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Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Coastal Processes Assessment and Project Re-Evaluation: Grande Isle, Louisiana, Shore Protection Project

July 2004

By

Brian K. Batten, Bruce A. Ebersole, David B. King and Ernest R. Smith

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Final report

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Contents

List of Figures

List of Tables

1 Introduction

The U.S. Army Engineer District, New Orleans (MVN) requested the technical assistance of the U.S. Army Engineer Research and Development Center's (ERDC) Coastal and Hydraulics Laboratory (CHL) to examine the past performance of the shore protection project at Grand Isle, Louisiana, and to propose a solution to the localized erosion problems that persist. The shore protection project was originally constructed in 1983 and 1984. Since that time actions to maintain the project have included occasional beach fill placement which has been done to address localized erosion problems, regular raking to gather beach debris and placement of the debris near the dune line to promote dune growth, and through construction of groins and detached breakwaters. The project presently suffers from severe erosion within a mile-long section on the eastern half of the island, where the project's dune has been heavily damaged on a number of occasions, and from local erosion hot-spots at a few other locations. This report summarizes the findings of the project re-evaluation study.

Study Objectives

The primary objective of the study was to make recommendations for needed improvements to the project, specifically a beach fill design for the critical erosion area. The technical portions of the study were accomplished through three interrelated tasks. These are:

Task 1: Geomorphic analysis (which is discussed in detail in Chapter 2).

Task 2: Short-term modeling of storm-induced beach erosion (which is discussed in detail in Chapter 3).

Task 3: Long-term modeling of longshore sediment transport and shoreline change (which is discussed in detail in Chapter 4).

A general statement of work that was performed under each of these three tasks is described below and in subsequent chapters. Chapter 5 of this report presents a summary of results, conclusions, and recommendations arising from this study.

Task 1: Geomorphic analysis

Historic data for this region of the Louisiana coast were sought (aerial photos, beach profiles and other topographic/bathymetric surveys, grain size data, wave and water level data, etc.); and they were acquired from a number of sources: the State of Louisiana (Geological Survey, LGS, Department of Natural

Resources, DNR, and Department of Transportation, LADOT), the Town of Grand Isle, the New Orleans District, contractors working for the Town and the State, and from local universities, Louisiana State University (LSU) and the University of New Orleans (UNO). Spatial data, such as aerial photographs, shoreline position data provided by other organizations, and bathymetric surveys, were integrated into a Geographic Information System (GIS) using the commercially available ArcMap software product. Where necessary, shoreline positions were extracted from aerial photos. Shoreline position data were crucial to the analyses performed in this study, and extraction was done very carefully to maximize accuracy and consistency. Beach profile data were entered and organized within EXCEL and the Beach Morphology Analysis Package (BMAP) for further analyses. Certain profile data were also entered into the GIS to facilitate extraction of shoreline position data. All spatial data in the GIS were geo-referenced to a consistent set of horizontal and vertical datums.

Shoreline position data were used to compute shoreline change rates, volume changes and rates of change; and then used to examine and quantify how those rates and the coastal morphology have evolved through time. Changes in shoreline and beach morphology which have occurred since construction of the project were examined and quantified to aid in designing a solution to the problem. Historical data also were analyzed to assess the influence of the Town's Rock Project (TRP), the Federal breakwaters, and the State (LADOT) breakwaters, on shoreline and beach changes on the island. Data sets were used to estimate the amount of sand that has begun to bypass the TRP during recent years, as well as the amount that enters the Grand Island littoral system across the Caminada ebb shoal. The shoreline changes and volume flux estimates at the eastern end of the TRP were used to develop the lateral boundary condition needed for the shoreline change modeling.

Beach profile data were used in the following ways: assess the grain size characteristics of the beach, ascertain the desired cross-sectional shape of a proposed beach fill, translate added fill volume into added beach width, and to support modeling of storm-induced erosion. This study task involved quantifying the grain size characteristics of the beach and potential borrow sites, assessing compatibility of borrow area sand with sand presently on the beach, and in estimating renourishment volumes for sand extracted from the different borrow areas.

Task 2: Storm erosion modeling

Wave and water level data for the most severe storms that have occurred and impacted the project since 1985 were sought and acquired from a number of sources: the U.S. Army Corps of Engineers (the Wave Information Study (WIS) Gulf of Mexico wave hindcast for 1990-1999, and a special high-resolution wave hindcast for this region done by the WIS staff in support of MVN for the years 1976 to 1995), the National Data Buoy Center (measurements from nearby deeper water buoys that are also supported by the WIS). LSU (wave gages they operate in the region), National Oceanic and Atmospheric Administration (NOAA) and the District (water level gages they operate at Grand Isle).

The SBEACH storm-induced beach erosion model was set up and applied for existing condition profiles to assess the degree of protection that is presently provided by the project, particularly in the critical area. The model was also set up and applied for altered profiles, reflecting changes in berm and dune geometry that are required to withstand the effects of the severe storms that have occurred since the project was constructed. Protection from the design storm, which was identified in the original project's General Design Memorandum (GDM 1979), also was examined. For a few representative profiles, selected severe storms were simulated for both a structure-free profile and profiles with a detached breakwater in place (when the breakwater becomes submerged during storms) to examine any added protection provided by the detached breakwaters.

Recommendations concerning the dune configuration and berm width that should be established and maintained in the critically eroded area are made. Calculations of the required cross-sectional fill volume density needed as a function of berm width and dune configuration are made.

Task 3: Shoreline change modeling

The nearshore wave transformation model STWAVE and the shoreline evolution model GENESIS were set up and applied for the eastern half of the island (STWAVE was set up for the entire island). The GENESIS domain was limited to half the island due to the amount of computational effort that was required, with the western GENESIS boundary located at the Town's Rock Project. The available wave hindcasts were used to characterize the offshore wave climate. Annual longshore sand transport rate variability along the eastern half of the island was assessed, and an average 4-year longshore transport climate (and the waves that produced it) was developed. The GENESIS model was calibrated and validated using recent shoreline position data that were processed and analyzed in Task 1.

The models were applied together to determine the initial beach planform (and volume) that effectively maintains the desired berm width during the 4-year renourishment interval. The GENESIS model was applied to evaluate the performance of the proposed fill. Sensitivity of the beachfill's planform evolution to above/below average longshore transport conditions was also investigated. Possible reduction in the renourishment volume that might result from changes to breakwater configuration was examined.

This application was a very demanding application of GENESIS, quite computationally intensive. Accurate resolution of the very short breakwaters required very fine 25-ft resolution, for nearly 20,000 ft of shoreline, which also required a short time step for the multi-year simulations.

2 Geomorphic Analysis

This chapter describes procedures for and results of analysis of aerial photography, shoreline data and beach profile data as outlined in Task 1 of the study objectives. This task is broken into six main topics: (1) shore protection structures, (2) beach profile data, (3) shoreline change rate analysis, (4) sediment budget, (5) borrow area evaluation, and (6) sand management recommendations. All data mentioned in this section were provided to CHL researchers from the New Orleans District office unless otherwise noted.

Shore Protection Structures

At present, 44 shore protection structures exist on the Gulf of Mexico coast of Grand Isle (Figure 2-1). The history, shoreline response, and present condition of the structures are reviewed in this section.

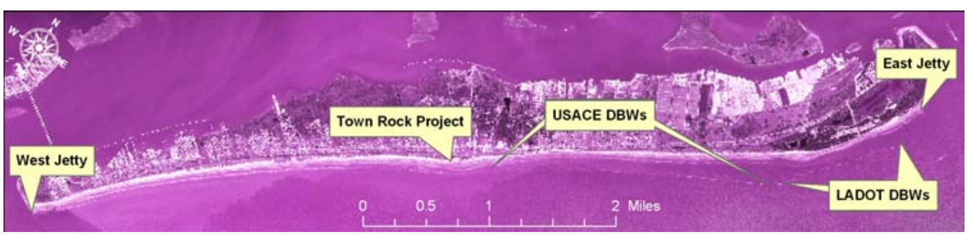

Figure 2-1. Shore protection structures along Grand Isle, LA

East and west jetties

The east jetty is the oldest structure on the island, constructed in 1958 and later extended in 1964 and 1984. Updrift and downdrift shorelines rapidly responded to the construction of the jetty. Within 4 years an estimated volume of one million cu yd had impounded updrift; and downdrift losses were estimated at 30 acres (Theis 1969). The impoundment fillet was used as a borrow source through the years, though this became limited after establishment of a State Park in the late 60's (Theis 1969; Combe and Soileau 1987).

