changes, as discussed in Section 10, suggests that the dune will migrate landward due to overwash. The backing marsh was designed to help maintain island elevation and provide a platform for the dune when rolling over. Sediment deposited on the backing marsh during overwash events provides additional elevation and volume for the dune when rolling over. Therefore, overwash was not included in the advanced fill as the volume is conserved within the system. Overwash was assumed to affect project performance with respect to acreage calculations.

15.5.4 Offshore Losses

The beach fill borrow area contains both sand and silt. While some of the silt will be washed out during construction, some will remain within the beach fill. As the island retreats the beach profile will recede and silts will be suspended by wave action and moved offshore, concentrating sand along the front face of the island (Campbell, 2005). Therefore, the fill volume must be increased to account for the silt lost offshore.

The beach fill borrow areas contain between 15 and 28% percent silts (Section 7). However, the percent silt measured from the vibracores does not give an accurate representation of what will be placed on the beach as a portion of the fine material is expected to wash out during fill placement. It was assumed that the beach fill will contain 10% silt following construction. Therefore, the advanced fill volume required to counteract the effects of offshore losses is approximately 5,300 cubic yards/year.

15.5.5 Settlement Losses

The advanced fill volume required to counteract the effects of settlement was calculated using the settlement curves provided by Fugro (Appendix D). The settlement loss can be viewed as a change in shoreline due to the lowering of the dune profile. The shoreline loss due to settlement was calculated by multiplying the dune settlement by the subaerial (above +1.0 feet, NAVD) slope of 1V:30H (i.e. for every 1 foot of settlement, the shoreline is expected to retreat 30 feet). Consequently, the advanced fill volume can be calculated by multiplying this shoreline change by the active profile height and the longshore distance. As the active profile height and settlement rate decrease, the settlement loss also decreases. Table 28 provides settlement losses for the +8 feet, NAVD dune constructed along 8,000 feet of shoreline with an active profile extending to -6 feet, NAVD.

Table 28. Anticipated Settlement Losses for a +8 feet, NAVD Constructed Dune

Years After Construction	Settlement (ft/yr)	Shoreline Change (ft/yr)	Active Profile Height (ft)	Settlement Loss (cy/yr)
1	1.2220	36.7	12.6	137,321
2	0.2380	7.1	12.4	26,177
3	0.1250	3.8	12.2	13,576
4	0.1250	3.8	12.1	13,404
5	0.1250	3.8	11.9	13,231
6	0.0698	2.1	11.8	7,326
7	0.0698	2.1	11.5	7,132
8	0.0698	2.1	11.4	7,076
9	0.0698	2.1	11.3	7,021
10	0.0698	2.1	11.2	6,965
11	0.0408	1.2	11.2	4,048
12	0.0408	1.2	11.1	4,024
13	0.0408	1.2	11.0	4,001
14	0.0408	1.2	9.2	3,342
15	0.0408	1.2	9.2	3,331
16	0.0236	0.7	9.2	1,921
17	0.0236	0.7	9.1	1,914
18	0.0236	0.7	9.1	1,908
19	0.0236	0.7	9.1	1,902
20	0.0236	0.7	9.0	1,895
			Total	267,516

15.6 Beach Fill Design Options

Three beach fill design options are discussed in the following sections. All beach fill design options were developed using variable shoreline retreat rates due to a decreasing active profile height caused by settlement of the underlying soils. Table 29 provides future with project shoreline change estimates calculated using advanced fill losses along 8,000 feet of shoreline and a variable active profile height for a dune constructed to +8 feet. A detailed discussion of these shoreline losses is provided in Section 15.5.

Table 29. Shoreline Change Estimates for a +8 feet, NAVD Constructed Dune

Target Year	Dune Elevation (ft, NAVD)	Active Profile Height (ft)	Longshore Loss (ft/yr)	Offshore Loss (ft/yr)	Overwash (ft/yr)	Relative Sea Level Rise (ft/yr)	Settlement Loss (ft/yr)	Total Loss (ft/yr)
1	8.0	13.9	7.5	1.1	0.0	2.2	0.0	10.8
2	6.8	12.6	8.2	1.2	0.0	2.4	36.7	48.5
3	6.5	12.4	8.4	1.2	0.0	2.5	7.1	19.3
4	6.3	12.2	8.5	1.2	0.0	2.5	3.8	16.0
5	6.2	12.1	8.6	1.3	0.0	2.6	3.8	16.2
6	6.0	11.9	8.8	1.3	0.0	2.6	3.8	16.4
7	5.9	11.8	8.8	1.3	0.0	2.6	2.1	14.8
8	5.6	11.5	9.1	1.3	0.0	2.7	2.1	15.2
9	5.6	11.4	9.1	1.3	0.0	2.7	2.1	15.3
10	5.5	11.3	9.2	1.3	0.0	2.7	2.1	15.4
11	5.4	11.2	9.3	1.3	0.0	2.7	2.1	15.5
12	5.3	11.2	9.3	1.4	0.0	2.8	1.2	14.7
13	5.3	11.1	9.4	1.4	0.0	2.8	1.2	14.8
14	5.2	11.0	9.4	1.4	0.0	2.8	1.2	14.8
15	3.4	9.2	11.3	1.6	4.0	3.3	1.2	21.5
16	3.4	9.2	11.3	1.6	4.0	3.4	1.2	21.5
17	3.4	9.2	11.4	1.7	4.0	3.4	0.7	21.1
18	3.3	9.1	11.4	1.7	4.0	3.4	0.7	21.2
19	3.3	9.1	11.5	1.7	4.0	3.4	0.7	21.2
20	3.3	9.1	11.5	1.7	4.0	3.4	0.7	21.3
		Total	**192.2**	**27.9**	**23.9**	**56.8**	**74.5**	**375.3**

15.6.1 Beach Fill Design Option 1

Beach Option 1 was developed by adding equilibrium losses and seven years of advanced fill to the design cross-section. Furthermore, the location of the constructed shoreline was designed such that the projected 2014 shoreline would not be exposed to the Gulf over the 20-year project life. This design results in a dune with a construction elevation of +8 feet, NAVD, a width of

270 feet, and a shoreline located 413 feet seaward of the 2014 shoreline. The total volume required to construct Beach Option 1 is 1,830,000 cubic yards. This is composed of 1,300,000 cubic yards to construct the design section and 530,000 cubic yards to construct the advance fill section.

Since the design volume contains more volume than the expected losses over the 13 year project life remaining after TY7, the beach at the TY20 should be more robust that the pre-construction condition.

The intent of the two section beach fill (design section and advanced fill) is that the design section should not be exposed and that the advanced fill is replaced when the design section becomes exposed. This is not the intent of beach option 1. The size of the dune has been designed to meet a design objective in TY7 though the project life is 20 years. Therefore, the design section only needs to be protected through 7 years of the 20-year project life.

15.6.2 Beach Fill Design Option 2

Beach Option 2 was developed by holding the landward toe of fill and matching the fill volume of Beach Option 1, but for a dune constructed with a crest elevation of +6 feet, NAVD. This allows direct comparison of a +8 and +6 dune elevation with respect to project performance. This design results in a dune with a construction elevation of +6 feet, NAVD, a width of 445 feet, and a shoreline located 468 feet seaward of the 2014 shoreline. The total volume required to construct Beach Option 2 is 1,840,000 cubic yards. The small difference in volume (10,000 cubic yards) between the two beach options is a rounding issue with the difference in shoreline position being only 0.2 feet.

15.6.3 Beach Fill Design Option 3

Beach Option 3 was developed by holding the landward toe of fill and using only advanced fill volume requirements. The advanced fill volume required for longshore, offshore, and relative sea level rise losses is 1,062,000 cubic yards (53,100 cy/yr x 20 yr = 1,062,000 cy). The advanced fill volume required for settlement losses for a +8 feet, NAVD constructed dune is 268,000 cubic yards. Therefore, the advanced fill volume requirement for a +8 foot dune over the 20-year project life is 1,330,000 cubic yards. This results in a dune with a construction elevation of +8 feet, NAVD, a width of 150 feet, and a shoreline located 293 feet seaward of the 2014 shoreline. Again, rounding to a more constructible beach width results in a small change in volume. The total volume required to construct Beach Option 3 is 1,310,000 cubic yards.

16 MARSH FILL DESIGN

This section discusses the development of the marsh design. The design elevation was developed based on the predicted elevation of the fill material over the life of the project, the rate of relative sea level rise, and construction templates used for previously constructed projects to achieve a productive marsh over the life of the 20-year project. A full discussion of the geotechnical analysis of the underlying soils is included in Appendix D.

Fill material used to construct the marsh will be dredged from the D-1 and Quatre Bayou borrow areas (See Section 7 for borrow area location map). The D-1 borrow area contains a mix of sand and fine grain material (48% fine grain material), which is suitable sediment for marsh creation. The D-1 borrow area contains approximately 1,472,000 cubic yards of marsh fill material, while Quatre Bayou provides an additional 5,088,000 cubic yards of material.

16.1 Existing Marsh

The average elevation of the existing marsh on the island, as surveyed by John Chance Land Surveys in August-October 2010, is approximately +1.0 feet, NAVD. This is comparable to the elevation of other marsh platforms in the area such as East Grand Terre, Chaland Headland, and Pelican Island. The present mean high water and mean low water elevations are +0.95 and -0.27 feet, NAVD, respectively, as discussed in Section 8.1.

16.2 Elevation Loss

The loss in elevation of the marsh platform is a function of three processes:

1. Geologic subsidence of the region (estimated at 0.0247 feet/year, Section 9).
2. Consolidation of the marsh fill material.
3. Settlement of the underlying soils due to the load of the marsh fill material.

Geologic subsidence is independent of the project and will occur under both with and without project conditions. For the purposes of design and analysis, the rate of subsidence was considered constant over the 20-year project life with a value of 0.0247 feet/year. A detailed discussion of subsidence is provided in Section 9.

The extent of marsh fill consolidation is a function of the properties of the underlying soils, borrow material (sand, silt, clay, and organic content), lift thickness, and placement techniques (i.e. dredge pipe discharge velocity, duration, containment, and cell capacity). The settlement of the marsh fill is mostly due to consolidation of the marsh fill material. The calculated marsh elevation also accounts for the settlement of the underlying soils due to loading by the placed marsh fill.

The soil underlying the fill material will consolidate due to the addition of overburden (the constructed marsh and beach fill). Consolidation of the underlying soils is a function of the consolidation properties, the area of the applied load, and the thickness of fill above the water table (surcharge load). It is also expected that consolidation of the underlying soils will vary due to differences in the lift thickness, variations in the properties of the marsh fill material, and differences in the project footprint. A settlement analysis of the consolidation of the underlying soils was completed by Fugro Consultants, Inc. and is included in Appendix D. This analysis ignored secondary compression, desiccation, and evaporation of water from the fill material because these factors have limited effect on fill consolidation when compared to other variables.

Settlement curves were developed for varying marsh construction elevations, assuming that marsh fill would be constructed using material similar to that within the West Belle Pass Barrier Headland Marsh (TE-52) fill borrow areas. Data sufficient to develop settlement curves using material from the D-1 or Quatre Bayou borrow areas was not available so consolidation curves previously developed by Fugro Consultants, Inc. based on the expected marsh fill material for the West Belle Pass Headland Restoration (TE-52) project (Thomson et al., 2009) were employed. Although the fill material analyzed for the West Belle Pass Barrier Headland project may vary from that of the D-1 or Quatre Bayou borrow areas, it provides an approximation of expected settlement. The average bottom elevation within the marsh fill area is approximately at 0.0 feet, NAVD. The curves from West Belle Pass were linearly extrapolated based on the lift thickness of the constructed marsh for the current project compared to that assumed by Fugro for West Belle Pass.

Marsh platform accretion will be the product of many environmental factors and is difficult to predict. These factors include detritus accumulation, plant productivity, and belowground (i.e. root) production. Fitzgerald *et al* (2003) estimated detritus accumulation rates of between 3 and 9 mm/year. A detritus accumulation rate of 0.0098 feet/year (3 mm/year) was assumed to be conservative and was included in the marsh settlement analysis and in the WVA performance projections beginning in TY5. The accretion rate is smaller than the consolidation and settlement rates and thus did not affect the marsh elevation analysis throughout the 20-year project life.

16.3 Marsh Elevation

A variety of marsh fill construction elevations were considered. Selection of the construction template elevation is based on consideration of the average marsh elevation over the life of the project with respect to intended functioning of the marsh from both a habitat stand point and meeting the project goal and objectives. One element of the marsh design is to maximize the time period that the marsh platform has an elevation in the bay intertidal zone for the wetland value assessment (WVA). The bay intertidal zone, as defined for the WVA, is between 0.0 and +2.0 feet, NAVD and is independent of the tide range expected at the project site. Previous marsh creation projects (CPE, 2003 and CPE, 2005) applied an even tighter marsh elevation range based on maximizing the time period that the marsh platform remained between mean high water and mean low water. Over the 20-year project life, with a construction completion date of 2014 and a eustatic sea level rise rate of 0.0056 feet/year (as discussed in Section 9), the mean high water and mean low water elevations are expected to rise from +0.97 to +1.07 feet, NAVD and from -0.24 to -0.14 feet, NAVD, respectively. As mentioned in Section 8.1 however, it appears that we are currently in the portion of the tidal epoch where the water level is higher than normal. The average mean high water level in the project area as measured using the CRMS station is approximately 1.3 feet, NAVD.

Figure 39 shows the marsh settlement curves for various construction elevations. A marsh fill template elevation of +2.5 feet, NAVD was selected for all Chenier Ronquille marsh options based on the marsh settlement analysis of the underlying marsh material performed by Fugro Consultants (Appendix D) and the properties of the marsh fill material reported for the East Grand Terre Island Restoration Project (BA-30) (CPE, 2004). Although this elevation will result in a marsh platform above MHW for much of the project life (Figure 39), back-barrier marshes on similar projects have successfully been implemented at this elevation.

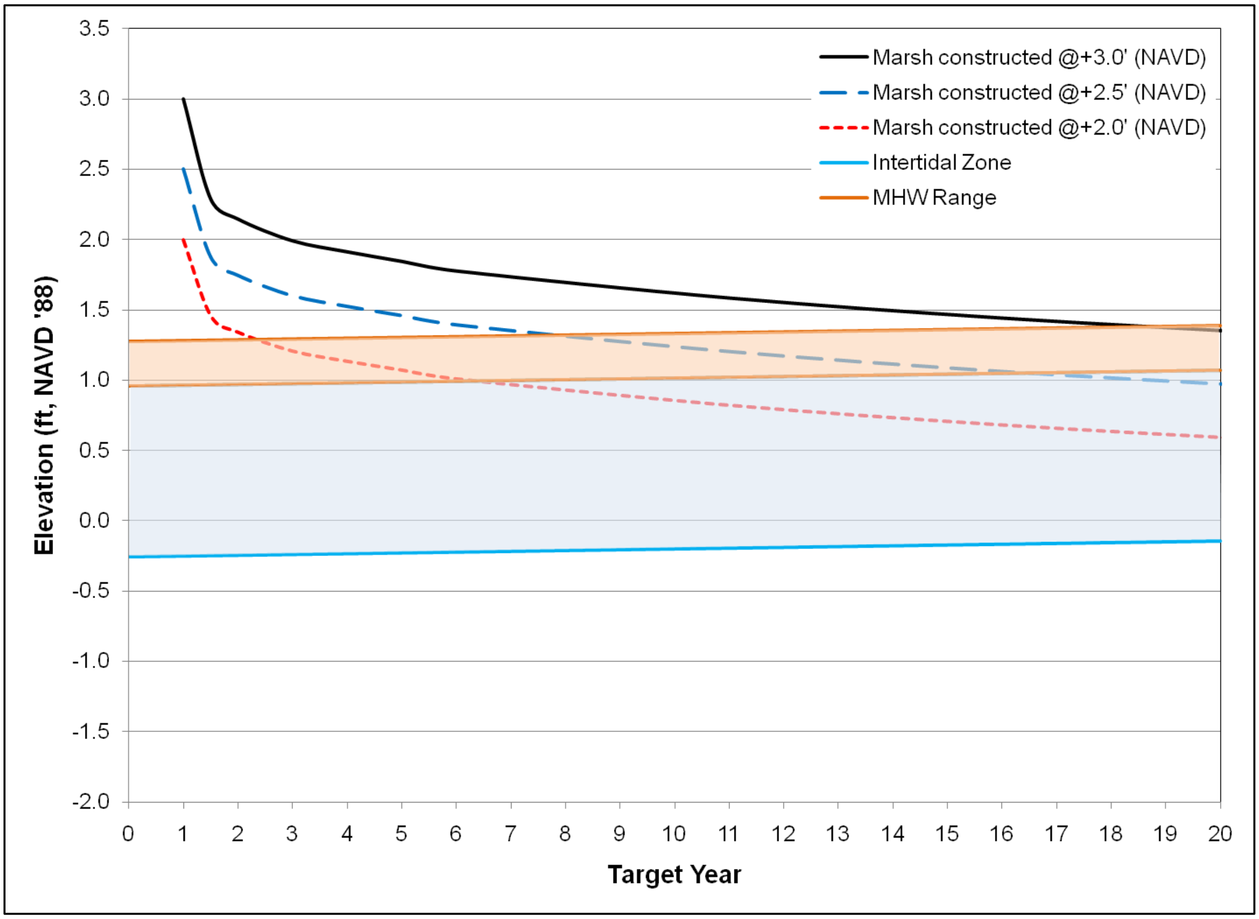


Figure 39. Marsh Elevation Settlement Curves

In comparison, the Chaland Headland Restoration Project (BA-38-2) had a marsh template elevation of +2.5 feet, NAVD while the East Grand Terre Island Restoration Project (BA-30) had a marsh template elevation of +2.3 feet, NAVD. As with these projects, it is proposed to survey the marsh fill elevation 30 days after fill has been placed to account for settlement of the material during the consolidation period.