The west jetty was constructed between 1971-72 in response to severe erosion along the western end of the island (HNTB 1993). The structure was later extended in 1987 (Combe and Soileau 1987). No information was found in the literature regarding the performance of the jetty. Dune and beach conditions adjacent to the structure appeared in good health during a site visit on 4 November 2003.

At present, both jetties are in poor condition and require rehabilitation. Examination of the literature indicated that maintenance has not been performed since 1987. Figure 2-2 shows the condition of the structures from a 11 July 2003 aerial photograph. Water level at the time of the photo appears to be at low-tide, given the position of the wetted-bound on the beach. Even at this low-tide stage, both structures appear porous to wave energy, with the inshore portion of the east jetty completely inundated. The condition of the jetties limits their ability to trap sand, which has implications on the stability of adjacent shorelines.

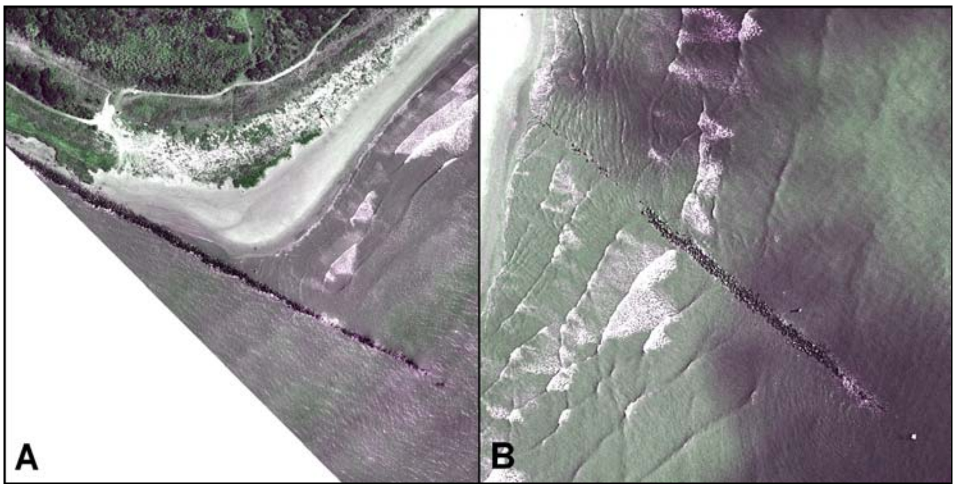

Figure 2-2. West (A) and East (B) jetties at Grand Isle, Louisiana. In panel A, a wave can be seen passing through a portion of the structure, in panel B, the entire inshore portion of the jetty is essentially non-existent

Town rock project

In November 1989, the Town of Grand Isle constructed a small breakwater and groin system located approximately in the middle of the island. These structures, known collectively as the Town Rock Project (TRP), consist of four 70-ft-long breakwaters between two 300 ft long groins. A seawall connects the groins along the dune centerline at their landward end (HTMB 1993). Following construction, sand has continually impounded updrift of the TRP, illustrated by Figure 2-3. Within 4 years of construction (1993), the compartment between the groins had become moderately filled. The compartment had completely filled by 1998, at this time accretion had extended the sub-aerial beach out to the westernmost breakwater. The breakwaters appear on the inter-tidal beach face in the November 2002 aerial (not shown), and by July 2003 the entire project is landlocked.

This evidence suggests that the TRP served as a sediment trap and did not allow sand bypassing in meaningful quantities until circa 1998. At the present time, the TRP is completely landlocked and bypassing is evident in the morphology of the downdrift shoreline.

Figure 2-3. Sand impoundment at the Town Rock Project, November 1993, February 1998, and July 2003. Arrows indicate positions of the structures

Detached breakwater fields

Two adjoining detached breakwater fields (DBWs) were constructed along the eastern half of Grand Isle during the 1990's. In 1995, the USACE constructed a field of 23 DBWs, beginning just east of the TRP and extending eastward (Figure 2-4). The project called for fill placement behind the structures; however, the local sponsor attempted to place material via truck hauling but had little success. Placement was subsequently aborted and funds diverted to the construction of structures (per comm. Jay Combe, MVN). The field moves further offshore to the east, with the westernmost and easternmost DBWs lying 450 ft and 1190 ft from shore (a difference of ~740 ft). On average, structure length and gap width are 190 ft and 315 ft, respectively. The present area of concern lies behind the western extent of these DBWs (evaluated from 2002 aerial).

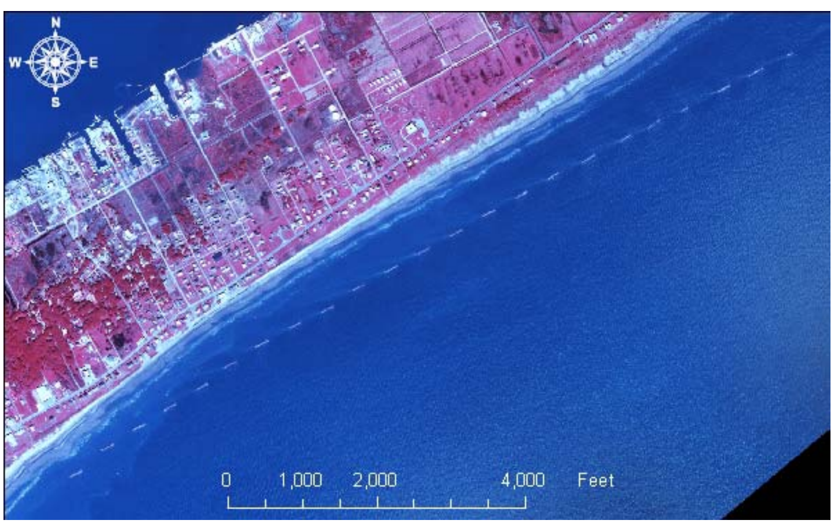

Figure 2-4. Field of 23 DBWs constructed by USACE. The field moves further offshore towards the east

The Louisiana Department of Transportation (LADOT) extended the USACE DBW field circa 1999 with an additional 13 breakwaters. The transition between the LADOT and USACE DBWs occurs as Grand Isle's shoreline sweeps towards the north at the eastern end of the island. On average, the length and gap width of the LADOT breakwaters is 210 ft and 220 ft, respectively. In comparison to the USACE DBWs, the LADOT DBWs are slightly longer and have narrower gaps (~95 ft less). The LADOT DBW's are not as uniform as the USACE field, length, gap width, and orientation of individual DBWs is variable (Figure 2-5). Breakwaters in both fields were designed and positioned to avoid any interference with pipelines.

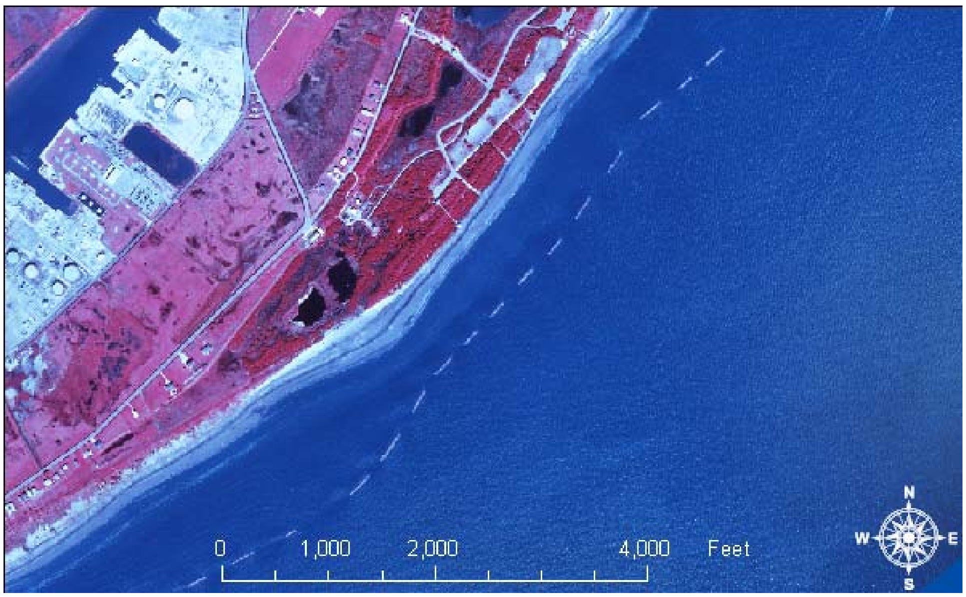

Figure 2-5. Field of 13 LADOT DBWs from state park on eastern end of island. On average, these structures are longer and have wider gaps than USACE DBWs

Performance of the DBWs is difficult to evaluate due to sand impoundment behind the TRP. Beach width progressively widens from the area of concern to the east, with vegetated dune fields fronting development. It is theorized that recent re-establishment of sediment bypassing around the TRP would eventually fill in the area of concern, though this process would likely take some time to occur, leaving the area vulnerably to storms in the meantime.

Inspection of 2002 and 2003 aerial photographs shows an erosion hotspot forming behind the gap between the two DBW fields. This gap is approximately double the average gap in the USACE field and triple that of the LADOT field. It appears that wave refraction and diffraction through this gap (Figure 2-6) are responsible for this localized hotspot. To address the problem, the gap should be closed by either placement of an additional DBW or extension of the existing structures.

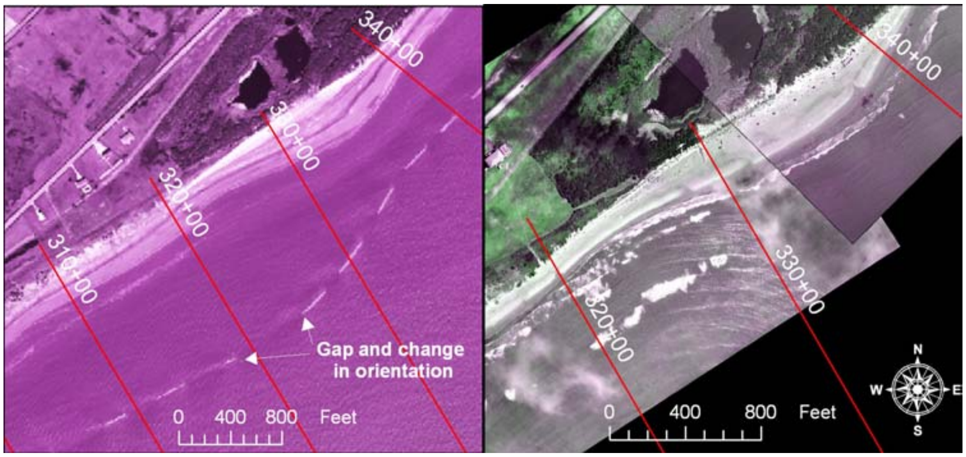

Figure 2-6. Wave refraction/diffraction through gap between USACE and LADOT DBWs has induced a localized erosion hotspot at eastern end of island

Beach Profile Data

Beach profiles are an important source of data for determining the health and ability of a coastline to withstand storm events. These data were essential for designing the beach fill template for Grand Isle. Profile data reduction and quality are described below.

Data assimilation

Ground and bathymetric survey data were provided to CHL for dates between 1992 and 2003. From the initial 830 format, files were reduced into a modified xyz format where original calculated distance, file names and survey dates were retained with each data point. The horizontal datum chosen for the project was Louisiana State Plane South (1702) NAD 1983, vertical datum NAVD 1988. As per MVN's request, distance units were reported in feet. Each dataset was plotted in ArcMap 8.3 to determine spatial distribution. In total, seven unique sets of beach profiles were identified; these dates are reported in Table 2-1.

After identification of each profile dataset, pre-established transect locations (referred to as the "main ranges") were identified and digitized in ArcMap. Data were then compared to transect locations and identified accordingly. Comparable profiles were assembled in Microsoft Excel. Profiles from each date consisted of a mix of both short and long profiles. While not documented, the offshore portion of the profiles appeared to be collected by echo-sounder for all dates. Once all comparative dates were assembled for each transect, data were exported into BMAP for further quality control and analysis.

Table 2-1 Dates of Beach Profile Surveys	
Year	**Month**
2003	May
1996	August
1995	June
1995	March
1994	June
1994	March
1992	September

Long profiles occupied the "main ranges," established on 1000 ft intervals along a baseline with an origin on the west end of the island. Data for these ranges had overlap between the onshore and offshore portions, and matched well except for the 2003 survey.

Quality Control. Each profile underwent three steps of quality control as the data were assembled.

(1) As data were converted into xyz format values were checked for a consistent number of significant digits

(2) Onshore and offshore portions of the profiles were checked for vertical offsets. Horizontal offsets were evaluated in ArcMap.

(3) As data were imported into BMAP, comparative profiles were evaluated for gross changes in morphology indicative of collection errors. If possible, large changes were compared to aerial photography for verification. Profiles were translated vertically or horizontally as needed.

Original files received from MVN reported distances along each transect to the whole foot, while northing and easting were reported to the hundredth of a foot. Attempts to recalculate distances from the baseline to the tenth of a foot (acceptable accuracy) were unsuccessful, as distances did not consistently match those given. For the most part, distances were similar; however, no systematic error could be identified. Either given distances are incorrect, or given locations of ranges along the baseline are incorrect in the original file. Plotting of the data in BMAP showed most data to be comparable, therefore the origins in the files are likely incorrect.

Several problems were observed with the 2003 data. A number of the transects did not correspond with the established ranges, with offsets ranging from 200 to 500 ft. Comparable transects from 2003 were mostly limited to 1992 short ranges, with 14 falling on lines populated by multiple dates (4 or greater). Comparison between these two dates showed that, in places, the 2003 data were either offset vertically, horizontally, or both. Large vertical offsets (3 ft or greater) were apparent along the offshore portions of several transects on the western portion of the island. The inshore portions of the surveys appeared to be much cleaner. While it is possible that the offsets on the 2003 profiles could be explained by gross morphology change in the six-year interval from the last comparable data, morphology change was inconsistent. For these reasons, the

2003 dataset is comparable to those collected on previous dates in a limited capacity, i.e., only at certain locations.

Depth of active transport

The depth of active transport (D_A) is defined as the zone that littoral forces regularly transport sediments on the cross-shore profile (Eq 2-1).

$$D_A = B + D_C \tag{2-1}$$

Where berm elevation (B) is the upper limit and the depth of closure (D_C) is the lower limit (USACE 2002).

Depth of closure was evaluated as the depth at which offshore profiles at a particular station over time converged (or "closed"). This is commonly assumed to be the most landward depth seaward of which there is no significant change in bottom elevation (Kraus et al. 1999). This depth was determined visually from the collection of profiles and verified by calculating standard deviation of elevation values for a minimum of three profiles at each station. The depth of closure was difficult to accurately identify due to the lack of closure at some stations, the short record of reliable profile data (1992-1996), and noise in the offshore portion of the profile. A short-term depth of closure was identified at -9 ft NAVD 88 for this period. Using selected profiles, a longer-term (1992-2003) depth of closure was identified at –12 ft NAVD 88. The -12 ft contour will be used for this study, as it is more representative of the times scales and conditions considered.

Berm crest elevation was determined via visual inspection of the beach profile data. Natural berm elevations at the seaward edge of the berm were observed to range between 3 to 5.5 ft NAVD 88; an elevation of 5 ft was considered to be a representative value for the geomorphic analysis portion of this study. Given a berm height of 5 and depth of closure of 12, the depth of active transport was calculated as 17 ft, (Eq. 2-1). An average berm elevation of 6 ft NAVD 88 was found to be representative of the average elevation of the entire berm, from the seaward edge to the toe of the dune, and this value was adopted for the shoreline change modeling work and for calculations of required fill volume.

Equilibrium beach profile

Among other uses, the equilibrium beach profile (EBP) is employed to estimate the median grain size of a beach (USACE 2002). Knowledge of the native grain diameter of the profile was essential for fill design and determination of the volume of dredged material needed for the project. To evaluate EBP shape, profiles in the critical area were translated to mean sea level (MSL) and horizontally aligned at the zero contour. An average profile was then determined, and the equilibrium profile was solved using least squares estimate for the nearshore slope to a depth of 9 ft (MSL). This depth was chosen to provide the best fit of the EBP to native profile. The solution of the EBP had a good fit to the nearshore slope and is shown in Figure 2-7. A median grain diameter of 0.13 mm was estimated by the EBP for the critical area.