The marsh fill is expected to consolidate approximately 0.5 feet in just under a year so the marsh is considered to be bay intertidal habitat (defined by the WVA) by TY2. The marsh elevation is below the upper range of the mean high water elevation by TY8. The marsh elevation remains above mean low water for the duration of the project life.

16.4 Marsh Fill Design Options

Numerous marsh fill planforms were considered during development of the Phase 0 report (Section 14). After conducting oil and gas infrastructure investigations (as recommended in the Phase 0 report), the access channel proposed in the Phase 0 report had to be relocated due to pipelines with only 4 to 5 feet of cover. The beach and marsh footprints were also shortened on the west end of the island in all alternatives to reduce volume and cost in light of on-going erosion and anticipated 2014 conditions. This section discusses the four marsh fill planforms that incorporate the latest survey data, design data, and cost considerations.

The southern extent of the marsh fill is bounded by the beach fill. The remainder of the marsh fill is bounded by the primary dike. One of the key design factors for Chenier Ronquille restoration is the multitude of pipelines that cross the project area and limit access. The Columbia Gulf pipeline in the project area has approximately 14 feet of cover, so dredging could be considered in the vicinity of the pipeline. This has been successfully employed on other Louisiana barrier island projects such as Chaland Headland and Bay Joe Wise. Other pipelines in the area have less cover (2-3 feet). Crossing or excavating near the shallow pipelines will be avoided.

16.4.1 Marsh Fill Design Option 1

The footprint of Marsh Option 1 was altered from the marsh footprint proposed in the Phase 0 report. The northern extent of Marsh Option 1 was shifted north to expand the marsh footprint and avoid oil and gas infrastructure while dredging the access channel. The 2010 survey showed that the water depths within the marsh footprint were shallower than estimated within the Phase 0 report, which allowed the marsh fill footprint to be expanded north without negatively impacting the marsh fill cost. Combined with a realignment of the access channel extending to Quatre Bayou Pass, this alignment also avoided crossing any oil infrastructure.

The access channel is located between the Plains and Columbia Gulf pipelines. The primary dike for Marsh Option 1 is north of the access channel and would be constructed on top of the Plains pipeline. With the access channel entering the project area at the western end of the island, no pipelines would have to be crossed. The access channel would be backfilled with marsh fill material as part of Marsh Option 1. Plan views of the various options are included in the next section (Section 17), which discusses the development of alternatives from the beach and marsh options.

The marsh fill footprint is 274 acres. The marsh width varies from 560 feet at the western extent to 1,990 feet at the eastern extent and has an average width of approximately 1,280 feet.

The volume required to construct the primary dikes is approximately 124,200 cubic yards (measured in place). Construction of the primary dikes will result in loss of the dredge material due to suspension losses. Assuming a 1.5:1 cut to fill ratio implies that approximately 186,300 cubic yards will be excavated to construct the primary dike. Since the access channel is located within the marsh footprint, an additional 186,300 cubic yards will have to be pumped from the borrow area to fill the access channel.

Overwash from the beach footprint into the marsh footprint will reduce the volume of required marsh fill volume by 13,500 cubic yards/year. Assuming construction in 2014, overwash reduces the marsh fill volume, estimated using the 2010 survey, by 54,000 cubic yards.

After accounting for overwash and the volume required to refill areas excavated to construct the dike, the total marsh fill volume required to construct marsh option 1 is 1,380,000 cubic yards

No renourishment is planned for the marsh during the 20-year project life.

16.4.2 Marsh Fill Design Option 2

Marsh Option 2 uses the same access channel layout as Marsh Option 1, but the primary dike is constructed to the south of the channel instead of to the north. The primary dike will be constructed on top of the Columbia Gulf pipeline. Constructing the primary dike to the south results in a smaller marsh footprint and does not require backfilling of the access channel.

The marsh width where it borders the open bay varies from 270 feet at the western extent to 1,700 feet at the eastern extent and has an average width of approximately 990 feet. The Marsh Option 2 footprint is approximately 220 acres.

The volume required to construct the primary dikes is approximately 127,300 cubic yards. The access channel will not be backfilled during construction, so only areas excavated within the marsh footprint to construct the dike along the east and west boundaries of the marsh will be backfilled. Assuming a cut-to-fill ratio of 1.5:1, the volume of marsh fill required to fill the excavated areas providing the primary dike fill is approximately 34,700 cubic yards.

The total volume (in place) required to construct Marsh Option 2 is 940,000 cubic yards. This includes a 13,500 cubic yard/year overwash benefit as with Marsh Option 1 and backfilling areas excavated within the marsh footprint to build the primary dikes.

16.4.3 Marsh Fill Design Option 3

Of the four marsh options presented in this report, Marsh Option 3 is the most similar footprint to the one proposed in the Phase 0 report. Marsh Option 3 has been altered from the Phase 0 preferred option in response to additional oil and gas infrastructure information. The access channel enters from Quatre Bayou Pass, south of the 12” Plains pipeline and north of the Columbia Gulf pipeline. The access channel is oriented due east and crosses the southwest-northeast oriented Columbia Gulf pipeline in the center of the island. The proposed alignment of the access channel requires that the Contractor cross the Columbia Gulf pipeline. Probing of the pipelines suggests that there is approximately 14 to 15 feet of cover at the location where the access channel is proposed to cross the pipeline.

The primary dike is located to the north of the access channel for Marsh Option 3. Since the primary dike is north of the access channel, the access channel will be backfilled with marsh fill material.

The marsh width varies from 560 feet at the western extent to 1,550 feet at the eastern extent and has an average width of approximately 1,060 feet. The Marsh Option 3 footprint is approximately 214 acres.

The volume required to construct the primary dikes is approximately 105,300 cubic yards. Assuming a cut to fill ratio of 1.5:1 to construct the dike, the volume excavated to construct the primary dikes is approximately 157,900 cubic yards. This volume must be included in the volume of marsh fill to be dredged from the borrow area because the access channel is within the marsh footprint.

The total volume (in place) required to construct Marsh Option 3 is 1,020,000 cubic yards. This includes the 13,500 cubic yards/year benefit from overwash and backfilling the excavated areas within the marsh footprint.

16.4.4 Marsh Fill Design Option 4

The access channel for Marsh Option 4 is identical to that for Marsh Option 3. However, the primary dike for Marsh Option 4 is located south of the access channel, resulting in a smaller marsh footprint compared to Marsh Option 3.

The marsh width varies from 210 feet at the western extent to 1,200 feet at the eastern extent and has an average width of approximately 710 feet. The Marsh Option 4 footprint is approximately 150 acres.

The volume required to construct the primary dikes is approximately 81,400 cubic yards. The access channel is outside the marsh footprint and will not be backfilled during construction. Only areas excavated within the marsh footprint to construct the dike along the east and west boundaries of the marsh will be backfilled. Assuming a cut to fill ratio of 1.5:1 to construct the dike, the volume excavated from within the marsh footprint that will be backfilled is approximately 18,700 cubic yards. This volume is a component of the total marsh fill volume.

The total volume (in place) required to construct Marsh Option 4 is 590,000 cubic yards. This includes the beneficial overwash volume and backfilling excavated areas within the marsh footprint.

16.5 Primary Dike

A primary dike elevation of +5 feet, NAVD with side slopes of 1V:8H are proposed. This slope is based on observations of the primary dikes constructed at the Chaland Headland. At East Grand Terre, the contractor constructed a primary dike with side slopes of approximately 1V:6H. It was assumed for construction purposes that the primary dikes will be constructed to +5.0 feet, NAVD with IV:8H front and back slopes.

Fugro performed a dike stability analysis assuming that the in situ material would be used to construct the primary dikes. A crest elevation of +5 feet, NAVD, crest width of 10 feet, side slopes of 1V:4H, and a freeboard of 2.5 feet were assumed. Potential failure surfaces initiating in the marsh and undermining the containment dike were evaluated. Dike cross sections with an excavated channel located adjacent to the toe of the dike (fill source for primary dike construction) were also evaluated for potential failures. The factor of safety for all of the scenarios was greater than 1.3. A constructed side slope of 1V:8H rather than 1V:4H as used in the analysis should increase the factor of safety. The details of the primary dike stability analysis are included in Appendix D.

Borings B5, B6, and B8 were not included in the primary dike analyses, as Fugro determined the high percentage of organic material to be not suitable for construction. Therefore, constructability issues may arise for marsh options 3 and 4, as the proposed primary dikes for these options are located near borings B5 and B6. The contractor will be required to use alternative methods of containment such as sheeting in problematic areas if necessary. The proposed primary dikes for marsh options 1 and 2 are not located near these areas.

It is more likely that the dikes will fail due to wave action since the north side of the marsh is exposed to waves forming and propagating across Bay Long, especially during the winter and fall months. The Contractor should expect the dikes to require maintenance during the project.

Fugro performed a settlement analysis on the primary dike and determined that consolidation of the dike material will occur during construction and is considered negligible. Based on a fill thickness of 5.0 feet, the expected settlement of the underlying soils throughout the project life was estimated. Differential settlement is expected to occur within the access channel that will be backfilled with hydraulically pumped material for marsh options 1 and 3. A settlement analysis was performed for the channel given anticipated borrow material properties reported for the East Grand Terre Island Restoration Project (BA-30) (CPE, 2004) and Fugro's analysis of the settlement of the underlying soils. Since the channel will be excavated to -7.0 feet, NAVD and filled to +2.5 feet, NAVD, a fill thickness of 9.5 feet was used to determine the self-weight consolidation. Fill material is expected to consist of approximately 48% fine grain material, which will contribute to self weight consolidation. The remaining sand fraction will not be a factor in self weight consolidation. The self weight consolidation for the backfilled channel was based on this grain size ratio and combined with Fugro's underlying soils' settlement projection. The material placed in the access channel will settle at a higher rate than the rest of the marsh platform and primary dike but should stay in intertidal range throughout the life of the project.

The boundary for marsh fill payment will be at the intersection of the marsh fill and primary dike. This will allow the shape and size of the primary dike to be at the discretion of the Contractor. The cost of the primary dike can then be paid on a per linear foot basis. The cost of refilling the access channel with marsh material from the borrow area (marsh options 1 and 3) is included in the marsh fill cost.

16.6 Marsh Fill Design Summary

Each footprint was refined from the Phase 0 concept to incorporate the most up to date survey and oil and gas infrastructure data. Four marsh options with varying acreages are proposed as components of Alternatives 1 to 6. Two options backfill the access channel with marsh fill material. A fill elevation of +2.5' NAVD is proposed for each option, as this will result in the optimum bay intertidal range through the majority of the project life. Marsh Options 3 and 4 will require dredging of the access channel across the Columbia Gulf Pipeline.

17 DEVELOPMENT OF ALTERNATIVES

Once the various beach and marsh options were established, alternatives were developed that combined these options to provide a range of performance and construction costs. Out of twelve possible alternatives that combine the three beach and four marsh options, six were selected to represent a diverse array of alternatives for evaluation. Alternatives not selected were closely related to the final six or were not sufficient to meet the project goal. Details regarding alternatives not considered may be found in Section 17.7.

Figure 40 through Figure 45 show the plan views of the six selected alternatives, while Table 30 summarizes the beach and marsh design options used to develop the alternatives. Cross-sections of the selected alternatives are available in Appendix A.

Table 30. Summary of Alternative Development Details

Selected Alternative	Fill Volume (cy)		Design Option		Construction Footprint (acres)
	Beach	Marsh	Beach	Marsh	
1	1,830,000	1,380,000	1	1	437
2	1,830,000	940,000	1	2	381
3	1,830,000	590,000	1	4	311
4	1,840,000	940,000	2	2	394
5	1,310,000	1,380,000	3	1	411
6	1,310,000	1,020,000	3	3	350

17.1 Alternative 1

Alternative 1 combines Beach Option 1 and Marsh Option 1. Beach Option 1 is the preferred option since it has the largest volume and a crest elevation of +8' NAVD. This beach option was combined with Marsh Option 1, which is the largest marsh option. This provides the largest construction footprint and volume of any alternative (Figure 40). This is the recommended alternative because it follows the standard engineering design of having both a design section and an advanced fill section and has the widest backing marsh to prevent breaching over the 20-year project life.

Alternative 1 requires 1,830,000 cubic yards of beach fill and 1,380,000 cubic yards of marsh fill. The construction footprint (above 0 feet, NAVD) is 437 acres.

17.2 Alternative 2

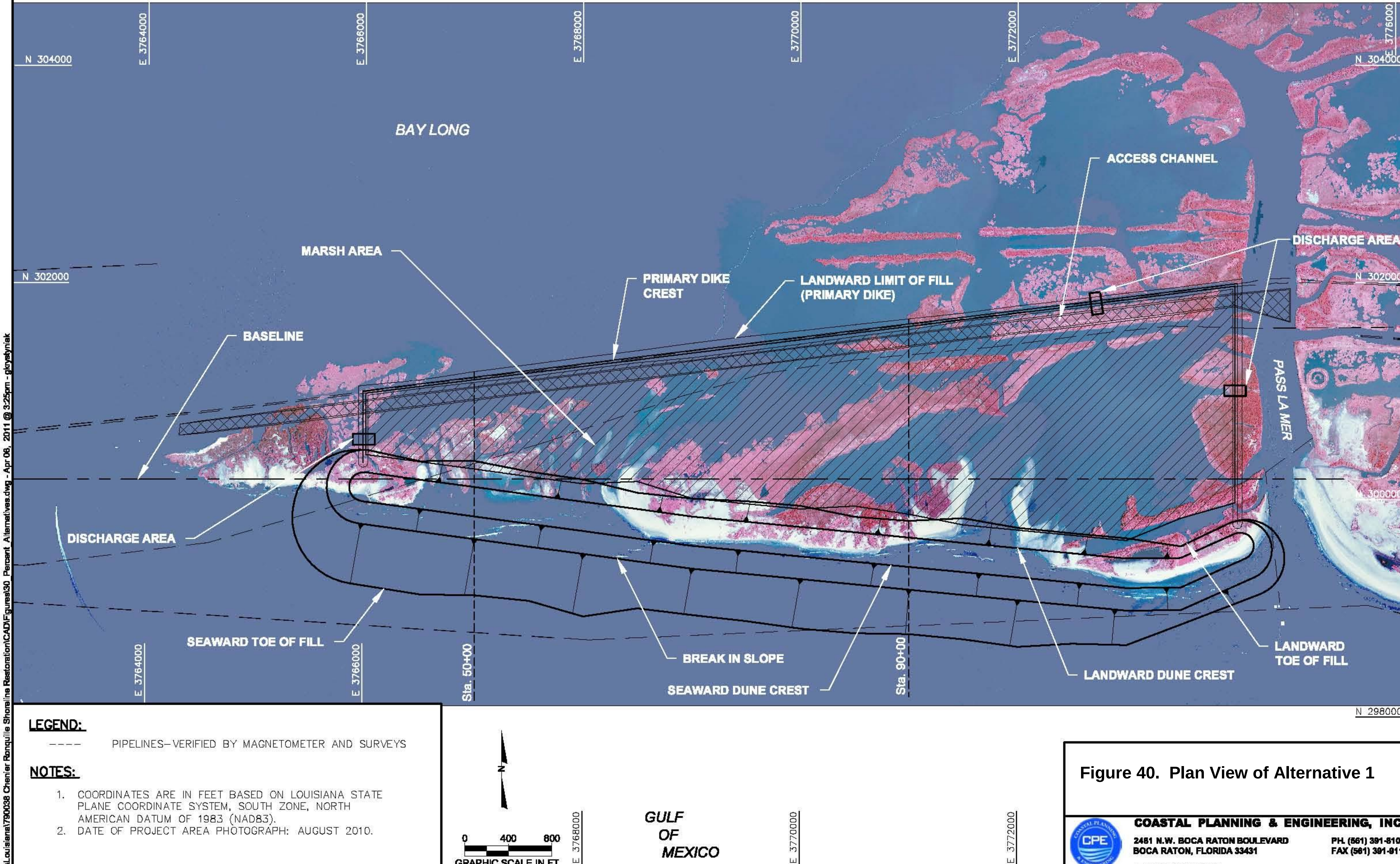
Alternative 2 combines Beach Option 1 and Marsh Option 2 (Figure 41 and Table 30). This alternative was developed to compare the cost and performance impacts of relocating the primary dike further south to avoid the Columbia Gulf pipeline but using the same access channel as in Alternative 1. Relocating the primary dike south results in a cost savings because less fill is needed to fill the marsh template and there is no backfilling of the access channel. The beach volume remains the same as in Alternative 1 (1,830,000 cubic yards) but the marsh fill volume drops to 940,000 cubic yards.