The EBP estimate was compared to samples collected along Grand Isle for the 1979 General Design Memorandum (USACE 1979). Median grain size for samples collected on the beach, 6 and 12 ft contours were averaged to determine the mean cross-shore grain size. Average grain size was 0.128, in agreement with the value determined by the EBP.

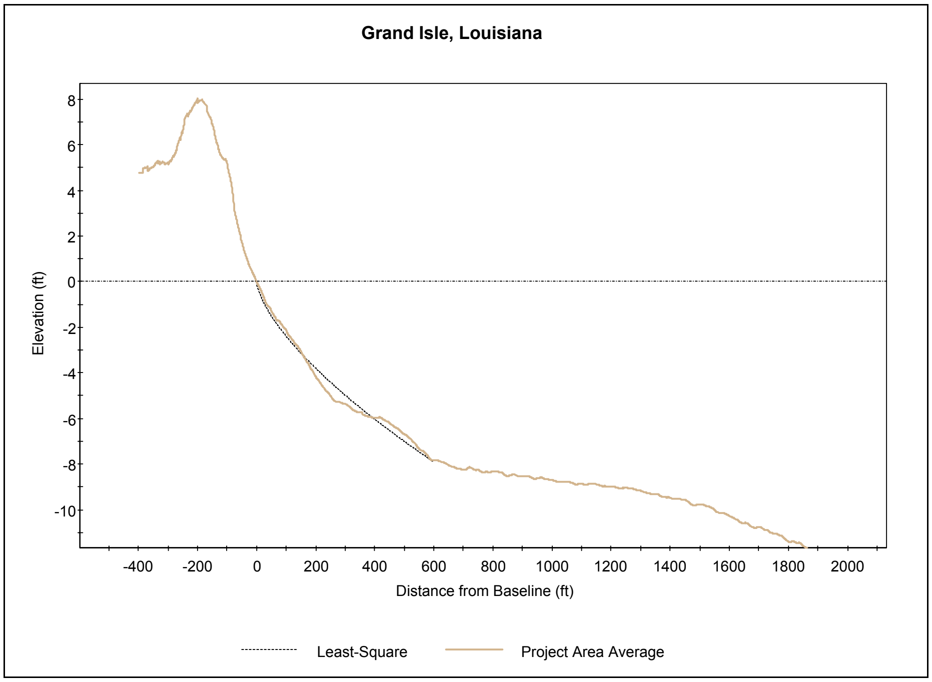

Figure 2-7. Equilibrium beach profile solution for critical area. The EBP indicated a median grain size of 0.13 mm, which agreed with sample data from the 1979 GDM

Shoreline Change Analysis

Shoreline definition

A shoreline is defined as the intersection of land and a given water elevation. Shorelines are typically identified on aerial photographs by the wetted-bound or wet/dry line, which appears as a transition from darker to lighter pixels on the sub-aerial beach. This feature is readily identifiable on most aerial photographs and may be enhanced by various image contrast stretching techniques. When comparing shorelines digitized from aerial photographs to ground surveys, the shoreline definition can affect the accuracy of the study results, as the wetted-bound does not directly correspond to any water level datum (Kraus and Rosati 1997). Due to this, several other identifiers of the shoreline have been proposed, such as the vegetation line and the berm crest.

Between the commonly used shoreline definitions for aerial photographs, the wetted-bound, vegetation line, and berm crest were considered for this study. Varying pixel resolutions between the available photographs ruled out consistent identification of the dune crest or berm crest. The vegetation line was also considered, however, the Town of Grand Isle has actively vegetated the dune and upper berm in addition to repairs and rebuilding of the dunes during the study time frame, making it unreliable for shoreline change. The wetted-bound allows identification of the shoreline across a range of image pixels resolutions and may be identified rapidly. For these reasons, it was chosen as the shoreline definition for this study.

Shoreline extraction from aerial photography

Eight aerial photographs were available in digital format for shoreline extraction, with dates between 1945 and 2003 (Table 2-2). All photographs were obtained from MVN except for the November 1989 and Febuary 1998 images, which were obtained from online sources, Terraserver (http://terraserver-usa.com/) and Louisiana State GIS Atlas, (LOSCO 1999) respectively.

Table 2-2
Aerial Photograph Properties

Year	Date	Pixel Size (ft)	Format	Datum
1945		4.0	mos	State Plane 1702, NAD83, ft
1985	Dec	4.2	tif	UTM, NAD 83, Zone 15N, m
1989	17-Jan	8.9	tif	UTM, NAD 83, Zone 15N, m
1989	9-Nov	3.3	jpg	UTM, NAD 83, Zone 15N, m
1993	20-Nov	4.0	mos	State Plane 1702, NAD83, ft
1998	4-Feb	3.3	sid	UTM, NAD 83, Zone 15N, m
2002	7-Nov	6.6	tif	UTM, NAD 83, Zone 15N, m
2003	11-Jul	1.4	tif	State Plane 1702, NAD83, ft

Georeferencing. Images were previously georeferenced apart from the December 1985 and January 1989 images. These images were rectified with the ArcMap 8.3 software package, using a second order polynomial transformation with 15 to 20 secondary control points. The February 1998 image set was employed as the source imagery for rectification. These images are 3.75 minute Digital Orthophoto Quarter Quadrangles (DOQQ), prepared by the Louisiana Oil Spill Coordinator's Office to conform to National Map Accuracy Standards (LOSCO 1999). Total RMS error for the rectification of the December 1985 and January 1989 images was 18.5 and 12.1 ft, respectively.

Error Statement. Due to the nature of aerial photography, positioning error is inherent when imagery is utilized for shoreline analysis (Anders and Byrnes 1991). Literature values for positioning errors in georeferenced images range from 20 ft (Foster and Savage 1989) to 25 ft (Crowell et all 1991). As images were acquired from various sources, error in the georeferencing of each image is difficult to assess. The aerials were qualitatively assessed to determine error by comparing positions of street intersections, houses, and shore protection structures. Overall, positions were comparable, though some displacements were apparent between the panels of the 1945 mosaic. To estimate error in the project, the December 1985 and January 1989 images will be considered, as georeferencing errors are known.

When comparing shorelines from two images, the error is the sum of the georeferencing in each image, i.e., in the case of the December 1985 and January 1989 images, total error between the two shorelines is 30.6 ft, in addition to error in the digitizing of the shoreline, usually the width of a pixel if the shoreline is identified correctly. This gives a total possible error of 44 ft between the two shorelines. Given a ten-year time interval, if a change rate was calculated at 10 ft/year, 4.4 ft/year of this value is possible error from georeferencing. Values above 6 ft/year in this interval would be considered meaningful. Error decreases with increasing time intervals and/or distance between the two shorelines. Additional errors due to variations of the wetted-bound at the time of the photograph digitizing error are also present. This added uncertainty is estimated at 3 percent. Error estimates for ranges of shoreline change rates are given in Table 2-3.

Table 2-3
Error Estimate

Change Rate (ft/year)	Percent Error
10	44%
15	29%
20	22%
35	13%
50	9%

Shoreline Digitizing. The position of the wetted-bound on each photograph was enhanced using standard deviation and histogram stretching techniques prior to digitizing. The zoom level was set according to each image's resolution to maximize accuracy of the digitized line, i.e., higher resolution images were digitized at a increased zoom level than lower resolution images. Zoom level was held constant as the shoreline was digitized across the image, beginning at the west end of the island and ending at the east end. Point density was varied as necessary to capture alongshore variations in shoreline position. Once the shoreline was complete, the digitized line was reviewed and individual points or sections adjusted as needed. Shorelines were captured in the native coordinate system of the image and converted to the project datum prior to shoreline change rate analysis.