Alternative 2 has a construction footprint of 381 acres above 0 feet, NAVD.

17.3 Alternative 3

Alternative 3 was designed to incorporate the preferred beach option (Beach Option 1), meeting the standard coastal engineering design process, while minimizing cost by pairing it with the smallest marsh option (Marsh Option 4). This alternative was developed to resemble the Phase 0 preferred marsh layout with the preferred beach option. Coupled with Alternatives 1 and 2, Alternative 3 can highlight the possible range of performance and costs. The beach volume is 1,830,000 cubic yards while the marsh fill volume has decreased further to 590,000 cubic yards, the lowest marsh volume in any alternative.

Alternative 3 has a subaerial footprint of 311 acres (Figure 42). This is the smallest footprint of the six alternatives.



LEGEND:

---- PIPELINES–VERIFIED BY MAGNETOMETER AND SURVEYS

NOTES:

1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
2. DATE OF PROJECT AREA PHOTOGRAPH: AUGUST 2010.

Figure 40. Plan View of Alternative 1

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PH. (561) 391-8102
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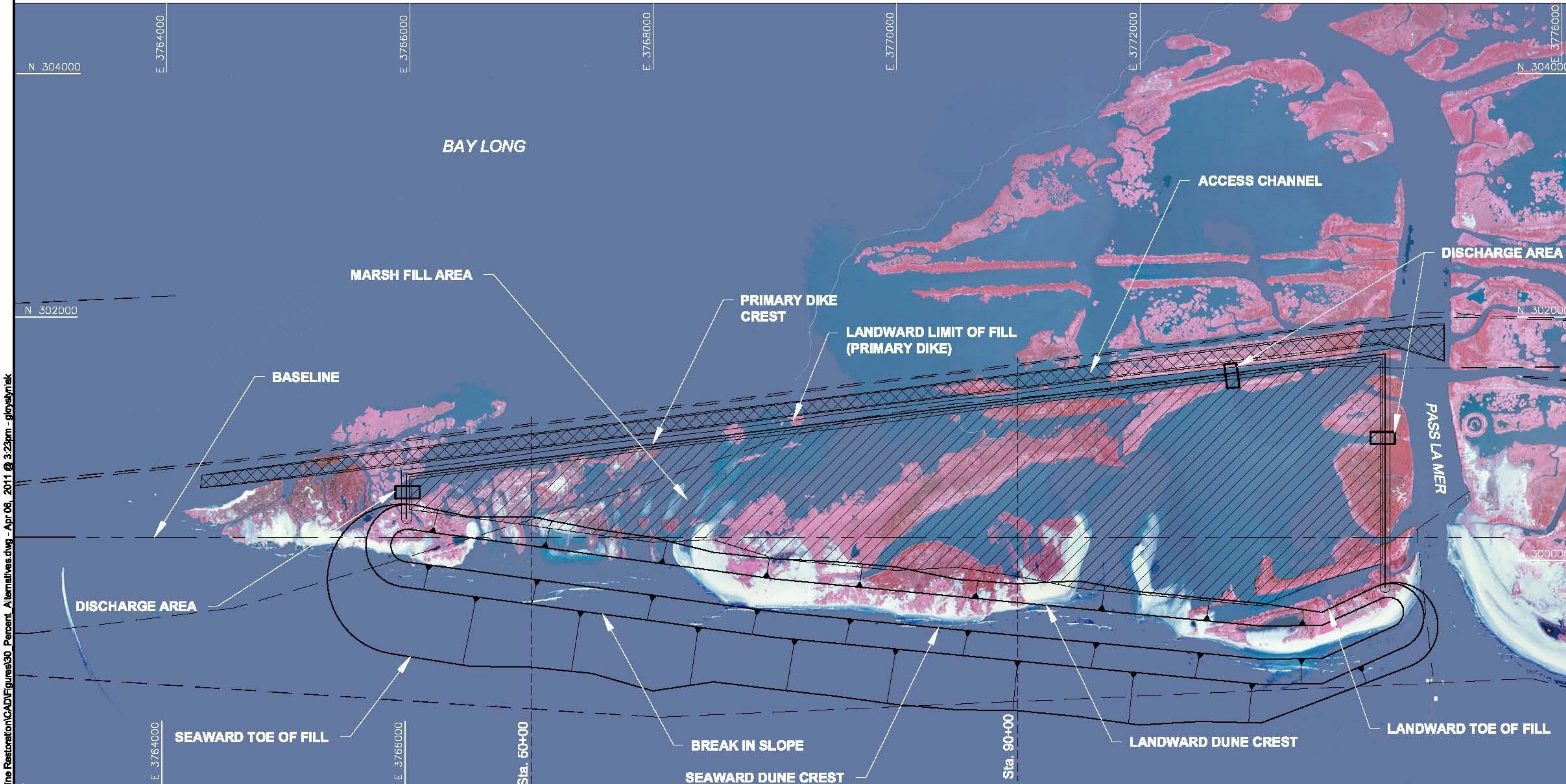

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NOTES:

1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
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Figure 41. Plan View of Alternative 2

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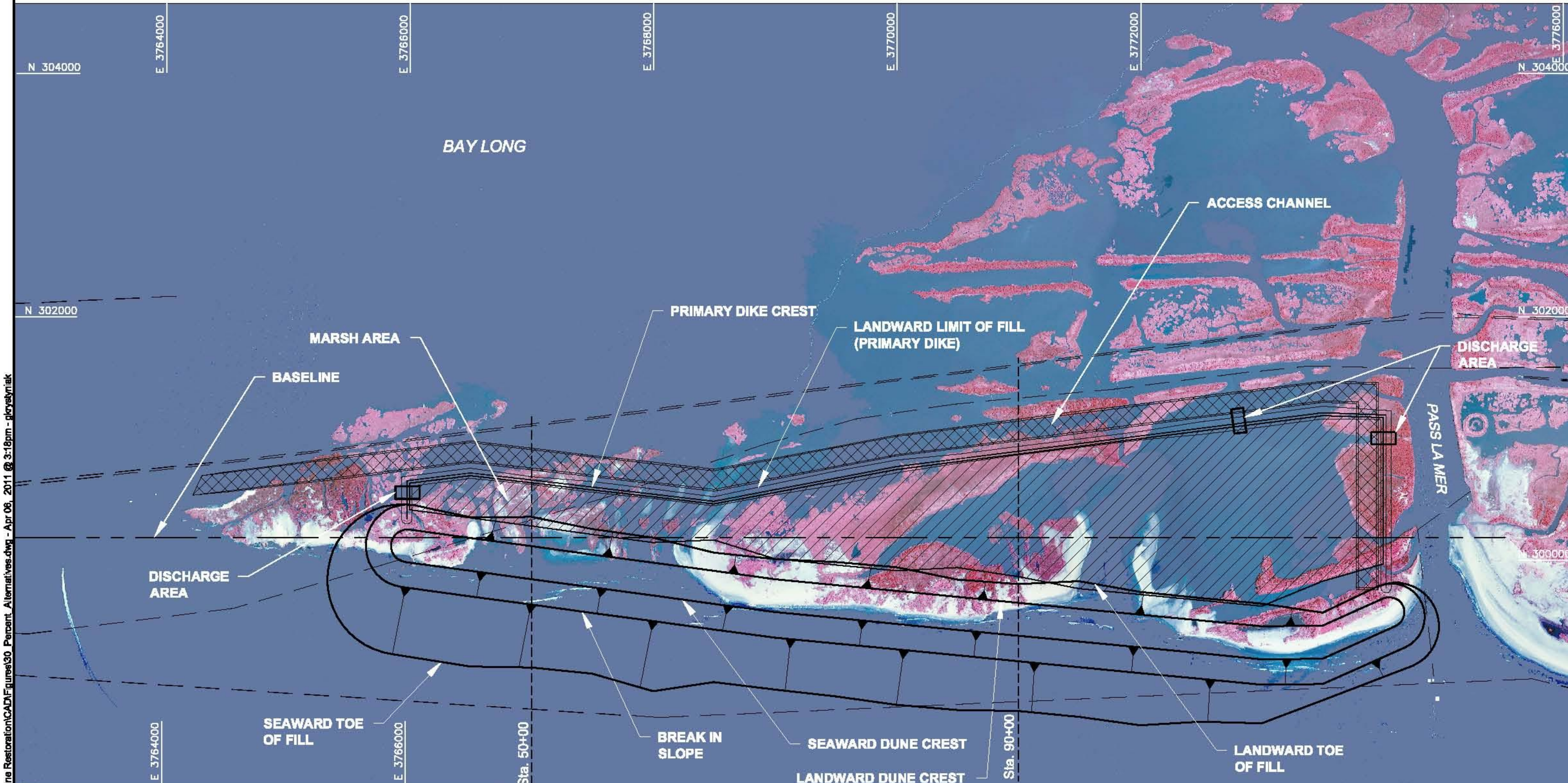

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Figure 42. Plan View of Alternative 3

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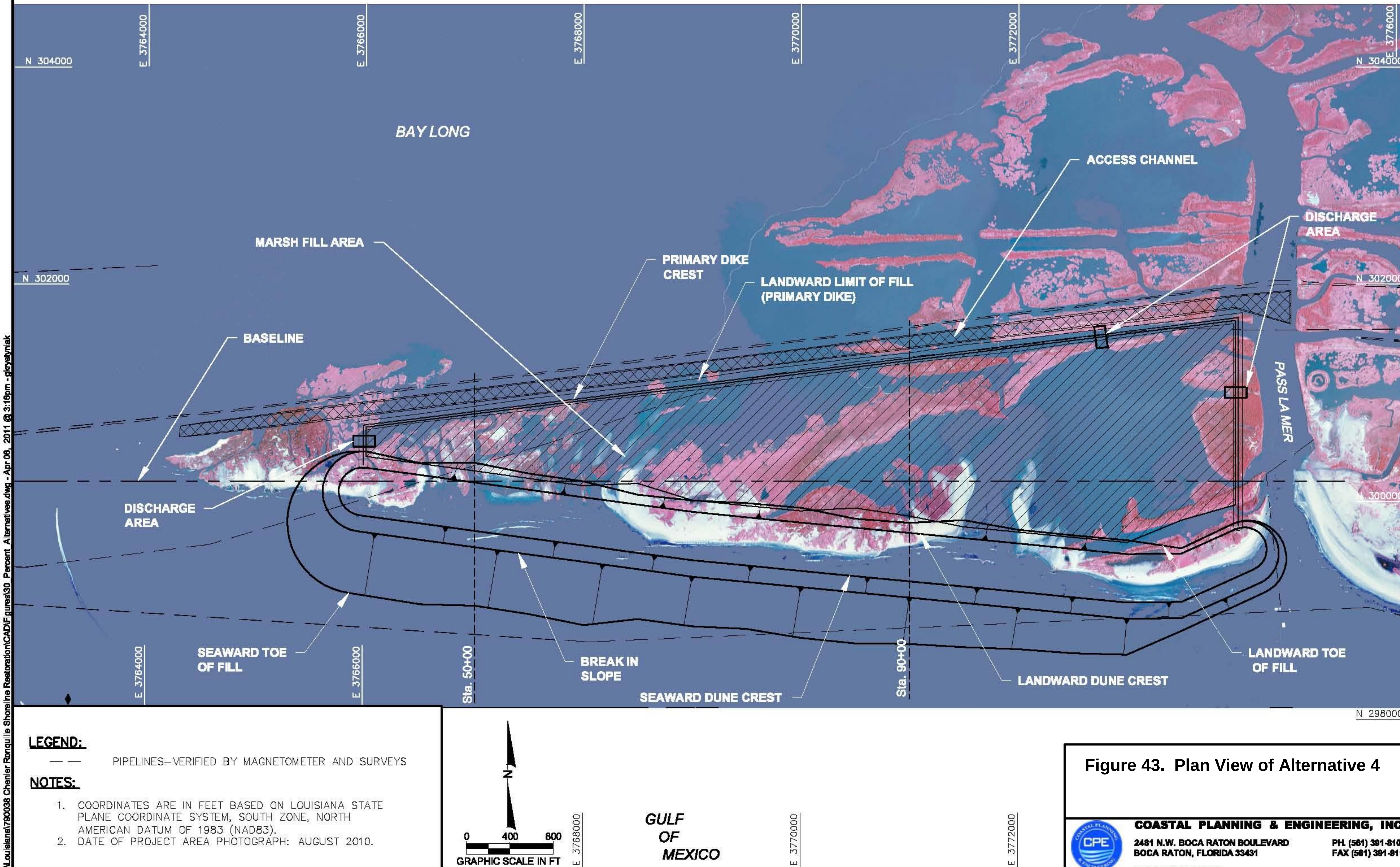

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Figure 43. Plan View of Alternative 4

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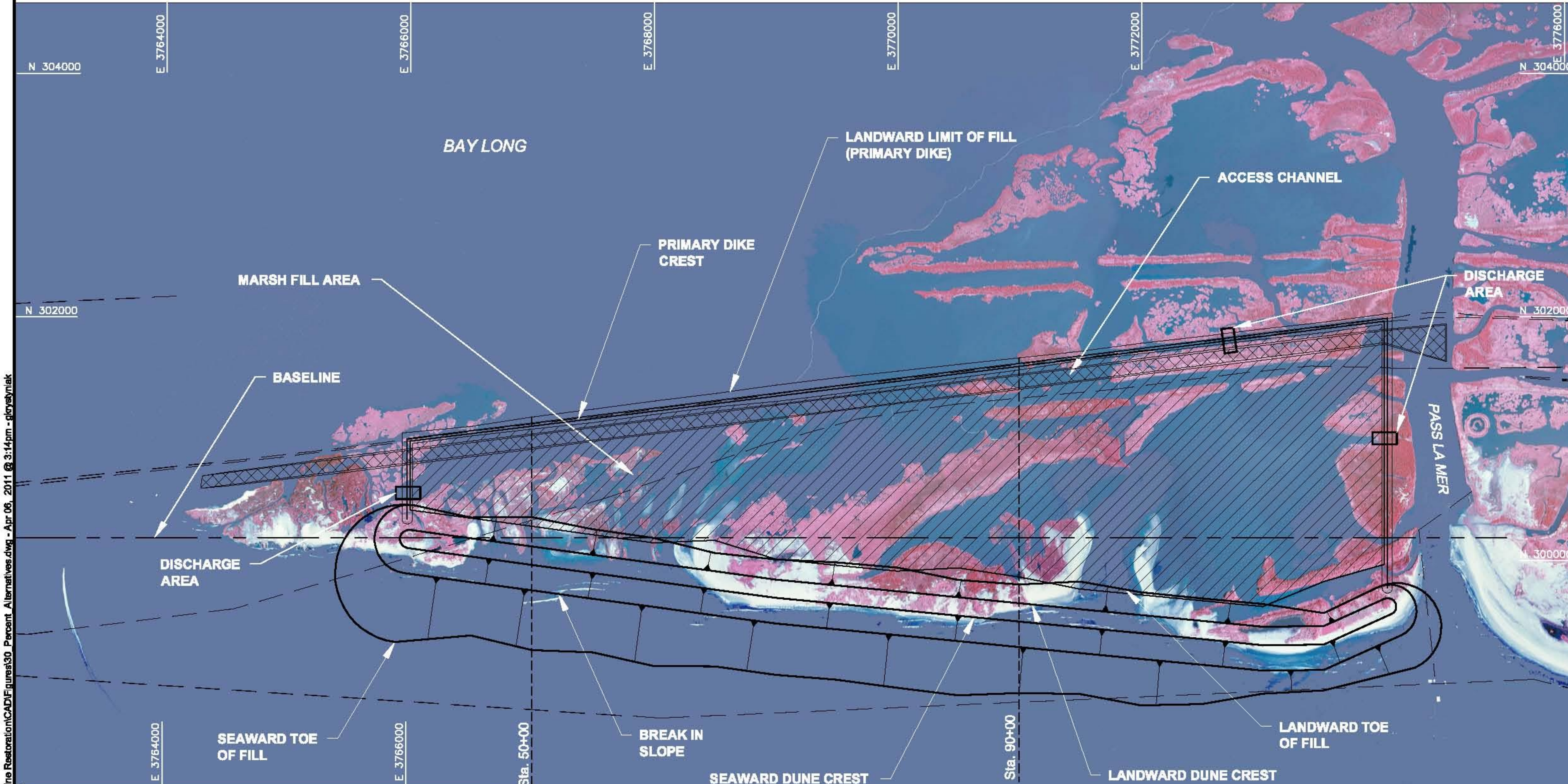