Shoreline extraction from beach profiles

Beach profiles surveys of Grand Isle represent a potential source of shoreline data to supplement those derived from the aerial photographs in support GENESIS modeling efforts. Ground surveys are directly tied to ground control monuments and provide a more reliable source for generating shoreline change rates than aerial photography. To make the most use of the data, it was essential to extract a shoreline that was compatible with those from the aerial photographs. Given that the wetted-bound was defined as the aerial shoreline, mean high water (MHW) was first considered as a compatible datum. The 2003 profile data and aerial photography provided an opportunity to ground-truth compatibility of the MHW contour with the wet/dry line at the site. The 2003 profile data were imported into ArcMap and a 3D TIN model of the data was created. The MHW contour was then extracted from the TIN and edited to remove artifacts from the interpolation process. This process is equivalent to extrapolating the shoreline from the elevation points of the profiles, but without the need to manually identify the elevation on each profile. The MHW contour was overlaid on the 2003 aerial photograph and compared with the location of the wetted-bound line. This comparison showed that the extracted MHW contour was seaward of the wetted-bound for the majority of the island. This was expected, as the MHW elevation was determined from a NOAA tide station located on the bayside of Grand Isle in a sheltered marina. Water levels on the swash-face are expected to be higher due to wave-induced setup in the more energetic Gulf climate. To compensate for this, the profile data were contoured at several different intervals in an effort to obtain a best fit to the aerial shorelines. Examination of the results showed the 3 ft contour (NAVD 88) to have the best overall fit to the wetted-bound. To illustrate the difference, the extracted MHW line and the 3 ft NAVD88 contour were evaluated against the wetted-bound. The distance of each shoreline from a baseline was measured at 100 ft intervals for the length of the island. The difference of the extracted shorelines from the digitized shoreline were then determined at each transect. On average, the MHW line had a difference of 29.6 ft from the wetted-bound line, while the 3 ft NAVD 1988 contour had a difference of 1.8 ft from the wetted-bound line. These results show the importance of ground-truthing results to obtain the most accurate shoreline position. This effort provided shorelines from the profile data that are compatible with each other and with the shorelines derived from the aerial photography.

Other shorelines

In addition to shorelines extracted from available aerial photography and profile data, two additional shorelines (1884 and 1996) were provided at request by University of New Orleans who have conducted numerous studies in the region (Penland, Conner and Beall 2004).

Definition of project area

The condition of the 2003 Grand Isle shoreline was evaluated against the design beach from the General Design Memorandum (GDM) for Grand Isle (USACE 1979). The design beach called for a cross-section with a 10 ft dune crown at 11.5 ft elevation (NGVD 1929), with a dune face at a 1/5 slope to 8.5 ft, and a berm slope of 1/33 to the 0 contour (NGVD 1929). Note that the NGVD 1929 datum 0.12 ft above the NAVD 88 datum. This design template was adjusted to the project datum and applied to the surveyed dune position from May 2003. The berm was extended to the 3 ft NAVD 88 contour (compatible w/ wetted bound) and overlaid on aerial photography taken from July 2003. It should be noted that the GDM design template would represent a best-case scenario for shoreline position, as post-nourishment readjustment of the fill would reduce the design beach width.

Overall, the analysis showed the island to be in good condition compared to GDM design. Four areas of erosion were apparent from the comparison, shown in Figure 2-8. The westernmost hotspot is adjacent to the west jetty, and is likely induced by the poor condition of the structure in addition to wave refraction/diffraction around the tip of the structure. The second area of erosion begins approximately 200 ft west of range 210+00 (208+00) and ends 100 ft east of range 260+00 (261+00) extending for approximately 5600 linear ft of shoreline. This area defines the extent of the problem area for this project. The last two areas are located near the eastern end of the island (Figure 2-8, pane C). The erosion hotspot between ranges 320+00 and 330+00 appears to be induced by wave refraction, diffraction, and sheltering processes as the detached breakwater field changes orientation (discussed earlier, see Figure 2-6 for details). The easternmost hot spot, extending from ranges 340+00 to 370+00 is adjacent to the east jetty.

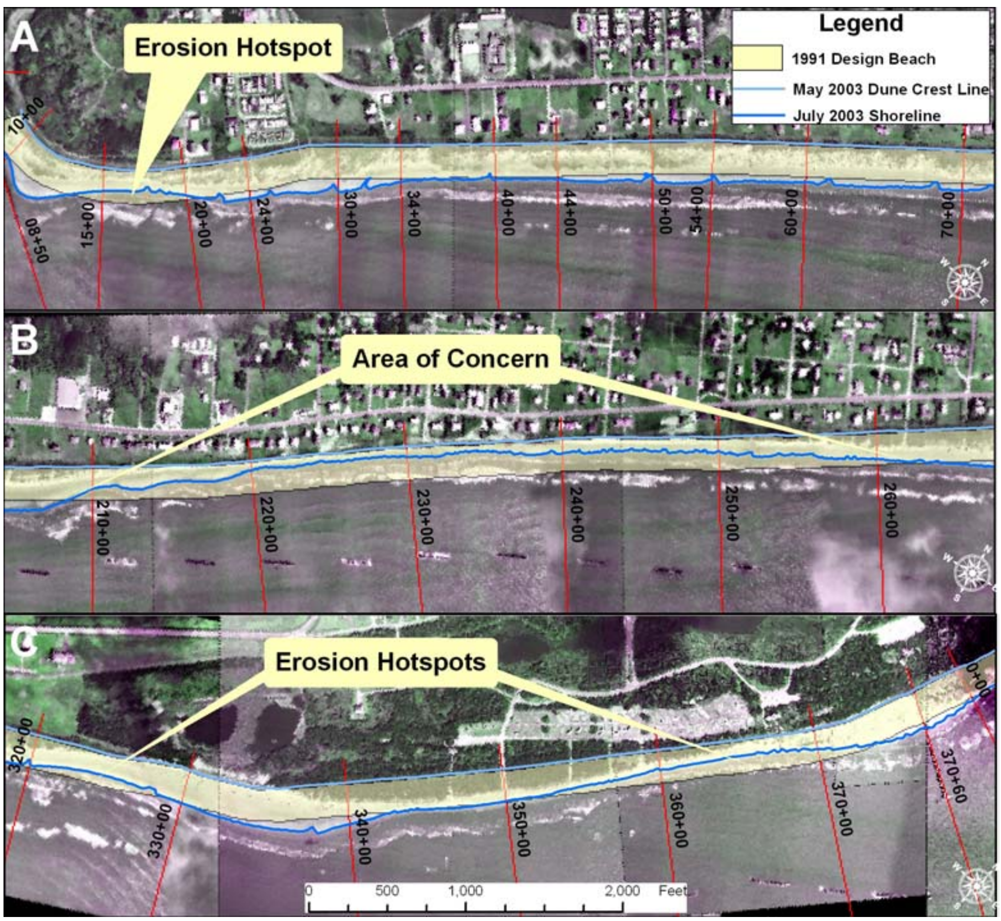

Figure 2-8. Design beach (USACE 1979) compared to July 2003 shoreline condition. Erosion hotspots (Panes A and C) and the area of concern (Pane B) for this study are shown

Shoreline change rate analysis

Shoreline change was evaluated using BeachTools, an ArcView 3.2 extension developed under the Coastal Inlets Research Program (Hoeke et al. 2001). To ensure change rates were compatible with GENESIS shoreline modeling the same baseline was employed for the shoreline analysis. The baseline originates at Grand Isle's east jetty and extends westward along the bay side of the island and extends at an orientation of 147 deg true north (Figure 2-9).

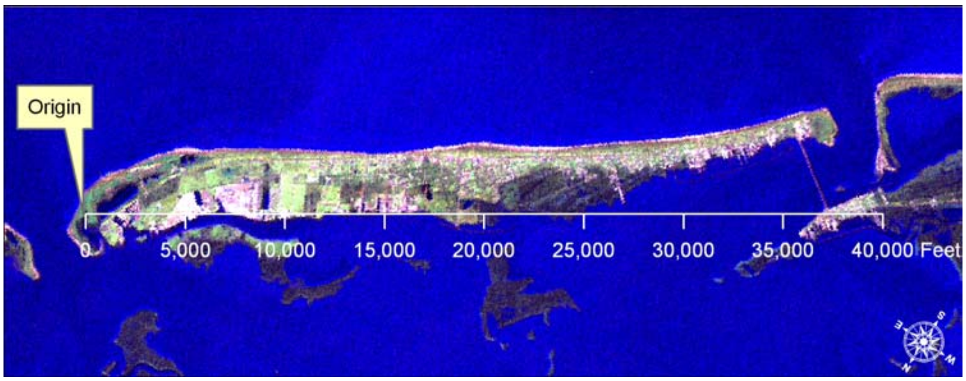

Figure 2-9. Shoreline change rate analysis baseline. The baseline originates at the location of the east jetty and is oriented at 147 deg true north

The BeachTools software was used to generate perpendicular transects from the baseline at an interval of 25 ft, from which the distance to each shoreline was measured. The 25-ft resolution was necessary to support the GENESIS grid, discussed in Chapter 4. Each shoreline was defined relative to the baseline, and results were exported and organized in Microsoft Excel. The data were then imported into Matlab for determination of shoreline change rates and plotting.