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1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
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Figure 44. Plan View of Alternative 5

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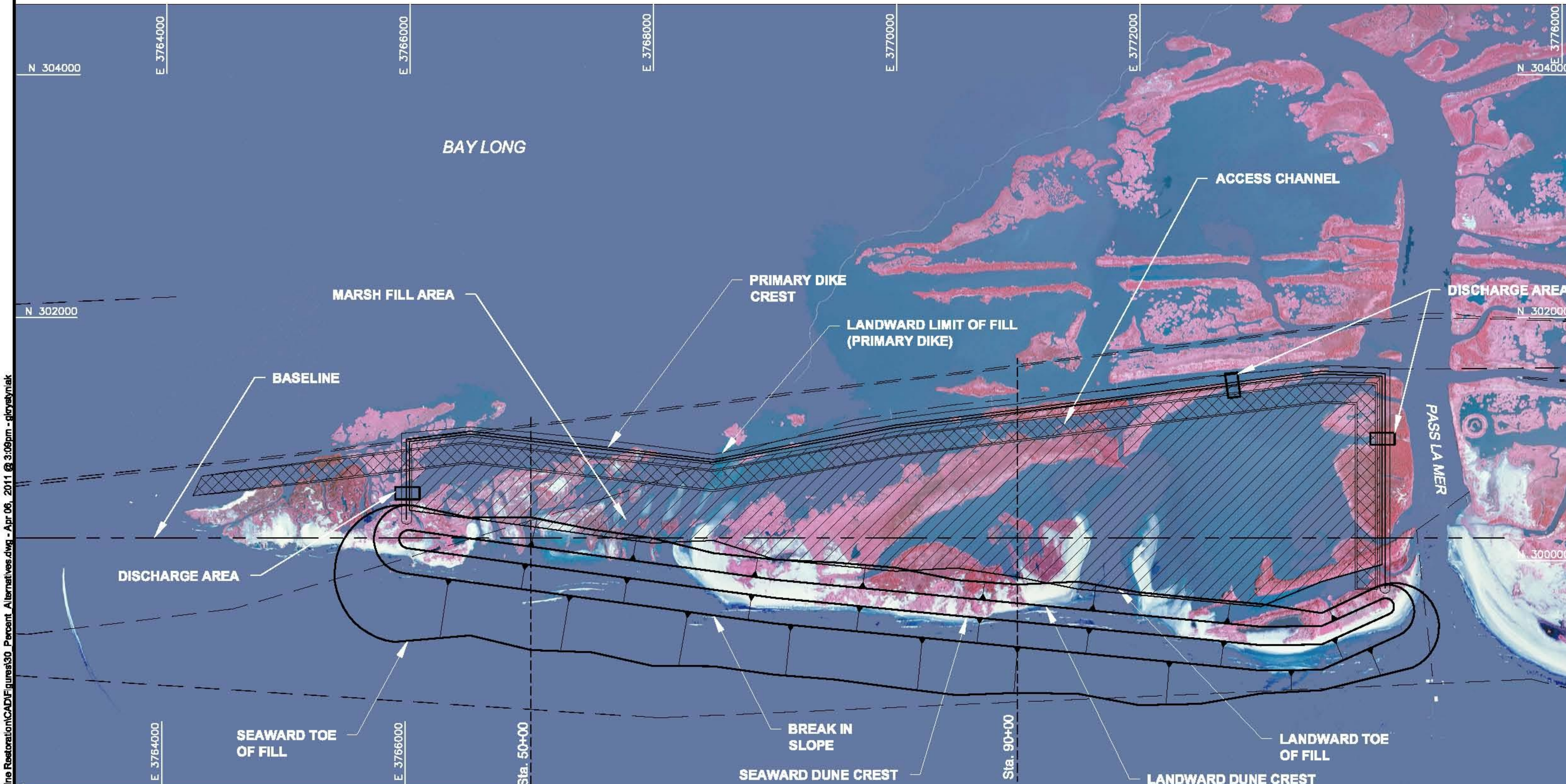

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1. COORDINATES ARE IN FEET BASED ON LOUISIANA STATE PLANE COORDINATE SYSTEM, SOUTH ZONE, NORTH AMERICAN DATUM OF 1983 (NAD83).
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Figure 45. Plan View of Alternative 6

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17.4 Alternative 4

Alternative 4 was designed to compare the +6 feet, NAVD and +8 feet, NAVD construction dune crest elevation options. Beach Option 2 has a +6 feet, NAVD dune crest elevation though the dune crest is wider than for Beach Option 1, so that the first two beach options have comparable construction volumes. Thus, Alternative 4 has a beach volume of 1,840,000 cubic yards.

Beach Option 2 was paired with Marsh Option 2 to develop Alternative 4. This pairing avoided the higher cost of Marsh Option 1 while maximizing marsh acreage. It also provided an alternative that was close to the estimated construction cost presented in the Phase 0 report. Marsh Option 2 contains 940,000 cubic yards of fill.

Alternative 4 has a construction footprint of 394 acres above 0 feet, NAVD (Figure 43).

17.5 Alternative 5

Alternative 5 was designed acknowledging that application of standard engineering practice regarding design and advanced fill may not be entirely applicable to coastal Louisiana and to provide a lower cost alternative while still maximizing project benefits. Given that the beach fill is more expensive than the marsh fill, it was decided to pair the smallest beach template (Beach Option 3) with the largest marsh template (Marsh Option 1). The larger marsh provides an additional factor of safety against breaching, should the project experience an above average storm climate over the 20-year project life.

Alternative 5 also allows the comparison of the effect of a smaller beach on project performance. Since Alternatives 1 and 5 have the same backing marsh, the effect of the beach fill can be directly compared.

The beach fill volume for Alternative 5 is 1,310,000 cubic yards while the marsh fill volume is 1,380,000 cubic yards. The construction footprint is 411 acres above 0 feet, NAVD (Figure 44).

17.6 Alternative 6

Alternative 6 was designed to provide the lowest cost alternative that could still meet the project goal. Alternative 6 comprises the smallest beach template (Beach Option 3) and the second smallest marsh template (Marsh Option 3). It was determined to exclude the smallest marsh template (Marsh Option 4) in the lowest cost alternative due to concerns that the marsh would be too narrow to trap all of the overwash. With Marsh Option 4, sand overwashing the primary dike would fall into the access channel where it would not provide any benefits during the 20-year project life. The bay area north of the primary dike under Marsh Option 3 is still shallow enough that if sand overwashes the primary dike, subtidal and potentially bay intertidal acreage could be created due to overwash. This occurred at Chaland Headland where the access channel was filled and overwash following Hurricanes Gustav and Ike created a sand flat north of the primary dike.

Alternative 6 has 1,310,000 cubic yards of beach and 1,020,000 cubic yards of marsh fill. The subaerial construction footprint is 350 acres (Figure 45).

17.7 Alternatives Not Carried Forward for Detailed Analysis

Six possible combinations of beach and marsh options were not carried forward for further analysis.

Beach Option 1, Marsh Option 3: This possible alternative is similar to Alternative 3 but more expensive. The benefits of the larger marsh planform could be assessed by comparing Alternative 2 and Alternative 3. The benefits of backfilling the access channel could be assessed by comparing Alternative 1 and Alternative 2 and did not require an additional alternative.

Beach Option 2, Marsh Option 1: Alternative 4 and Alternative 2 were developed to directly compare two dune elevations of +6 feet and +8 feet NAVD, respectively. These alternatives were designed to maximize the marsh acreage, while reducing costs by not backfilling the access channel. Therefore, this additional alternative was not considered to compare the two dune designs coupled with marsh option 1 (the largest marsh planform).

Beach Option 2, Marsh Options 3 and 4: Beach Option 2 was developed to directly compare two dune elevations of +6’ and +8’ NAVD, which was accomplished by comparing Alternative 4 and Alternative 2, respectively. Once it was clear that the dune elevation in beach option 2 would not meet as many project objectives, the project team decided to not incorporate additional alternatives using beach option 2. The two additional alternatives created by these marsh and beach option combinations would provide minimal information that would be beneficial to the alternative selection process.

Beach Option 3, Marsh Option 2: This possible alternative is similar to Alternative 5. Since Alternative 5 was developed using the smallest beach option to produce a cost effective alternative, the larger marsh option (Marsh Option 1) is preferred to maximize benefits.

Beach Option 3, Marsh Option 4: The project team determined that pairing the smallest beach design with the smallest marsh design may compromise project performance. This alternative may be too narrow to provide a large enough planform to capture overwashed sediment, and sand deposited in the unfilled access channel provides no quantifiable benefit during the project life. Alternative 6 is preferred to provide a least cost alternative.

Hard structures were considered but not carried forward for further analysis. A terminal groin was considered at the west end of the island to reduce sediment transported longshore and lost into Pass Ronquille. Thomson et al (2009) investigated the cost effectiveness of a terminal groin at West Belle Pass Barrier Headland. They determined that while the structure performed as intended the alternative including the terminal groin was the least cost effective alternative.

The analysis of a single offshore breakwater at the western terminus of the West Belle Pass Barrier Headland project (Thomson et al, 2009) was also reviewed to determine whether a similar structure would be effective at Chenier Ronquille. The analysis was inconclusive as to whether the single breakwater improved project performance despite the cost of breakwater construction. This alternative was ranked fifth out of the six West Belle Pass Barrier Headland alternatives in terms of cost effectiveness (cost per AAHU). Therefore, a single breakwater was not included in the six alternatives carried forward for further consideration.

A segmented breakwater field along the entire gulf shoreline of Chenier Ronquille was also considered. This would require approximately 18 breakwaters assuming a length of 300 feet with a gap width of 300 feet. An initial estimate was that a breakwater field could reduce the loss rates by 20 to 25%. However, the cost of this structural field was estimated at $9.5M which was approximately 25% to 33% of the budget for the non-structural options. To maintain the budget, the beach fill would be reduced by almost 50%, with a corresponding reduction in project benefits. Given the cost of this alternative coupled with the low cost effectiveness of the terminal groin and single breakwater, it was decided to carry forward additional dune height and marsh width options rather than a structural alternative.

17.8 Design Alternative Cost Estimates

Cost estimates for the six alternatives are shown in Table 31. A 25% contingency has been applied, which is standard at this preliminary design phase. The basis and breakdown of the cost estimates is included in Appendix G.

Table 31. Design Alternative Cost Estimates

Design Alternative	Total Cost	Cost/Constructed Acre
Alternative 1	$37,805,000	$104,700
Alternative 2	$34,449,000	$112,100
Alternative 3	$31,776,000	$134,000
Alternative 4	$34,570,000	$108,600
Alternative 5	$31,468,000	$93,300
Alternative 6	$28,730,000	$103,600

18 PROJECT PERFORMANCE

The project goal and objectives outlined in Section 3 provide both qualitative and quantitative benchmarks. Projects sponsored under CWPPRA are also evaluated for environmental benefits using quantitative projections of planform performance employing the Wetland Value Assessment model (WVA) among other performance evaluation tools. This section will discuss the expected performance of various alternatives, including the no action alternative, with respect to project objectives, general performance, and WVA benefits.

18.1 Analytic Model Predicting Habitat Acreage Change

An analytic model was developed that estimated the acreages of various habitat elevations for each alternative, including the no action alternative. This model provides an estimate of island performance. The quantities and the model results for the Future Without Project (FWOP) and

Future With Project (FWP) conditions are shown in Appendix F. A summary of the key elements of the analytic model is included in the paragraphs below.

The analytic habitat acreage change model incorporated the following processes:

1. Gulf and bay shoreline reduction at the western extent as the shorelines receded due to the island's wedge shape planview geometry.
2. Gulf shoreline recession due to longshore losses, relative sea level rise, overwash, and the silt fraction in the beach (offshore losses).
3. Change in the gulf shoreline elevation and active profile height resulting in the loss of acreage and a conversion of one habitat type to another (dune to supratidal and supratidal to bay intertidal).
4. Subsidence resulting in conversion of one habitat type to another (dune to supratidal and supratidal to bay intertidal).
5. Net decrease in marsh platform elevation due to historical subsidence which offset detritus accumulation in vegetated areas. This results in a conversion of one habitat type to another (supratidal to intertidal).
6. Annual storm overwash resulting in conversion of one habitat type to another (dune to supratidal or bay intertidal to supratidal).
7. Bay shoreline recession resulting in loss of bayside acreage (bay intertidal and subtidal) due to anticipated waves propagating from the north. This is assumed to be 3 feet/year based on the observed back bay erosion in Bastian Bay (Thomson and Wycklendt, 2009).

For the FWP alternatives, the analytic habitat acreage change model also incorporated the following processes:

8. Gulf shoreline recession the year following construction as the constructed profile equilibrates to the natural profile. Equilibration of the profile results in a loss of acreage from the highest constructed habitat type (dune).
9. Settlement and subsidence of the constructed dune due to the additional load applied to the underlying substrate. This process is assumed until the target year that the gulf shoreline elevation becomes equivalent to natural barrier island elevation. This results in a conversion of one habitat type to another and additional acreage loss due to shoreline recession (dune to supratidal).
10. Consolidation, settlement, and subsidence of the constructed marsh platform due to the additional load applied to the underlying substrate. This results in a conversion of one habitat type to another (supratidal to bay intertidal).
11. A change in the active profile height due to lowering of the dune that occurred following the two significant (10-year) storm events, estimated to occur in TY7 and TY14. A probability analysis suggests that a 10-year storm event has a 50% chance of occurrence by TY7 (Thomson et al., 2009).
12. Conversion of habitat (dune to supratidal and bay intertidal to supratidal) due to major storm overwash as dune elevation is lowered and material deposited landward onto the marsh platform.
13. Increase in the natural gulf shoreline elevation and depth of closure due to sea-level rise. The difference in elevation with respect to mean high water (MHW) is maintained to account for sea-level rise.

At each target year, the shoreline recession and lowering of island elevations is converted to a loss of acreage based on the variable shoreline lengths, profile heights, and the yearly elevation changes. The entire profile is translated so losses only occur in the uppermost habitat area. All values assume that construction is completed by the end of 2014, which defines TY1.

The main drawback on the analytic model is that it does not account for sandy sediment transported to the west of the beach fill footprint during the FWP conditions. The model accounts for the armoring of the shoreline reach to the western tip of the island, but it may not accurately account for the volume of sediment overwashed onto the backing marsh during annual storm events. The introduction of sand into the system may stabilize the western tip and thus reduce historical erosion rates. An additional drawback to the model is that it assumes a uniform marsh platform resulting in a uniform overwash elevation and accumulation rates which may not be representative for the FWOP conditions.

18.2 Future Without Project (FWOP) Alternative

Future without project conditions were based on projections made for acreage and shoreline change rates for the various habitat areas. These values were determined by analyzing 1998 and 2006 LIDAR data, 1998 and 2010 aerial imagery, and survey data collected in 2010.

Subaerial acreages, including dune, supratidal, and intertidal acreages, were estimated by analyzing the rate of acreage loss and projecting it forward for any acreage that was within the WVA boundary (see Appendix F for details). The acreages were estimated from the 1998 and 2006 LIDAR data sets, and then a linear interpolation was used to estimate the rate of acreage loss (Table 32). From these land loss rates, the acreages at each target year were extrapolated. For comparison, the land loss rates were used to extrapolate the instantaneous percent acreage loss for the various habitat types in 2011 as shown in Table 32.

Table 32. Land Loss Rates Extrapolated between 1998 and 2006 LIDAR data

Habitat Type	Absolute Loss Rate (ac/yr)	Percent Loss Rate (%/yr)
Dune	0.4	30.7%
Supratidal	3.2	20.0%
Gulf Intertidal	-	1.0%
Bay Intertidal	5.6	5.1%
Subtidal	3.8	4.9%

The FWOP gulf shoreline was projected by analyzing the shoreline retreat rate between 1998 and 2006. This was overlaid on the 2008 aerial and where the shoreline was located in open water, it was assumed that the shoreline was breached in this location or had been eroded. Breaching will increase the shoreline retreat rate, but this was ignored resulting in a conservative (higher) estimate of future without project acreage. The west end of the island has experienced erosion thus reducing the shoreline length. The length of shoreline in TY20 (2034 for purposes of the analysis) was estimated to be 9,900 feet long compared to 11,600 feet in 2006.