For each time interval, shoreline change rates were calculated for the Gulf shoreline by the method of robust linear regression for all shorelines in the interval. This method samples all shorelines in a given time interval, rather than relying on the two end-members, thus minimizing skewing of the rates by a single event. This method has proven effective when data have good temporal distribution (Dolan et al 1991). If a given interval was limited to two shorelines, the end-member rate was determined. A low pass filter was applied to the change rates to remove noise induced by the dense spatial sampling of the shorelines. Average rates of change were then determined for the island reach, as well as the to east and west reaches for the island, with the boundary at the east side of the Town Rock Project. Also, a rate of change was determined in the project area as defined in the previous section.

Results

The temporal distribution of available shorelines allowed both historical and contemporary time periods to be investigated. The available data were divided into three eras: Era I (1884-1945), Era II (1945-1985), and Era III (1985-2003).

The time intervals allow investigation of the shoreline's response to long-term natural processas (Era I), long-term response to construction of jetties (Era II), and short-term response to the town rock project and detached breakwaters (Era III). Shore protection structure positions along the baseline are shown in all shoreline figures, the key to these structures is given in Figure 2-10.

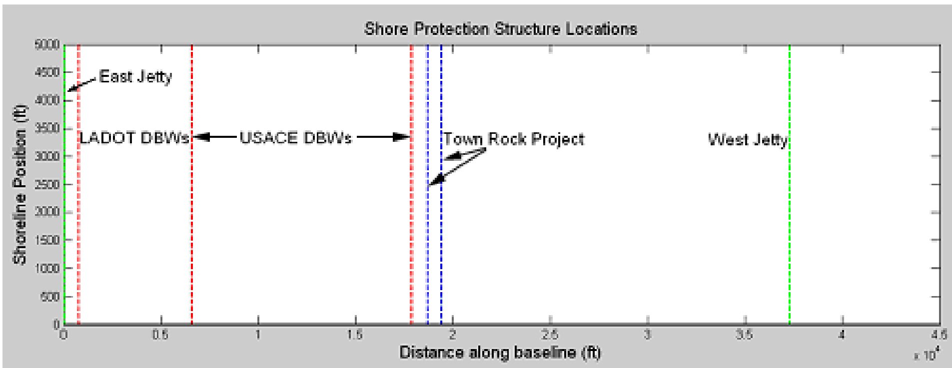

Figure 2-10. Shore protection structure locations along the Grand Isle shoreline change baseline

Era I. 1884-1945. Grand Isle experienced large-scale geomorphic change between 1884 and 1945 as the island narrowed to the west and accreted in the east in response to longshore transport (Figure 2-11). The western half of the island eroded at an average rate of 14 ft/year while the eastern half experienced accretion at an average rate of 7 ft/year. On average, the island eroded at a rate of 4 ft/year. Shoreline orientation of the island rotated approximately 15 to 20 degrees about a midpoint located near the present day western extent of the detached breakwater field. Planform evolution is this period provides strong evidence for a net northeasterly direction of longshore transport.

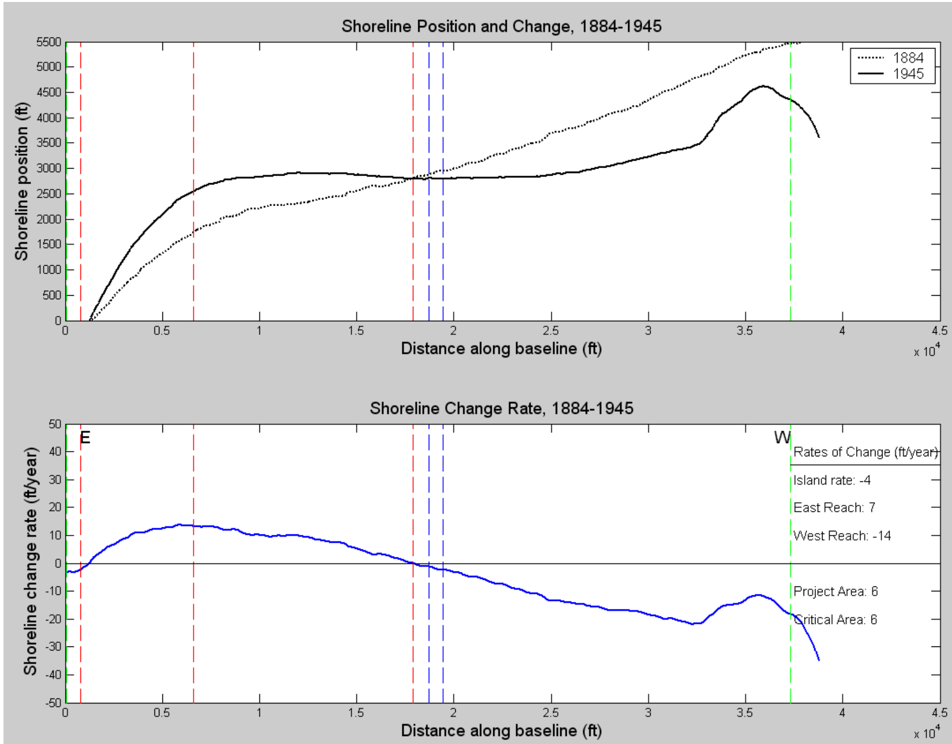

Figure 2-11. Era I. Shoreline position and change rates between 1884 and 1945

Era II. 1945-1985. During this era, Grand Isle accreted at an average rate of 8 ft/year (Figure 2-12). The west end accreted at 3 ft/year, while the east end accreted at a higher rate of 14 ft/year. The continued growth of the east end of Grand Isle is likely a response to construction of the east jetty in 1958. Within 4 years of construction, approximately one million cu yd of sand was reported impounded behind this structure (Theis 1969). This response reaffirms a strong longshore transport to the east.

Between 1983 and 1984, a hurricane protection project was constructed on Grand Isle, placing 2,800,000 cu yd of nourishment fill along the Gulf shoreline (Combe and Soileau 1987). Much of the smaller scale accretion along the island in Era II may be attributed to this. The proximity of the borrow areas to the shoreline resulted in the formation of two salients along the western reach of the island (Combe and Soileau 1987). The footprint of these features on the shoreline planform is apparent in Figure 2-12. Considering that this interval spans 40 years, the short-term response of the shoreline to nearshore dredging is notable.