The FWOP performance acreages at given target years are summarized in Table 33. Appendix F contains the details of the supporting calculations. The analytic model predicted that dune and supratidal acreage would be lost sometime between TY1 and TY5, bay intertidal acreage will be lost by TY17, and subtidal acreage will be lost by TY18. Comparison of the 1998 and 2006 LIDAR data indicated an increase in gulf supratidal acreage. Part of this increase may be due to overwash but can also be contributed to difficulty in defining gulf intertidal habitat verses bay intertidal habitat. Regardless, a gain in habitat is obviously not sustainable. Projecting total acreage forward suggests that all subaerial acreage will be lost by TY20, which required an assumption that the gulf intertidal loss rate was 1%/year. The No Action Alternative resulted in 39 average annual habitat units (AAHU's) over the 20-year period of analysis.

Table 33. Planform Performance Projection for Future Without Project (FWOP) Conditions

Target Year	Habitat (acres)					
	Dune	Supratidal	Gulf Intertidal	Bay Intertidal	Subtidal	Total
TY 0	1	10	18	97	70	196
TY 1	1	6	18	92	66	183
TY 5	0	0	17	64	47	128
TY 10	0	0	15	36	28	79
TY 15	0	0	14	8	9	31
TY 16	0	0	13	3	5	21
TY 17	0	0	10	0	2	12
TY 18	0	0	4	0	0	4
TY 20	0	0	0	0	0	0

The No Action alternative (FWOP) does not meet the project goal or any of the objectives stated in Section 3. A discussion of the project goal and each objective under the no action alternative is provided below.

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

   The existing island has a maximum gulf shoreline elevation of +3.0 feet, NAVD or less and has been breached. Due to a lack of sediment input, Chenier Ronquille will continue to disintegrate as it transitions into a tidal overwash flat. The back-barrier marsh will continue to be exposed to the Gulf increasing erosion rates and negatively impact marsh habitats. Continued loss of ecosystem habitat is expected. Therefore, the FWOP alternative does not meet the project goal.

2. *Prevent island breaching over the 20-year project life.*

   Chenier Ronquille already has multiple breaches in the island with insufficient sediment volume available to close the breaches naturally. Therefore, the FWOP alternative does not meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   The FWOP alternative does not restore any marsh habitat. At TY1, the project area is estimated to have 183 acres of habitat of which 158 are considered marsh habitat. While marsh habitat with tidal exchange will exist in TY4, no increase in acreage will result from the FWOP alternative.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20.*

   The existing island has an average gulf shoreline elevation of +3.0 feet, NAVD or less with spoil banks at the east end that contain dune acreage. It is expected that the spoil banks will continue to degrade over time and that there will be no acreage remaining above +4 feet, NAVD by TY 20. Therefore, the FWOP alternative does not meet this objective.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Similar to the assessment for objective 4, no dune acreage is estimated to remain beyond TY5. Therefore, the FWOP alternative does not meet this objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Subaerial acreage at TY1 is anticipated to be at 117 acres with no subaerial acreage at TY20 due to natural processes under the FWOP conditions. Therefore, the FWOP alternative is not expected to meet this objective.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline.*

   Under the FWOP condition, shoreline recession is anticipated to continue at a rate of between 32.0 and 58.4 feet/year as observed between 1998 and 2010. By TY20, the FWOP shoreline location will be landward of the pre-construction shoreline. Therefore, the FWOP alternative is not expected to meet this objective.

18.3 Future With Project Conditions

Future with project (FWP) performance was developed based on an analytic model. A similar analytic model was applied for other Louisiana shoreline restoration projects including the Barataria Basin Barrier Shoreline LCA Project, Shell Island Appendix (Thomson, et al., 2008) and West Belle Pass Barrier Headland Restoration Project (Thomson, et al., 2009).

It is expected that the constructed island will equilibrate and dune elevation will lower gradually over time until it reaches a natural barrier island condition. Observations of barrier islands in Louisiana that have sandy littoral systems, including West Grand Terre Island, the Chandeleur Islands, and Breton Island, indicate that a natural gulf shoreline elevation between +2.5 and +4 feet, NAVD can be expected. At this point in time, there should be sufficient sandy sediment remaining within the littoral system to maintain a continuous gulf shoreline with no breaches. Within the analytic model, a natural island elevation of +3.0 feet (NAVD) at TY0 was assumed that would increase in elevation at the rate of eustatic SLR (~0.0056 feet/year) to maintain the difference in elevation relative to MHW (0.96 feet, NAVD, at TY0). Once the constructed dune is lowered to the natural elevation, it increases at the rate of SLR.

Due to natural processes, including settlement of the underlying substrate, historical subsidence, and major storm events, the constructed dune decreases in elevation until it reaches a natural island elevation. Analysis was performed by Fugro Consultants, Inc. on the underlying substrate to quantify settlement due to the load of the constructed dune. Settling is generally due to consolidation (i.e., a decrease in void fraction) of the supporting soil (Lindeburg, 2003). NOAA published a relative sea-level rise at Grand Isle, Louisiana estimated at 9.24 mm/year and a eustatic sea-level rise of 1.7 mm/year (USACE 2009) resulting in a historical subsidence rate of 7.54 mm/year. 10-year storm events were assumed to impact the island in TY7 and TY14, which lower the dune elevation and result in overwash events that deposit sediment onto the marsh platform. SBEACH modeling was used to estimate the expected lowering of the dune crest elevations at TY7 and TY14 after the 10-year storm events. The model was also used to approximate overwash distances, which contributes to the acreage that can be converted from bay intertidal habitat to supratidal habitat. Overwash distance is also considered based on the availability of sediment along the gulf shoreline. Once the dune lowers to the natural shoreline elevation, annual overwash is initiated and the shoreline and overwash fans then increase in elevation at the rate of sea-level rise. Thus, acreage loss due to settlement and subsidence is incorporated into the shoreline retreat rate once the natural equilibrium elevation is achieved. Two dune elevations (+6.0 and +8.0 feet, NAVD) were considered for the alternatives and the expected lowering of the dune elevation due to settlement, historical subsidence, and storm events are shown in Figure 46.

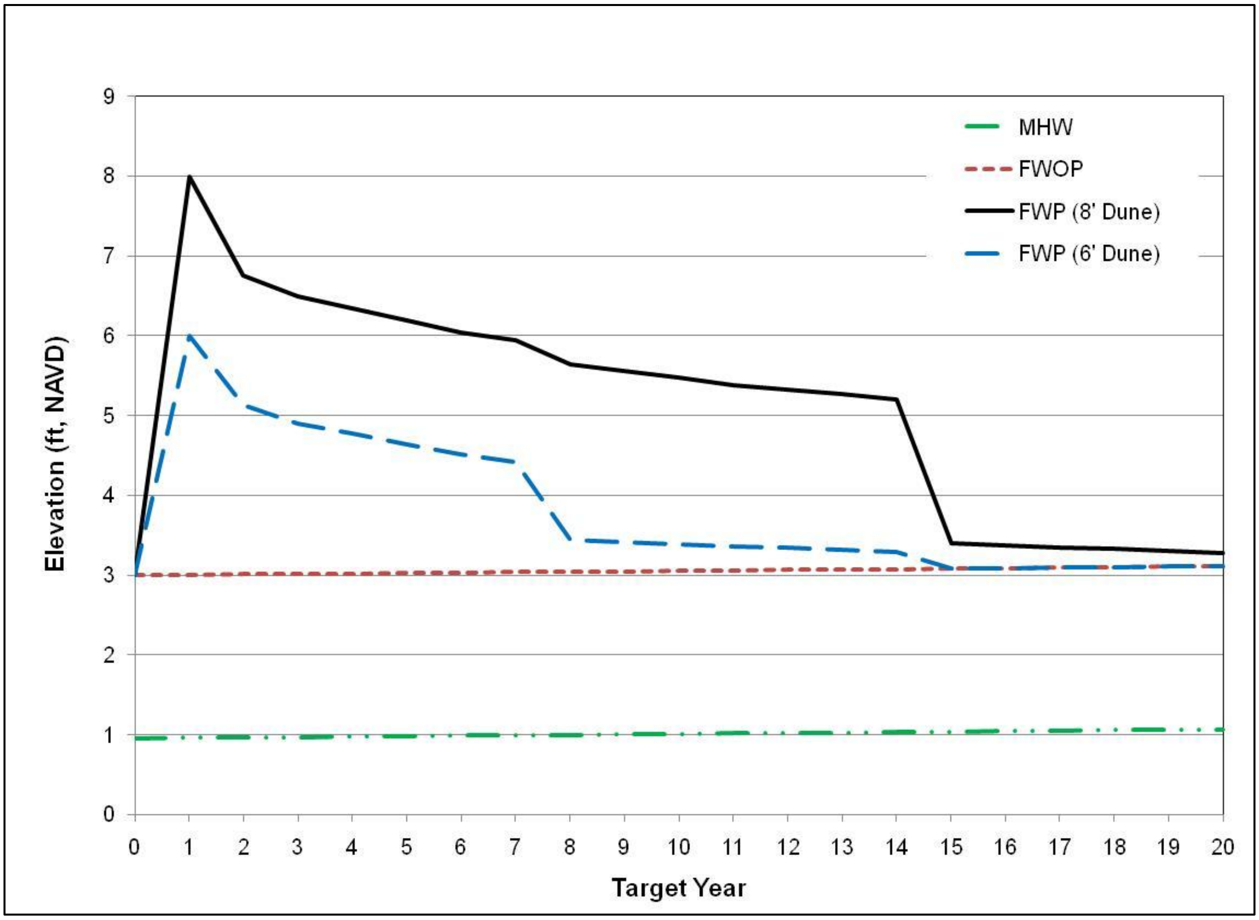

Figure 46. Lowering of dune elevation due to settlement, historical subsidence, and major storm events.

The various alternatives become susceptible to being overtopped resulting in annual overwash being initiated at different target years based on their construction dune widths, crest elevations, and shoreline recession rates. As the dune is lowered during these storm events, sediment is transported landward onto the backing marsh, creating a wider and lower crest. After these major storm events lower the dune to the natural equilibrium gulf shoreline elevation, it is assumed that the island will roll over and continue to overwash onto the backing marsh platform.

The natural processes governing the marsh platform were fundamentally different as compared to the beach. The elevation was assumed to be affected due to historical subsidence, consolidation of the placed fill material, settlement of the underlying substrate, and detritus accumulation from vegetation. The rate of detritus accumulation was estimated between 3-9 mm/year (Fitzgerald et al., 2003), but a conservative estimate of 3.0 mm/year was assumed. Detritus was assumed to start in TY5 to allow time for the project area to vegetate and start to produce detritus. The net result of subsidence, consolidation, settlement, and detritus accumulation was a lowering of the marsh platform throughout the project life. The marsh platform elevation was assumed to decrease throughout the project life as the constructive processes (detritus accumulation) do not persist at a rate that can offset subsidence, consolidation, and settlement.

For the alternatives that required the access channel to be filled with hydraulically placed material during marsh construction, similar assumptions and processes were applied to that of the marsh platform. Within the footprint of the access channel, the time rate of consolidation was scaled based on the increased lift thickness. The mudline after excavation will be -7.0 feet, NAVD, and the marsh platform will be +2.5 feet, NAVD resulting in a lift thickness of 9.5 feet. Thus, consolidation of the placed material was greater resulting in a greater decrease in elevation when compared to the rest of the marsh area. A similar consolidation trend was assumed for the primary dike, which has an average base elevation of 0 feet, NAVD and a crest elevation of +5.0 feet, NAVD resulting in a lift thickness of 5 feet.

The beach component of the alternatives extended from the east end of the island to within approximately 1,500 feet of the west end. The portion of the island to the west of the constructed beach fill footprint did not receive direct placement of sandy material. However, sand will be transported to this portion of the island due to longshore transport as well as diffusion loss from the fill area. A diffusion analysis was performed to estimate the shoreline advance along this reach from sand placed during construction. Assuming a breaking wave height of 0.5 feet and a sediment transport coefficient of 0.8 to be conservative, the shoreline near Station 30+00 accumulated approximately 30 feet of beach width within one year post-construction (Figure 47). The figure shows the shoreline advance (perturbation) due to placed beach fill and accretion at the west end, assuming that the shoreline location at each profile station immediately pre-construction (TY0) is equal to zero. It was assumed that this section of beach would maintain a natural island elevation of +3.0 feet, NAVD due to natural processes. The western reach was assumed to perform similarly to the constructed dune throughout the project life with respect to shoreline retreat rates.

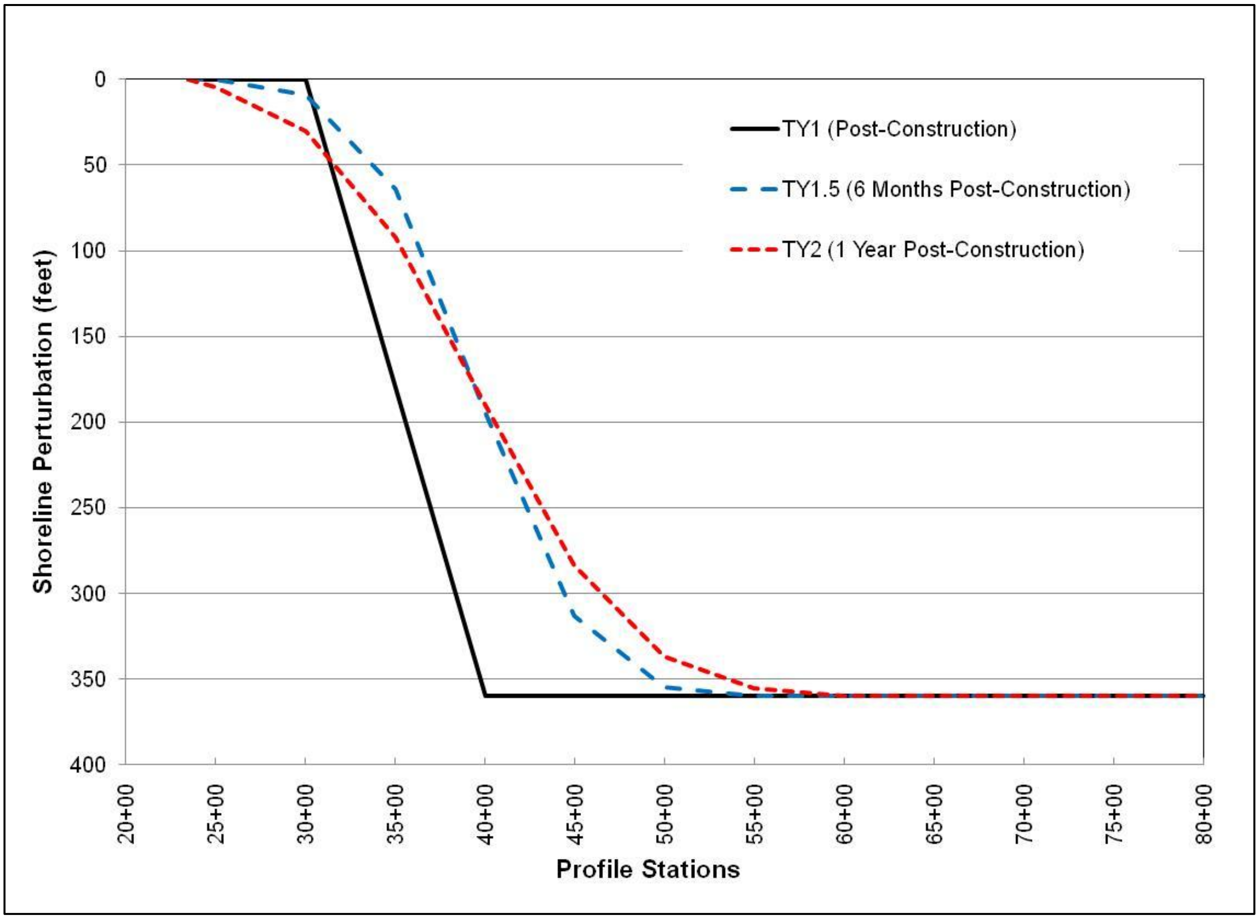

Figure 47. Diffusion at the West End of the Beach Fill.

The diffusion analysis suggests that the west end of the island erodes at a rate similar to the FWP conditions. Therefore, the gulf and bay shorelines were reduced in length based on the rate of FWP shoreline recession. As the shoreline lengths were reduced, acreages correlated to these shoreline dimensions, including a supratidal, gulf intertidal, bay intertidal, and subtidal acreages, were reduced proportionally throughout the project life from TY0 to TY20.

The WVA model assumes that the project construction will be completed by TY1. The WVA boundary was developed to capture benefits resulting from the construction and performance of all alternatives. The boundary is larger than the project footprint to capture benefits that will accrue over time in areas outside the footprint. For example, the west end of Chenier Ronquille will benefit from longshore sediment transport after construction. A consistent boundary was applied for the analysis of each alternative to accurately compare the benefits realized by the alternatives. Details on the development of the WVA boundary are located in Appendix F.

The acreages outside the project footprint were estimated by quantifying land loss rates between the 1998 and 2006 LIDAR data around the perimeter of the constructed project footprint. The acreage was then estimated from the 2006 data at TY1 (2014) based on these loss rates. It was estimated that 1.3 acres/year were lost at the west end of the island and approximately 0.7 acres/year were lost elsewhere within the WVA boundary. The acreages outside the constructed footprint were added to the constructed project acreages to estimate the total habitat acreages at TY1.

18.3.1 Alternative 1

Alternative 1 is designed to meet the project goal by following a standard coastal engineering design, comprising a design section, advanced fill and a wide marsh to limit sediment losses due to overwash. The alternative consists of a beach (beach option 1) with a dune crest elevation of +8.0 feet, NAVD, dune crest width of 270 feet, and a backing marsh platform (marsh option 1) constructed to +2.5 feet, NAVD. The access channel will be backfilled with hydraulically placed material during construction resulting in the largest marsh footprint of any alternative.

After construction, the active profile height is reduced over time as the dune crest elevation decreases due to subsidence, settlement, and overtopping during two 10-year storm events. As a result, the gulf shoreline recession rate generally increases throughout the project life. The estimated recession rates for the analytic model included 11 feet/year during construction, 48 feet/year in TY2 of which 37 feet is associated with the initial settlement of the dune, 15 feet/year in TY8 following the first major storm event, and 21 feet/year after the second major storm event in TY14.

Settlement of the dune as predicted by Fugro’s analysis on the underlying substrate coupled with historical subsidence suggested that the dune will be lowered from a constructed elevation of +8.0 feet, NAVD to an elevation of +5.9 feet, NAVD prior to the first 10-year storm event in TY7. SBEACH modeling indicated that dune elevation will be lowered to approximately +5.6 feet, NAVD following the storm resulting in overwash onto the marsh platform. By TY14 prior to the second major storm event, the gulf shoreline elevation will be +5.2 feet, NAVD due to 7 years of subsidence and settlement. After the storm, SBEACH modeling indicated that the shoreline will be lowered to an elevation of approximately +3.4 feet, NAVD resulting in overwash onto the marsh platform. Overwash is the principle method of dune acreage loss, resulting in sand being transported onto the marsh platform, which converts bay intertidal acreage to supratidal habitat due to sediment gain in the backing marsh.

Beginning in TY15, annual overwash is initiated as the dune elevation has been lowered to approximately the natural barrier island elevation. The rate of overwash averages 9.4 feet/year for the remainder of the project life. The annual overwash is realized by a conversion of bay intertidal acreage to supratidal habitat.

Table 34 shows the planform performance projections of the various habitat types throughout the 20-year project life. It should be noted that there are additional habitat acres outside the project footprint but within the WVA boundary that were included in the analysis.

Immediately post-construction (TY1), approximately 87 acres of dune, 366 acres of subaerial, and 36 acres of subtidal habitats will exist within the WVA boundary. The marsh is constructed as supratidal habitat, but due to settlement and consolidation of the placed material it is expected to settle into the bay intertidal range by TY2 as denoted by the conversion of acreage.

Table 34. Planform Performance Projection for Alternative 1

	Habitat (acres)					
Target Year	**Dune**	**Supratidal**	**Gulf Intertidal**	**Bay Intertidal**	**Subtidal**	**Total**
TY 0	1	10	18	97	70	196
TY 1	87	325	21	20	36	489
TY 2	61	45	21	293	35	455
TY 3	56	43	21	292	35	447
TY 4	51	41	21	291	35	439
TY 5	47	39	20	291	35	432
TY 6	43	37	20	290	35	425
TY 7	39	36	20	289	35	419
TY 8	36	48	20	275	35	414
TY 9	33	46	20	275	35	409
TY 10	29	46	20	274	35	404
TY 11	25	45	20	272	35	397
TY 12	22	45	20	271	35	393
TY 13	19	44	20	270	34	387
TY 14	15	44	20	269	34	382
TY 15	0	119	20	203	34	376
TY 16	0	115	20	200	34	369
TY 17	0	112	20	197	34	363
TY 18	0	109	20	194	34	357
TY 19	0	106	20	191	34	351
TY 20	0	103	20	187	34	344

Alternative 1 is estimated to provide 302 AAHU's as compared to the no action alternative, which is expected to provide 39 AAHU's, resulting in a net increase of 263 AAHU's due to the project (Table 41, see Appendix F for detailed analysis). Given a construction cost of $37,805,000, the cost per AAHU is $143,700.

A discussion of the performance of Alternative 1 with respect to the project goal and objectives follows.

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage*

   Alternative 1 meets this goal as sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 and dune acreage is preserved through TY14. Over 300 acres of beach and marsh are created during construction.

2. *Prevent island breaching over the 20-year project life.*

   Alternative 1 was specifically designed to prevent breaching of the shoreline in TY20, without consideration of the benefit of the backing marsh, which will provide additional protection. Therefore, Alternative 1 meets this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 291 acres of bay intertidal marsh at TY4. Gapping of the primary dikes at naturally low areas immediately following construction should allow tidal exchange to develop naturally by TY4. Therefore, Alternative 1 is expected to meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20*.

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Approximately 36 acres of dune habitat are expected to remain following the first 10-year storm, which was assumed to occur in TY7. Alternative 1 was designed specifically to meet this objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Alternative 1 has 453 acres following construction. Approximately 68% of the constructed subaerial acreage (310 acres) remains in TY20, which exceeds the objective of 50%. Therefore, Alternative 1 meets this objective.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline*.

   The location of the constructed shoreline was designed such that the projected 2014 shoreline would not be exposed to the Gulf over the 20-year project life. Once Alternative 1 performs as designed, it should meet this objective.

18.3.2 Alternative 2

Alternative 2 is similar to Alternative 1 except a smaller marsh platform (marsh option 2) was proposed to reduce project costs. The access channel is located in the same location as marsh option 1, but the primary dike is constructed to the south and the channel is not backfilled during construction. This reduces project costs as the marsh fill volume required is less with the smaller footprint.

Since the constructed dune (beach option 1) is the same and the marsh footprint is only marginally smaller than that of Alternative 1, the project performance of the Alternative 2 is similar to Alternative 1. Table 35 shows the expected planform performance projection of the various habitat types throughout the 20-year project life. It should be noted that there are additional habitat acres outside the project footprint within the WVA boundary that were included in the analysis.

Table 35. Planform Performance Projection for Alternative 2

Target Year	Dune	Supratidal	Gulf Intertidal	Bay Intertidal	Subtidal	Total
	Habitat (acres)					
TY 0	1	10	18	97	70	196
TY 1	87	269	21	22	44	443
TY 2	61	44	21	240	44	410
TY 3	56	42	21	239	44	402
TY 4	51	40	21	238	44	394
TY 5	47	39	20	238	43	387
TY 6	43	37	20	237	43	380
TY 7	39	36	20	236	43	374
TY 8	36	47	20	222	43	368
TY 9	33	46	20	222	43	364
TY 10	29	46	20	221	43	359
TY 11	25	45	20	220	43	353
TY 12	22	45	20	219	43	349
TY 13	19	44	20	218	43	344
TY 14	15	44	20	217	43	339
TY 15	0	119	20	150	42	331
TY 16	0	115	20	147	42	324
TY 17	0	112	20	144	42	318
TY 18	0	109	20	141	42	312
TY 19	0	106	20	138	42	306
TY 20	0	103	20	135	42	300

Immediately post-construction, approximately 87 acres of dune, 312 acres of subaerial, and 44 acres of subtidal habitats will exist within the WVA boundary. The performance of the dune due to settlement, subsidence, major storm events, and overwash is the same as Alternative 1 because the dune crest elevation and width is identical. The marsh is constructed as supratidal habitat, but due to settlement and consolidation of the placed fill and historical subsidence it is expected to settle into the bay intertidal range by TY2 as denoted by the conversion of acreage.

Alternative 2 is estimated to provide 275 AAHU's compared to the no action alternative, resulting in a net increase of 236 AAHU's due to the project (Table 41). This is less than that of Alternative 1 because of the reduced marsh footprint. The cost per AAHU for Alternative 2 is $146,000, which is less than that of Alternative 1 because constructing additional marsh acreage is cost effective.

A discussion of the performance of Alternative 2 with respect to the project goal and objectives follows.

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

Alternative 2 meets the goal as sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 and dune acreage is preserved through TY14. Approximately 285 acres of beach and marsh are created during construction with another 99 being restored within the project footprint. Given the expected acreage remaining at TY20, Alternative 2 meets the goal of maintaining a functional barrier island ecosystem.

2. *Prevent island breaching over the 20-year project life.*

   The beach section of Alternative 2 was specifically designed to prevent breaching of the shoreline in TY20, without consideration of the benefit of the backing marsh, which will provide additional protection. Therefore, Alternative 2 should meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 238 acres of bay intertidal marsh at TY4. Gapping of the primary dikes at naturally low areas immediately following construction should allow tidal exchange to develop naturally by TY4. Therefore, Alternative 1 is expected to meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20.*

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Since Alternatives 1 and 2 have identical beach templates and the marsh in Alternative 2 is sufficient to contain any overwash, Alternative 2 is expected maintain a dune similarly to Alternative 1. Approximately 36 acres of dune habitat will remain following the first 10-year storm, which is assumed to occur in TY7. Therefore, Alternative 2 meets this objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Alternative 2 maintains 65% of its post-construction subaerial acreage by TY20 (258 acres in TY20 compared to 399 acres in TY1). Thus, Alternative 2 meets this objective.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline*.

   The beach option within Alternative 2 is identical to Alternative 1 and specifically designed to meet this objective.

18.3.3 Alternative 3

Alternative 3 consists of a dune (beach option 1) constructed with a crest elevation of +8.0 feet, NAVD and a reduced marsh platform (marsh option 4) constructed to an elevation of +2.5 feet, NAVD. The marsh platform is similar to the Phase 0 preferred option and provides an alternative with a lower total project cost. The access channel is constructed to the north of the marsh footprint and is not backfilled during construction thus reducing the volume of marsh fill required and the cost to construct the primary dike.

The performance of the constructed dune is similar to Alternative 1 while the difference in project performance is due to the smaller marsh platform. Table 36 shows the expected planform performance projection of the various habitat types throughout the 20-year project life.

Table 36. Planform Performance Projection for Alternative 3

	Habitat (acres)					
Target Year	Dune	Supratidal	Gulf Intertidal	Bay Intertidal	Subtidal	Total
TY 0	1	10	18	97	70	196
TY 1	87	199	21	32	70	409
TY 2	61	44	21	181	70	377
TY 3	56	42	21	180	69	368
TY 4	51	40	21	180	69	361
TY 5	47	38	20	179	69	353
TY 6	43	37	20	179	69	348
TY 7	39	35	20	178	69	341
TY 8	36	47	20	164	69	336
TY 9	33	46	20	164	68	331
TY 10	29	46	20	163	68	326
TY 11	25	45	20	162	68	320
TY 12	22	45	20	161	68	316
TY 13	19	44	20	160	68	311
TY 14	15	44	20	159	68	306
TY 15	0	119	20	93	67	299
TY 16	0	115	20	90	67	292
TY 17	0	112	20	87	67	286
TY 18	0	109	20	84	67	280
TY 19	0	106	20	82	66	274
TY 20	0	103	20	79	66	268

Immediately post-construction, approximately 87 acres of dune, 252 acres of subaerial, and 70 acres of subtidal habitats will exist within the WVA boundary. The marsh is constructed as supratidal habitat, but due to settlement and consolidation of the placed fill and historical subsidence it is expected to settle into the bay intertidal range by TY2 as denoted by the conversion of acreage.

This alternative has the lowest acreage due to the small marsh, which results in Alternative 3 having one of the lowest number of net AAHU's (216 AAHU's) of any alternative (Table 41).

The small marsh also results in Alternative 3 having one of the highest costs per AAHU, $147,100/AAHU.

Alternative 3 performs similarly to Alternative 1 with respect to the performance of the beach. A discussion of each project goal and objective and how it is met is provided below.

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

   Alternative 3 meets this goal as sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 and dune acreage is preserved through TY14. Approximately 196 acres of beach and marsh are created during construction with another 183 being restored. While the marsh option within Alternative 3 is the smallest of the three alternatives with beach option 1, Alternative 3 is still expected meet the project goal.

2. *Prevent island breaching over the 20-year project life.*

   The beach section of Alternative 3 was specifically designed to prevent breaching of the shoreline in TY20, without consideration of the benefit of the backing marsh, which will provide additional protection. Therefore, it is expected that Alternative 3 will meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 180 acres of bay intertidal marsh at TY4. Three years after construction should provide enough time for tidal exchange to develop naturally. Therefore, Alternative 3 is expected to meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20*.

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Approximately 36 acres of dune habitat will remain following the first 10-year storm which was predicted to occur in TY7. As with Alternatives 1 and 2, Alternative 3 is composed of beach option 1, which is designed to meet this objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Alternative 3 meets this objective with 60% of the subaerial acreage remaining by TY20. It is anticipated that 202 acres of subaerial acreage will remain in TY20 compared to 339 subaerial acres following construction in TY1.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline.*

   The beach cross-section was developed by adding equilibrium losses and seven years of advanced fill to the design cross-section. Furthermore, the location of the constructed shoreline was designed such that the projected 2014 shoreline would not be exposed to the Gulf over the 20-year project life. Alternative 3 meets this objective.

18.3.4 Alternative 4

Alternative 4 is comprised of beach option 2 and marsh option 2. Beach option 2 is constructed to +6.0 feet, NAVD with a crest width of 445 feet. Marsh option 2 is constructed to +2.5 feet, NAVD and is one of the larger marsh options but does not backfill the access channel during marsh construction. Alternative 4 was developed to allow comparison of a +8 feet, NAVD constructed dune crest and a wider but lower constructed dune crest at +6 feet, NAVD (with a similar beach volume). In order to allow comparison of the effect of the dune, Alternative 4 should be compared to Alternative 2.

Similar to beach option 1, beach option 2 will lower in elevation following construction due to settlement, historical subsidence, and overtopping during major storm events. Settlement of the dune as predicted by Fugro's analysis on the underlying substrate coupled with historical subsidence revealed that the dune will be lowered from a constructed elevation of +6.0 feet, NAVD to below +5.0 feet, NAVD by TY3. This results in a loss of dune acreage by conversion to supratidal habitat. Prior to the first major storm event in TY7, the dune is projected to have an elevation of +4.4 feet, NAVD. SBEACH modeling indicated that dune elevation will be lowered to approximately +3.4 feet, NAVD following the storm resulting in overwash onto the marsh platform. Although the beach crest elevation is below +5 feet, NAVD, there is sufficient sand remaining within the beach system to maintain a berm crest elevation that is higher than sand starved islands. After the second storm event in TY 14, it is estimated that the dune will have been lowered to the natural barrier island elevation. At this time, the dune system will maintain its elevation relative to MHW by building elevation at the rate of SLR.

Due to the lowering of the dune elevation, the active profile height is reduced. As a result, the gulf shoreline recession rate generally increases throughout the project life. The estimated recession rates for the analytic model included 13 feet/year during construction, 39 feet/year in TY2 (of which 25 feet is associated with the initial settlement of the dune), and 22 feet/year in TY8 following the first major storm event. Appendix F shows the input and output from the analytic model for each target year along with the various shoreline recession rates. Table 37 shows the expected planform performance projection of the various habitat types throughout the 20-year project life.