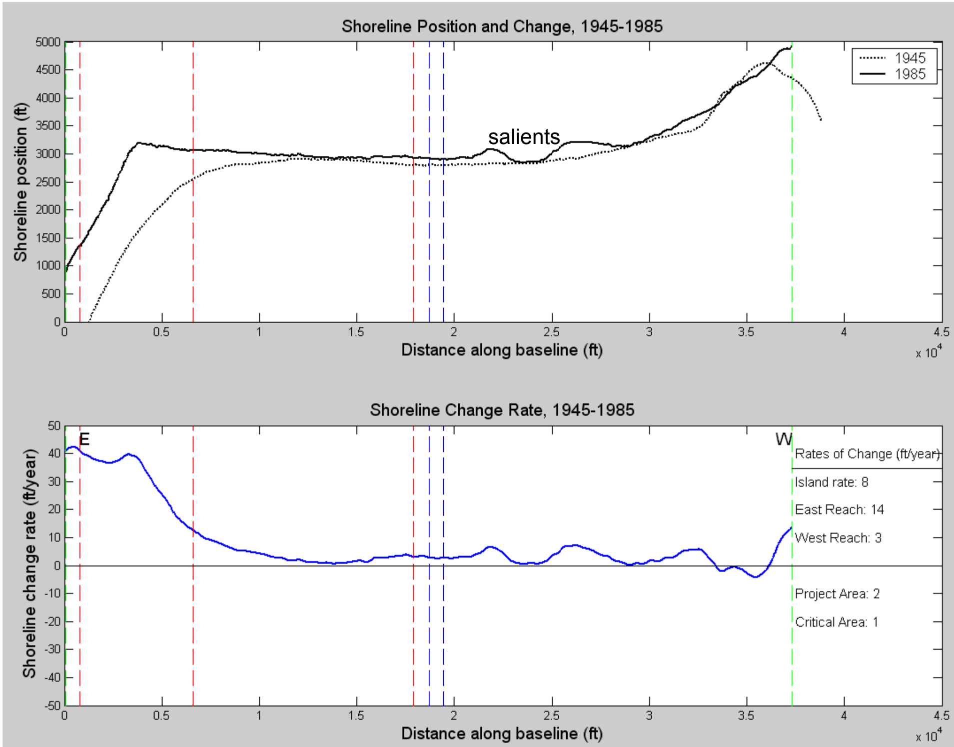

Figure 2-12.Era II. Shoreline position and change rates between 1945 and 1985

Era III. (1985-2003). Era III is characterized by shoreline response to the construction of structures. Two strong shoreline responses are evident: (1) Impoundment of sediment behind the Town Rock Project, and (2) reduction of sediment supply to the east end of the island (Figure 2-13). Shoreline change rates between 1985 and 2003 diverge from previous patterns; with the western reach experiencing accretion (11 ft/year) while the eastern reach erodes (-3 ft/year). The position of the 1985 shoreline (December) represents impacts from Hurricanes Danny (August 1985), Elena (September 1985), and Juan (October 1985). Hurricanes Danny and Elena were not intense, though Juan was reported to erode an estimated 370,000 cu yd from the Grand Isle shoreline (Combe and Soileau 1987). The natural cross-shore beach recovery following these events is likely to be a contributor to the accretion calculated during this interval. The reasoning in using this shoreline in the analysis was to demonstrate the health of the present shoreline planform along the western reach of Grand Isle, and to highlight the reduced variability of the shoreline behind the breakwater field. The best estimate of shoreline change rates in the modern interval is given by the 1993-2003 interval, discussed in the sediment budget section of this chapter.

In the western reach, changes in the planform suggest that the borrow areas in the western reach filled during this interval, as both salients disappeared. This was confirmed from the beach profile data. Most, or all of the material that comprised the salients appears to be present around the Town Rock Project.

Along the east reach of the island, assuming post-storm recovery occurred following the storms of late 1985, the erosion trend in the lee of the DBW project would be stronger than the shoreline change data suggest. The combination of the Town Rock Project, and perhaps to a lesser degree the detached breakwater field has reduced sediment input from the west. Behind the USACE portion of the detached breakwater field, sand moved from the vicinity of the present area of concern downdrift toward the east. At the beginning of the LADOT breakwater field, erosion rates increase.

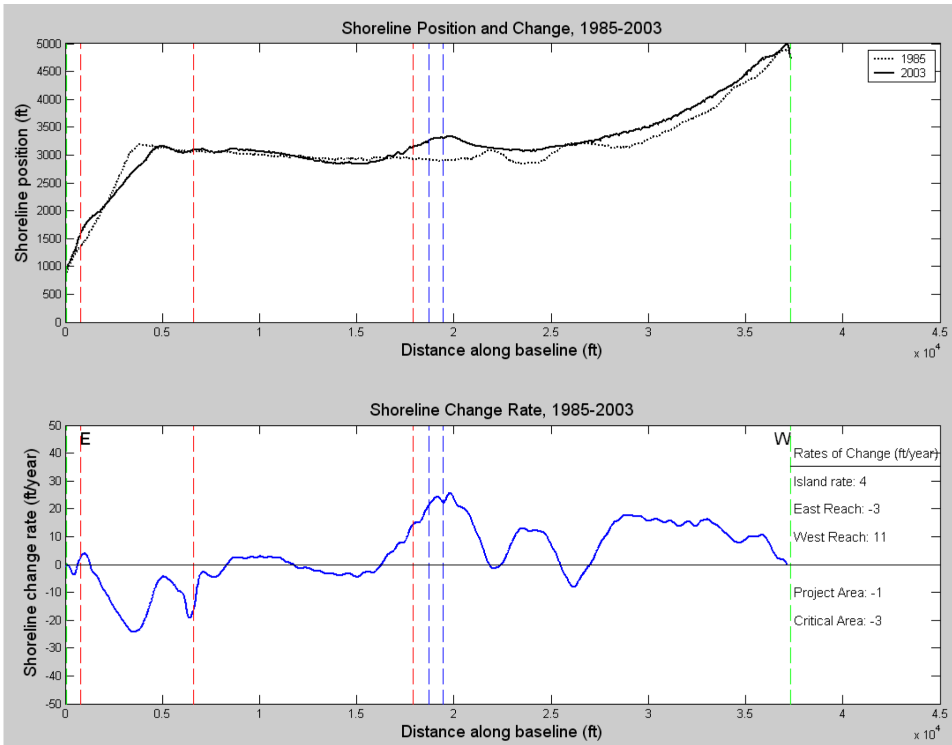

Figure 2-13. Era III. Shoreline position and change rates from 1985-2003

These elevated rates are possibly due to an increase in the longshore transport rate, induced by changes in the shoreline orientation and amplified by tidal currents flowing through Barataria Pass. The shoreline change data suggest that material eroded from the middle of the LADOT breakwaters has been transported to the east, into the vicinity of the terminal groin.

Support of GENESIS modeling

Shoreline change rates were calculated for the following intervals to support calibration of the GENESIS shoreline change model: 1985-1989, 1993-1995, 1992-1995, 1996-2003,and 1995-2003. These intervals combined use of shorelines from both profile data and aerial photographs. The 1992 to 1995 interval was discarded due to the nature of the 1992 profile data (post-hurricane Andrew). Change rates from 1995-2003 were similar to the 1996-2003 interval, giving confidence to the use of the profile shorelines in the analysis. The 1996-2003 interval is shown below (Figure 2-14).