Table 37. Planform Performance Projection for Alternative 4

	Habitat (acres)					
Target Year	**Dune**	**Supratidal**	**Gulf Intertidal**	**Bay Intertidal**	**Subtidal**	**Total**
TY 0	1	10	18	97	70	196
TY 1	98	270	21	22	44	455
TY 2	74	46	21	240	44	425
TY 3	0	113	21	239	44	417
TY 4	0	107	21	238	44	410
TY 5	0	101	21	238	43	403
TY 6	0	95	21	237	43	396
TY 7	0	90	21	237	43	391
TY 8	0	139	21	180	43	383
TY 9	0	134	21	179	43	377
TY 10	0	129	21	177	43	370
TY 11	0	124	21	175	43	363
TY 12	0	121	21	172	42	356
TY 13	0	117	20	169	42	348
TY 14	0	114	20	166	42	342
TY 15	0	110	20	163	42	335
TY 16	0	106	20	160	42	328
TY 17	0	103	20	157	42	322
TY 18	0	100	20	154	42	316
TY 19	0	97	20	150	41	308
TY 20	0	94	20	147	41	302

The loss of dune acreage in TY2 results in the conversion of acreage in TY3 to supratidal habitat. The first storm event in TY7 results in material being transported landward converting bay intertidal acreage into supratidal habitat. Following the storm, the dune elevation is lowered to approximately the natural barrier island elevation and annual overwash is initiated. The annual overwash of sediment from the gulf face onto the marsh platform results in a loss of bay intertidal acreage and an increase in supratidal habitat. The annual overwash distance following the first storm event averaged 7.7 feet/year.

Alternative 4 is estimated to provide a net of 230 AAHU's (Table 41). This is less than Alternative 2 because of the loss of dune habitat in TY2 compared to TY14 for Alternative 2 (see Appendix F for detailed analysis). While Alternative 4 has the third highest cost, it is the least cost effective alternative at $150,300/AAHU due mainly to the early loss of dune acreage by TY3 and subsequent decrease in HSI, compared to the other alternatives that maintain dune acreage through TY7.

Alternative 4 should be compared to Alternative 2 in order to assess the effect of the constructed dune height. A discussion of each project goal and objective follows:

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

Alternative 4 meets this goal though not to the same degree as other alternatives as the dune acreage is only preserved through TY2. However, sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 satisfying island longevity and shoreline acreage. In addition, 147 acres of back barrier marsh acreage exist in TY20 which helps provide intended ecosystem function.

2. *Prevent island breaching over the 20-year project life.*

   The beach section of Alternative 4 was specifically designed to prevent breaching of the shoreline in TY20, without consideration of the benefit of the backing marsh, which will provide additional protection. Despite the lower elevation of the dune crest, sufficient volume is being placed to prevent breaching. Therefore, Alternative 4 should meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 238 acres of bay intertidal marsh at TY4 and gapping of the primary dike is expected to assist with the development of tidal exchange. Therefore, Alternative 4 should meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20*.

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Due to high settlement rates of the underlying soils, a dune constructed to +6 feet NAVD cannot maintain dune acreage following a 10-year storm. Therefore, Alternative 4 does not meet this objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Subaerial acreage remaining at TY20 (261 acres) is 64% of the 411 subaerial acres following construction at TY1. Therefore, Alternative 4 should meet this objective.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline*.

   The beach section is designed with the shoreline located 468 feet seaward of the 2014 shoreline. Due to the expected shoreline recession rates post-construction, it is anticipated that the Gulf shoreline will be 9 feet landward of the pre-construction shoreline, which just misses meeting this project objective.

18.3.5 Alternative 5

Alternative 5 is comprised of beach option 3 and marsh option 1. This alternative is best compared to Alternative 1 to compare the effect of the different beach option (decreased dune width) on project performance. Beach option 3 is constructed with a dune crest elevation of +8.0 feet (NAVD) and a dune crest width of 150 feet compared to +8 feet, NAVD and 270 feet wide for Alternative 1. Beach option 3 requires the least volume of any beach option with the objective of having the shoreline in a similar position at TY20 as the 2014 shoreline position. The marsh platform is the same as that of Alternative 1 and requires back filling of the access channel.

Due to settlement, consolidation, and historical subsidence, the constructed marsh platform is lowered to an elevation within the bay intertidal habitat range and the area associated with the access channel is lowered to subtidal habitat. The marsh platform within the footprint of the access channel experiences a greater change in elevation due to the thicker lift of placed material required to fill the channel that was excavated to -7.0 feet (NAVD). Table 38 shows the expected planform performance projection of the various habitat types throughout the 20-year project life.

Table 38. Planform Performance Projection for Alternative 5

	Habitat (acres)					
Target Year	Dune	Supratidal	Gulf Intertidal	Bay Intertidal	Subtidal	Total
TY 0	1	10	18	97	70	196
TY 1	63	324	20	20	36	463
TY 2	40	44	20	293	35	432
TY 3	34	42	20	292	35	423
TY 4	30	40	20	291	35	416
TY 5	26	38	20	291	35	410
TY 6	22	36	20	290	35	403
TY 7	18	35	20	289	35	397
TY 8	0	160	20	175	35	390
TY 9	0	154	20	174	35	383
TY 10	0	149	20	172	35	376
TY 11	0	144	19	170	34	367
TY 12	0	140	19	167	34	360
TY 13	0	136	19	164	34	353
TY 14	0	132	19	162	34	347
TY 15	0	128	19	159	34	340
TY 16	0	125	19	156	34	334
TY 17	0	122	19	153	34	328
TY 18	0	119	19	150	34	322
TY 19	0	116	19	147	34	316
TY 20	0	113	19	143	33	308

Due to settlement, historical subsidence, and a major storm event in TY7, dune acreage is lost by TY8. The natural processes resulted in a conversion of dune acreage to supratidal habitat. The overwash associated with the storm event resulted in the conversion of bay intertidal habitat comprising the marsh platform to be converted to supratidal acreage.

Alternative 5 is estimated to provide a net of 234 AAHU's (Table 41). This is less that of Alternative 1 because although the marsh platform is the same, beach option 3 has a smaller footprint and constructed starting acreage (see Appendix F for detailed analysis). At $134,500/AAHU, Alternative 5 is the most cost effective alternative of the six alternatives carried forward for evaluation.

A discussion of the performance of Alternative 5 with respect to the project goal and objectives follows:

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

   Alternative 5 meets this goal as sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 satisfying island longevity. Dune acreage is preserved through TY7. Alternative 5 also includes the largest marsh that assists with island longevity. The decreased fill volume suggests that the shoreline condition at TY20 will contain a similar volume of sand along the gulf face that is present at the time of construction. Alternative 5 is deemed to meet the project goal.

2. *Prevent island breaching over the 20-year project life.*

   Alternative 5 has 20 years of advanced fill so after 20 years it is expected that the shoreline will return to its pre-construction condition. Since there are existing breaches in the island, it is important that the backing marsh still be in place. Given that Alternative 5 has the largest marsh and it is very wide, it is expected that the island will not breach within the project lifetime. Furthermore, the marsh will act as a catch basin for overwash, which will increase the resistance to breaching. It is anticipated that Alternative 5 will meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 291 acres of bay intertidal marsh at TY4. As with previously discussed alternatives, Alternative 5 should meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20*.

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Due to the reduced beach width, a 10-year storm event in TY7 is anticipated to overwash the beach lowering the dune crest elevation below +5 feet, NAVD. Thus following the storm, it is not anticipated that dune elevation will be maintained so Alternative 5 is not expected to meet this project objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Subaerial acreage remaining at TY20 (275 acres) is 55% of the 499 subaerial acres anticipated at TY1. Therefore, Alternative 5 should meet this objective.

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline*.

   The beach section was developed by holding the landward toe of fill and using only advanced fill volume requirements. Therefore, the TY20 shoreline will be approximately 177 feet landward of the pre-construction shoreline, which means it will not meeting this project objective.

18.3.6 Alternative 6

Alternative 6 is comprised of beach option 3 and marsh option 3. The constructed dune crest elevation is +8.0 feet, NAVD with a crest width of 150 feet and a marsh option that closely resembles the Phase 0 marsh concept. Marsh option 3 is similar to marsh option 4 applied in Alternative 3 except the access channel is included within the marsh footprint. Marsh option 3 requires the channel to be backfilled during marsh construction resulting in greater volume of marsh material and greater cost to construct the primary dike. Despite the cost to backfill the channel, Alternative 6 is the lowest cost alternative due to the smaller volume of sand required to construct beach option 3.

Dune elevation will be maintained until the first 10-year storm event, anticipated in the model to occur by the end of TY7. Following this storm, dune acreage will be converted to supratidal habitat and bay intertidal will be converted to supratidal due to overwashed sediment, as highlighted in Table 39.

Table 39. Planform Performance Projection for Alternative 6

	Habitat (acres)					
Target Year	Dune	Supratidal	Gulf Intertidal	Bay Intertidal	Subtidal	Total
TY 0	1	10	18	97	70	196
TY 1	63	263	20	29	54	429
TY 2	40	44	20	242	54	400
TY 3	36	42	20	241	54	393
TY 4	32	40	20	241	53	386
TY 5	29	38	20	240	53	380
TY 6	26	37	20	240	53	376
TY 7	23	35	20	240	53	371
TY 8	0	175	20	117	53	365
TY 9	0	169	20	117	53	359
TY 10	0	165	20	115	53	353
TY 11	0	161	20	114	53	348
TY 12	0	158	20	112	52	342
TY 13	0	155	19	110	52	336
TY 14	0	152	19	108	52	331
TY 15	0	149	19	106	52	326
TY 16	0	146	19	103	52	320
TY 17	0	144	19	101	52	316
TY 18	0	141	19	98	52	310
TY 19	0	139	19	95	51	304
TY 20	0	137	19	93	51	300

Alternative 6 is estimated to provide a net 209 AAHU's. Although Alternative 6 has a larger marsh footprint than Alternative 3, the constructed beach is smaller resulting in the fewest number of net AAHU's (209) (see Appendix F for detailed analysis). The large marsh and small beach result in Alternative 6 being the second most cost effective alternative at $137,500/AAHU.

A discussion of how Alternative 6 addresses each project goal and objective follows:

1. *Reestablish and maintain a functional barrier island ecosystem for fish and wildlife habitat by restoring and creating shoreline, dune and back-barrier marsh acreage.*

   Alternative 6 meets this goal as sufficient fill volume is placed to have a sandy beach face remaining at the end of TY20 satisfying island longevity. Dune acreage is preserved through TY7. Alternative 6 has a backing marsh that will assist with maintaining the longevity of the project and providing ecosystem function. Alternative 6 meets this goal.

2. *Prevent island breaching over the 20-year project life.*

   Alternative 6 has 20 years of advanced fill, so after 20 years it is expected that the shoreline will return to its pre-construction condition. Since there are existing breaches in the island, it is important that the backing marsh still in place. Although Alternative 6 has the second smallest marsh and with the narrowest beach width, it is expected that the island will not breach within the project lifetime. Furthermore, the marsh will act as a catch basin for overwash, which will increase the resistance to breaching. It is anticipated that Alternative 6 will meet this objective.

3. *Provide an intertidal marsh platform with tidal exchange by Target Year 4.*

   This alternative will result in approximately 241 acres of bay intertidal marsh at TY4. Again, the marsh elevation and size is sufficient to meet this objective.

4. *Maintain an elevation of greater than +4 feet NAVD at Target Year 20*.

   This objective cannot be met under the current budget constraints. A beach/dune design may be developed to meet this objective for a significant cost increase.

5. *Maintain a dune elevation of greater than +5 feet NAVD following the first 10-year storm event.*

   Due to the reduced beach width, a 10-year storm event in TY7 is anticipated to overwash the beach lowering the dune crest elevation below +5 feet, NAVD. Thus following the storm, it is not anticipated that dune elevation will be maintained so Alternative 6 does not meet this project objective.

6. *Maintain 50% of the Target Year 1 subaerial acreage throughout the 20-year project life.*

   Alternative 6 meets this objective because 66% of the TY1 subaerial acreage (375 acres) remains at TY20 (249 acres).

7. *Maintain the Target Year 20 shoreline seaward of the pre-construction shoreline*.

   The TY20 shoreline will be approximately 177 feet landward of the pre-construction shoreline so Alternative 6 does not meet this particular project objective.

18.4 Project Performance Summary

The No Action Alternative does not meet any of the project objectives. Continued degradation of Chenier Ronquille should be expected.

The six proposed alternatives all meet the goal of reestablishing and maintaining a functional barrier island ecosystem for fish and wildlife habitat. All of the six alternatives meet the project objectives of preventing breaching over the 20-year project life, providing intertidal marsh with tidal exchange by TY4, and maintaining 50% of the TY1 subaerial acreage over the 20-year project life (Table 40). After analyzing the settlement of the soils underlying the proposed dune, it was determined that none of the alternatives would maintain elevation of greater than +4 feet, NAVD at TY20. An objective was developed that required a dune elevation of greater than +5 feet, NAVD to be maintained following the first 10-year storm event. All alternatives constructed with a +8 feet, NAVD dune crest meet this objective but Alternative 4, with a dune crest constructed +6 feet, NAVD, does not. Alternatives 1, 2, and 3 meet the project objective of maintaining the TY20 shoreline seaward of the pre-construction shoreline. This is achieved by the more robust beach option 1.

Table 40. Summary of Alternative Performance with Respect to Project Objectives

Objective	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
1. Prevent breaching over 20-year project life	Yes	Yes	Yes	Yes	Yes	Yes
2. Provide intertidal marsh with tidal exchange by TY4	Yes	Yes	Yes	Yes	Yes	Yes
3. Maintain elevation of greater than +4' NAVD at TY20	No	No	No	No	No	No
4. Maintain dune elevation of greater than +5' NAVD following first 10-year storm event	Yes	Yes	Yes	No	No	No
5. Maintain 50% of the TY1 subaerial acreage throughout 20-year project life	Yes	Yes	Yes	Yes	Yes	Yes
6. Maintain TY20 shoreline seaward of pre-construction shoreline	Yes	Yes	Yes	No	No	No

Table 41. Quantitative Summary of Project Performance

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Years Dune Elevation (> +5.0 ft, NAVD) (yr)	14	14	14	2	7	7
Total Constructed Subaerial Acreage (acres)	361	307	237	318	337	277
Cost/Constructed Acre ($)	$104,700	$112,100	$134,000	$108,600	$93,300	$103,600
TY20 Net Acreage (acres)	290	238	182	241	256	230
Cost/Net Acre ($)	$130,362	$144,744	$174,593	$143,444	$122,922	$124,913
Net AAHU's	263	236	216	230	234	209
Cost/Net AAHU ($/AAHU)	$143,745	$145,970	$147,111	$150,304	$134,479	$137,464

The wide beach option at +8 feet, NAVD (Alternatives 1, 2 and 3) is expected to maintain dune acreage through to TY14. The lower beach (Alternative 4) even with the same volume may lose all dune acreage by TY 3 due to settlement. The advanced fill only beach, constructed to +8 feet, NAVD, is expected to maintain dune elevation up to TY7 at which time the first 10-year storm event is anticipated to overwash the beach and lower the crest elevation below +5 feet, NAVD.

All alternatives have a constructed marsh elevation of +2.5 feet, NAVD. Given the rate of settlement, it is expected that the marsh area will be considered bay intertidal habitat by TY 2.

Alternative 1 provides the most constructed subaerial acreage (361 acres), resulting from the combination of the largest beach and marsh options, as well as the most net acres at TY20 (290 acres). Following Alternative 1, the largest constructed subaerial acreage and total net acreage is Alternative 5. Alternative 4 follows with the lower, wider dune section (+6 feet NAVD). Alternative 2 has the fourth greatest constructed subaerial total followed by Alternative 6, which was developed to be the lowest cost alternative that will still meet the project goal. Alternative 3, which pairs the largest beach option with the smallest marsh option, results in the lowest constructed subaerial and net acreages. Alternative 5 is the most cost effective alternative with the lowest cost per constructed acre at $93,321 and the lowest cost per net acre at $122,922.

Alternative 1 provides the greatest number of AAHU's, as would be expected as it has the largest marsh and beach options (Table 40). There are further decreases in AAHU's as the size of the marsh is decreased for Alternative 2 and then again to Alternative 3, which has the second lowest AAHU's of any alternative. Alternative 4 has slightly fewer AAHU's than Alternative 2 though they have almost identical marsh and beach fill volumes. The difference is due to the higher construction dune elevation in Alternative 2 providing greater benefits than the lower but wider constructed dune crest of Alternative 4. Alternative 5 highlights that a large marsh with a smaller beach can provide a commensurate performance from a WVA perspective. Without a large marsh or a large beach, performance is lower as shown by Alternative 6.

19 CONSTRUCTION

19.1 Construction Methodology

Construction will require the hydraulic placement of both beach and marsh fill within the project area. A cutterhead dredge will be used for construction of both the beach and marsh fill components of the project, given the proximity of the borrow areas to the project area and the relatively shallow bathymetry in the borrow areas. It is unlikely that a booster pump will be required as the borrow areas are within 3 miles of the project area.

Construction of the beach and marsh fill will require the use of heavy machinery to manage the pipeline and construct containment dikes. The existing island provides a sufficient base for equipment deployment with placed fill used to supplement existing material for containment.

The access channel at the west end of the island will provide a passageway to the construction footprint. The channel will extend from the -7.0 feet, NAVD contour within Quatre Bayou Pass and join the borrow source for the primary dike. Contractors have indicated on other projects that an access channel with minimum dimensions of 80 feet wide and 6 feet deep (-7 feet, NAVD) will be required. A staging area will be delineated along the south side of the access channel at the west end of the island where there is existing subaerial acreage to land barges and offload equipment, crew, and shore pipe.

Beach fill will be delivered hydraulically to the project area via a submerged pipeline. The submerged pipeline is transported to the site on pontoons in approximately 500-foot sections. Once in the vicinity of the project area, the various sections of pipeline are joined together into lengths of up to 2,500 feet. Once sufficient lengths of submerged pipeline are joined (the pieces are connected by ball joints), the pipeline is floated into position and the 2,500-foot sections of submerged pipeline are joined. The connected pipeline is then allowed to sink to the bottom. Floating pipeline is attached to the submerged line at the borrow area while the end of the submerged line is dragged ashore in the project area. Once the submerged line is in place, the dredge will be connected to the floating line and traverse the borrow area to mine sediments. Shore pipe will be added to the end of the discharge pipe as the beach fill progresses alongshore. It is anticipated that the dredge Contractor will use Bay Long as a temporary storage area for the submerged line and connect the various sections in the lee of the Chenier Ronquille.

The Contractor will construct training dikes extending in a shore parallel direction to manage or partially contain the discharge of beach fill material and minimize offshore losses. The sand will settle out while the water returns to the Gulf of Mexico at the end of the dikes. The dikes are made of sand that has already been pumped to the beach. These dikes are construction features that are several hundred feet long. Once the beach fill has been filled to grade, the shore pipe will be extended by adding additional pipe to the end. The dikes will be leveled and the beach graded to the required construction slope. While the Contractor has some control of the fill above mean low water, where the bulldozers can manage the sand, the Contractor has limited control of the fill below mean low water. The only method that the Contractor has to control the beach slope below mean high water is to alter the length of the dikes.

It is expected that the Contractor will construct the beach in an eastward direction from the west end of the beach fill terminus. While constructing the beach, it is anticipated that hydraulically placed beach fill material will naturally plug the breaches along the existing shoreline. It is not expected that sand will have to be directly pumped into the breaches during construction. Due to the shallow water depths and limited tidal connectivity to the Gulf, additional measures to close the breaches such as stockpiling sand adjacent to the breach and then bulldozing it closed during low tide should not be required. At the western and eastern ends of the beach fill, the Contractor will be permitted to use closed containment to reduce the loss of material outside the construction footprint and into Quatre Bayou Pass and Pass La Mer, respectively. Closed containment will involve constructing containment dikes with water control structures installed to contain the discharged material while allowing excess water to drain into the Gulf.

Marsh fill will also be pumped hydraulically to the project area. The beach will act as the southern limit of fill while primary dikes will contain the north, east, and west sides of the marsh fill. A flexible PVC pipe will be used to distribute the marsh fill. Marsh buggies will be used to move the end of the discharge in order to uniformly fill the marsh area. If the fill material has a high percentage of sand, the PVC pipe may be replaced by steel pipe, previously used during beach fill placement, as the PVC pipe is less durable and susceptible to damage while pumping sand rich slurries. The Contractor may opt to construct internal training dikes within the marsh platform to assist with the settlement of the material, though the cost of these dikes is not included as a bid tab item.

The primary dikes are included as a line item on the bid tab. The primary dike will be constructed using a clam shell or bucket dredge. The dike fill will be excavated from the designated access channel near the north boundary of the marsh fill boundary. The dikes will be constructed to a crest elevation of +5 feet, NAVD with a minimum crest width of 5 feet. This is the minimum section and the Contractor will be allowed to construct a larger primary dike if need be. The primary dike fill source has approximately double the volume of the primary dike. The eastern and western portions of the dike will be built on top of the existing marsh platform, which will require less volume to construct than the dikes constructed in the open bay area near the middle portion of the project area. The contractor may construct additional secondary dikes within the marsh footprint to control the fill. The extent and location of the secondary dikes will be at the Contractor's discretion and cost. Water control structures will be required to allow marsh fill material to settle out of the water column and to drain excess water from the fill area. Three locations for water control structures are proposed with the preferred discharge being the westernmost one, as it is located behind a more vegetated area that can help the fine material to fall out of suspension.

19.2 Construction Sequence

The construction sequence will be at the discretion of the Contractor. However, this section presents a construction scenario to evaluate the feasibility of the project.

The constructed beach serves as the southern dike for the marsh fill, so it will be constructed first. It is expected that the Contractor will construct the entire beach first because the S-1 and a portion of the D-1 borrow areas contain adequate volumes of surficial sands. It is not anticipated that overburden removal will be necessary to access beach compatible material. The S-1 borrow area will be excavated first, as it has a lower silt content. Once dredging of S-1 is complete, the contractor will relocate to D-1. Finishing the beach fill while in D-1 will limit any additional movement of the dredge to access marsh material, reducing relocation costs and down time. Additional marsh material will be dredged from the Quatre Bayou Disposal Area. The primary containment dikes along the north, east, and west sides of the marsh will be constructed concurrently with the beach fill.

Depending on the alternative chosen, the access channel may or may not be backfilled during marsh construction. If the primary dike is constructed south of the channel, then the channel will be located to the north of the marsh footprint and will not be filled while material is hydraulically placed during marsh construction. On the other hand, if the dike is constructed to the north of the channel, then the channel will be within the marsh footprint and backfilled during construction. Backfilling the access channel will require additional marsh compatible material to fill the template. The volume of material excavated from within the marsh footprint to construct the primary dike has been included in the marsh fill volumes for each of the alternatives for which the channel will be backfilled.

Limited gapping of the primary dike may occur once the marsh fill has been accepted. The number and location of these gaps will be determined in the field at the end of construction. The gaps will be located near lower sections of the constructed marsh in order to assist with drainage. Additional gapping may be required as a future maintenance event if initial gapping or natural erosion prove insufficient.

19.3 Anticipated Construction Rates

Production rates for various barrier island restoration projects were analyzed to develop an expected production rate for Chenier Ronquille. Production rates are important in determining cost estimates as well as project timelines. A discussion of production rates on various projects follows.

At Holly Beach (Holly Beach Sand Management Project (CS-01)), beach fill production rates in excess of 30,000 cubic yards/day were recorded for the dustpan dredge Beach Builder, but the average production rate, including all downtime, was 11,970 cubic yards/day (CPE, 2003a). This project experienced greater downtime than would typically be expected for a beach construction project because the dredge had to be removed from the project site twice due to the passages of Hurricanes Lili and Isidore, the submerged pipeline had to be fixed on numerous occasions, and the dustpan had to be cleared of clay frequently (at least twice daily). The borrow area was located approximately 5 miles offshore with another 3 miles of shore pipe for a maximum pipe length of 8 miles. A booster pump was placed in the line to assist the dredge in pumping material this distance. The maximum pumping distance at Chenier Ronquille Island is slightly shorter (approximately 4.5 miles) than at Holly Beach so maximum production rates will be higher.

Chaland Headland is located immediately adjacent to Chenier Ronquille, on the east side of Pass La Mer. During construction of Chaland Headland Restoration Project (BA-38-2), the beach fill production rates were approximately 23,300 cubic yards/day while operational though with down time of 29% reduced this to 16,600 cubic yards/day (based on pay volume) (CPE, 2008). The marsh fill production rate was higher at 65,600 cubic yards/day though stoppages reduced the average to 44,400 cubic yards/day. The Quatre Bayou borrow area used to construct the project was located only 3 miles from the project area, which is comparable to the distance to the borrow areas proposed for Chenier Ronquille. Thus, the beach and marsh fill production rates are applicable for the current project. The 30-inch cutterhead dredge “Tom James” (recently renamed the “Captain Frank”) was used on the Chaland Headland project.

For the Pass Chaland to Grand Bayou Pass Restoration Project (BA-35), the production rate for fill placed within the template is approximately 24,300 cubic yards/day (E-mail correspondence with Barry Richard and Michael Poff, 3/30/09). The borrow area is approximately 8 miles from the project site with an additional 3 miles of shore pipe required. This is longer than the pumping distance to the Chenier Ronquille Island Restoration project and thus production rates are expected to be higher. The cutterhead dredge “Alaska” was used to construct the project.

East Grand Terre Island is located just west of Chenier Ronquille, on the west side of Quatre Bayou Pass. During construction of East Grand Terre Island Restoration Project (BA-30), the beach fill production rates were approximately 25,600 cubic yards/day while operational though with down time of 46% reduced this to 13,800 cubic yards/day (based on placed volume) (CPE, 2011). The borrow area used during beach construction was located approximately 3.3 miles from the project site with a maximum pumping distance of 5.0 miles. After beach construction had commenced, a single booster pump was installed in the submerged line to increase production rates. The marsh fill production rate was higher at 41,000 cubic yards/day though stoppages reduced the average to 24,000 cubic yards/day (based on placed volume) (CPE, 2011). The marsh fill borrow areas were located approximately 1.8 miles for the project site with a maximum pumping distance of 3.9 miles. The 30-inch cutterhead dredge "Captain Frank" that was used to construction Chaland Headland was also used during construction of East Grand Terre Island. The pumping distance for East Grand Terre Island marsh fill work is similar to that for Chenier Ronquille, thus beach and marsh fill production rates are expected to be approximately equal.

To estimate the production rate for Chenier Ronquille, the beach and marsh fill production rates from Chaland Headland Restoration and East Grand Terre Island Restoration projects were averaged. A beach fill production rate of 15,200 cubic yards/day and marsh fill production rate of 34,200 cubic yards/day were used to develop a construction timeline, which includes downtime.

19.4 Conceptual Construction Timeline

It generally takes two to three weeks following issuance of the Notice to Proceed, for the Contractor to mobilize to the site and start performing the pre-construction survey. The pre-construction survey will require approximately 13 miles of surveys to be performed, which will require 3 weeks to complete. Therefore, construction could commence within 6 weeks of the Notice to Proceed.

Once the surveys are complete, it is assumed that beach fill will begin immediately. Mobilization of equipment to the project area, and placement of the submerged line, can occur while surveying is ongoing. A beach fill production rate of 15,200 cubic yards/day has been assumed for this work though the actual pumping rates may be different given the variety of equipment available to perform the work and unforeseeable weather delays. Assuming this pumping rate, the construction durations to complete the beach fill were estimated (Table 42). The difference in construction days is due to the difference in beach fill volume.

Table 42. Estimated Beach Fill Construction Durations

Design Alternative	Beach Volume (cy)	Duration (days)
1	1,830,000	121
2	1,830,000	121
3	1,830,000	121
4	1,840,000	122
5	1,310,000	87
6	1,310,000	87

It is assumed that marsh fill will be started 2 days after the completion of beach fill to allow for relocating the dredge and submerged line to the marsh borrow area. It is also assumed that no additional contract time is needed to construct the primary dikes because they will be constructed concurrently with the beach fill. Given a pumping rate of 34,200 cubic yards/day, the marsh fill construction durations were estimated (Table 43).

Table 43. Estimated Marsh Fill Construction Durations

Design Alternative	Marsh Volume (cy)	Duration (days)
1	1,380,000	41
2	940,000	28
3	590,000	18
4	940,000	28
5	1,380,000	41
6	1,020,000	30

A waiting period of 30 days will be required prior to surveying of the final marsh platform section. The final survey will require less than a week to complete because the majority of the surveys will have been completed during construction of the project. Monitoring surveys will be performed after construction, according to the monitoring plan. Typically these surveys occur at TY 1, 3, 5, and 20.

It is likely that the Contractor will have demobilized most of their equipment by this point. However, for estimating construction time, an additional 40 days should be added to allow for the potential for the Contractor to address any deficiencies and again wait for the 30 day period to conclude. Therefore, the demobilization period should extend 70 days beyond the completion of dredging.

Finally, an additional 60 days of contract time should be included in the contract time to allow some flexibility in starting time for the Contractor. This additional contract time will potentially reduce the bid costs because the Contractor has less risk of encountering liquidated damages and can better stage his work between various projects. Delays due to significant weather events will be addressed in the specifications to allow an extension in contract time and is not considered as part of the contract time discussed in this paragraph.

The total recommended contract time for each of the alternatives is shown in Table 44. The recommended contract times ranged from 289 to 334 days.

Table 44. Recommended Contract Time

	Key Construction Durations (days)					
Design Alternative	Mobilization/ Pre-Construction Survey	Beach Fill	Marsh Fill	Demobilization	Start Date Flexibility	Total Contract Time
1	42	121	41	70	60	334
2	42	121	28	70	60	321
3	42	121	18	70	60	311
4	42	122	28	70	60	322
5	42	87	41	70	60	300
6	42	87	30	70	60	289

20 CHANGES FROM PHASE 0 APPROVAL

20.1 Summary of Phase 0 Design

The Phase 0 report (Thomson and Wycklendt, 2009) proposed a 265-foot wide dune crest at +6 feet, NAVD. A 1V:30H construction slope above +1 feet, NAVD and a 1V:60H slope below +1 feet, NAVD was proposed. The beach fill volume was approximately 1,471,000 cubic yards and was estimated to create approximately 102 acres of sandy dune, supratidal and gulf intertidal acreage.

The Phase 0 design included a backing marsh that would create 145 acres of marsh habitat with a total footprint of 259 acres. A fill elevation of +3.0 feet, NAVD was proposed resulting in an estimated fill volume of 1,596,000 cubic yards.

The proposed borrow areas were the ones remaining following construction of the East Grand Terre and Chaland Headland Restoration projects. It was estimated that 3.9M cubic yards of beach fill and 4.9M cubic yards of marsh fill would be available.

A project cost of $33.1M was presented in the Phase 0 report, which included a 25% contingency.

20.2 Changes to Phase 0 Design

There have been several changes to the design from the Phase 0 report. These are described below.

1. The length of the beach fill has been shortened from the Phase 0 report in response to expected erosion and regression of the western island tip and to control losses off the western end of the island. This has reduced the beach fill volume and thus the cost.

2. The dune crest elevation for five of the six alternatives has been increased in elevation from +6 feet, NAVD to +8 feet, NAVD. Additional analyses suggested that the settlement of the dune was underestimated in the Phase 0 report.

3. Three additional marsh fill footprints have been added to the Phase 0 preferred option (Marsh options 1, 2 and 4). Marsh Option 3 has revised the primary dike location slightly from the Phase 0 report.

4. The elevation of the marsh fill has been reduced from +3 feet to +2.5 feet, NAVD. This change was made because the marsh fill area is shallower than was estimated during development of the Phase 0 report. This reduced the expected settlement of the fill and thus allowed for a reduced fill elevation. This has resulted in a decrease in the marsh fill volume.

5. The access channel location has been revised to access the project area from the west end of the project rather than from the northwest. This change was required due to shallow oil infrastructure in the vicinity of the access channel.

6. The unit costs for dredging marsh and beach fill were increased by $0.50 over the unit costs in the Phase 0 Report. The Phase 0 Report used the costs for the East Grand Terre project and it was decided to increase the costs due to inflation. The unit costs for the primary dike were changed to linear foot.

21 RECOMMENDATIONS AT THE 30% DESIGN LEVEL

It is recommended that Alternative 1 be constructed to best fulfill the project goal and objectives. The design of Alternative 1 follows standard coastal engineering design principles of including a design section and sufficient advanced fill to last over the project life (20 years). Alternative 1 involves the construction of a 1,830,000 cubic yard beach fill template and a 1,380,000 cubic yard marsh fill template. The proposed construction dune crest elevation is +8.0 feet, NAVD while the constructed marsh elevation is +2.5 feet, NAVD. Along with the highest constructed subaerial acreage (361) and net acreage (263), Alternative 1 has the highest net benefit of 263 AAHU's. The expected construction cost for Alternative 1 is $37,805,000 including a 25% contingency.

If this option is not chosen due to cost considerations, Alternative 5 is the second recommended option. Alternative 5 is preferred over the remaining alternatives as it the most effective alternative with respect to cost/AAHU. Furthermore, no pipelines have to be crossed to construct the primary dike. It provides the largest marsh, which will minimize the potential for breaching, especially as the cost savings have been realized by reducing the beach fill volume. Alternative 5 also provides the lowest cost per constructed subaerial acre and the lowest cost per net acre. The expected construction cost for Alternative 5 is $31,468,000 including a 25% contingency.

Borrow areas to construct the project are located approximately 1.5 to 3 miles southwest of the project area. These borrow areas were developed for the East Grand Terre and Chaland Headland Restoration projects and contain sufficient fill material remaining to construct any alternative.

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