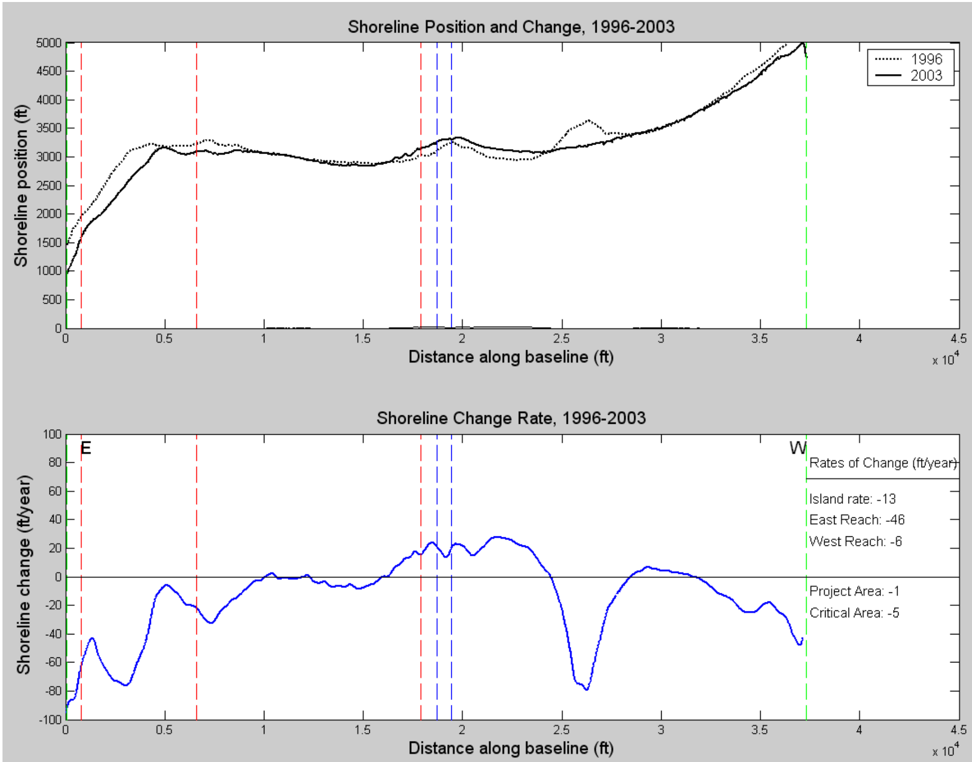

Figure 2-14. Shoreline change rate for 1996-2003

Sediment Budget

A sediment budget was determined for the Gulf shoreline of Grand Isle between 1993 and 2003. Evolution of the Grand Isle shoreline planform in this interval is represented by shorelines from 1993, 1996, 1998, 2002 and 2003 (Figure 2-15). All shorelines were derived from aerial photography as previously discussed. This time interval was chosen for several reasons. The interval begins after the 1991 fill placement, which skewed shoreline analysis toward accretion for the 1989-1993 interval. The 1992 shoreline could not be used, as it was collected immediately after Hurricane Andrew. Comparison of shoreline change rates from 1992-1995, and 1993-1995 showed that rates determined from the 1992 shoreline were padded toward accretion, likely due to post-storm recovery processes, as fill was not placed during this time.

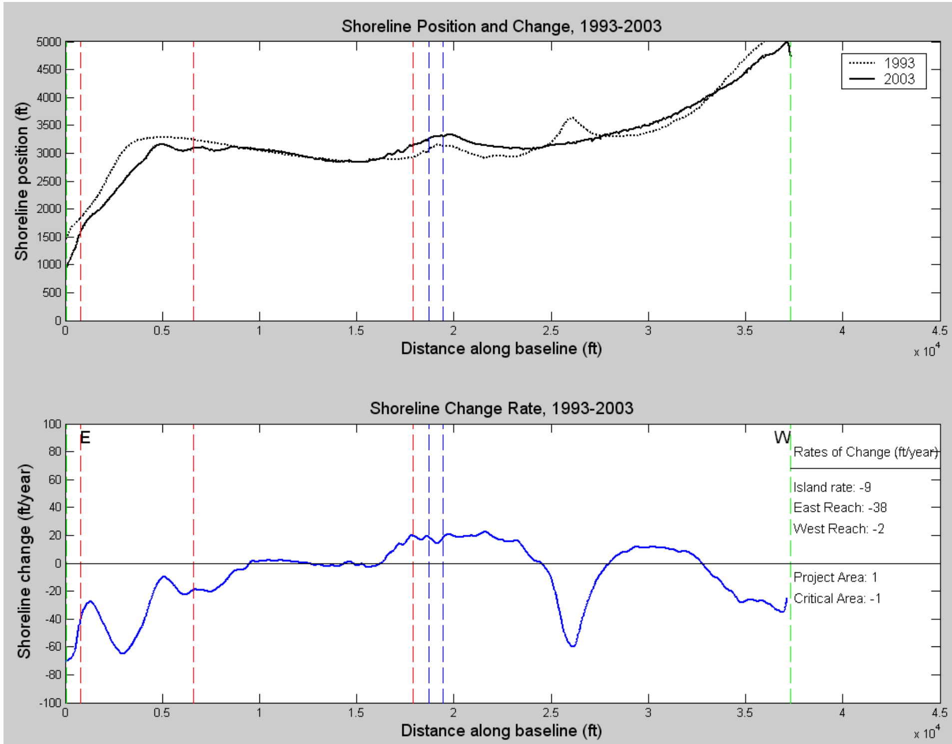

Figure 2-15. Shoreline position and change between 1993 and 2003. Notable changes include the disappearance of the large salient in the west reach and impoundment around the rock project. Accelerated erosion rates are apparent at both ends due to the present condition of the jetties.

The 1993 to 2003 interval provides a sufficient amount of time to evaluate the impact of both the Town Rock Project and the detached breakwater field while minimizing noise. Shoreline change rates were determined as described in the previous section. The Gulf shoreline of Grand Isle was divided into nine littoral cells. Cell boundaries (Figure 2-16) were defined where the shoreline change rate crossed zero (Cells 1, 2, 3, 6, 7), at the TRP to evaluate impoundment and bypassing rates (4, 5), or where large gradients in the shoreline change rate appeared (8, 9). The average shoreline change rate was determined in each cell. Volumetric change was calculated from shoreline change rates using the equilibrium beach profile model (USACE 2002):

$$\Delta V = \Delta y \bullet \Delta x \bullet d \qquad (2\text{-}2)$$

where

Δy = shoreline change rate.

Δy = alongshore cell length.

d = depth of active transport (17 ft)

Volumetric fluxes in and out each cell were then determined using the sediment continuity equation (Komar 1998) solved for the flux out of a littoral cell:

$$Q_{out} = Q_{in} - \left(\frac{\Delta V}{\Delta t} \right) \tag{2-3}$$

where

ΔV = change in volume for a littoral cell.

Q_{in} = longshore transport rate into the cell

Q_{out} = longshore transport rate out of the cell

Δt = the time interval

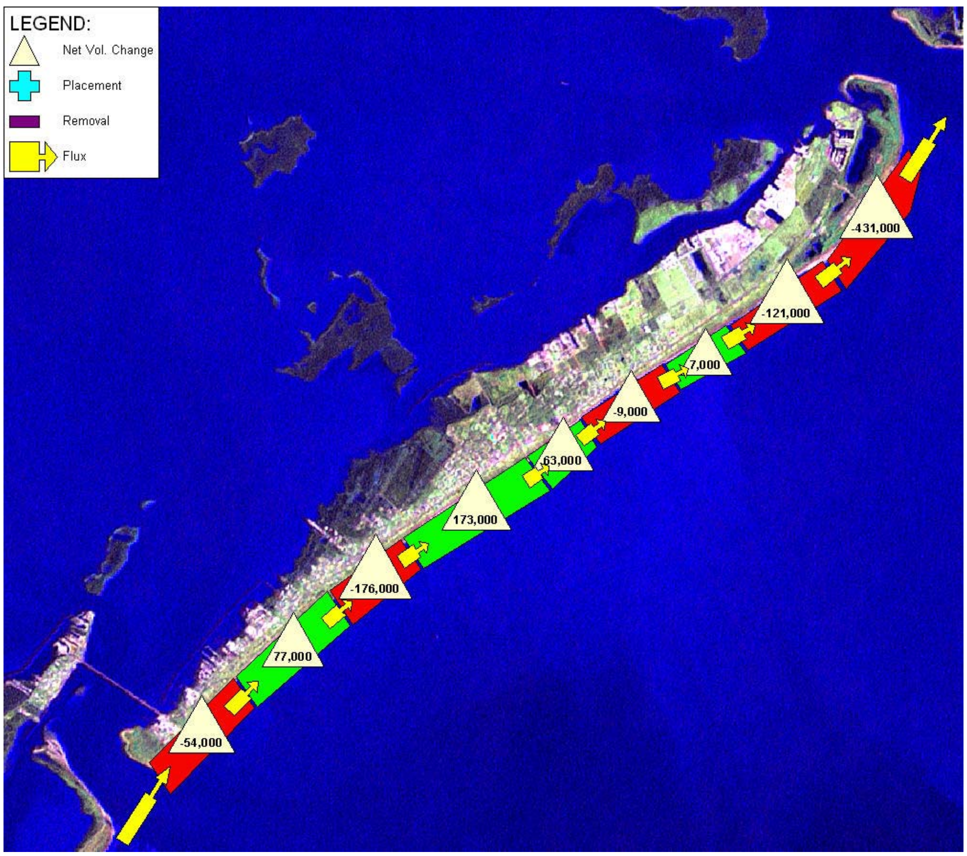

Figure 2-16. Grand Isle sediment budget cells superimposed on 2002 satellite imagery. Cell 6 represents the project area. Red cells indicate volume loss (sediment source), green cells indicate volume gain (sediment sink)

Results

Table 2-4 shows alongshore distance, shoreline change rate and volume change for each cell. Volume change values should be considered approximations.

Table 2-4
Calculated Volume Change

Cell #	Distance (ft)	Shoreline Change Rate (ft/year)	Volume Change (cu yd/year)
1	4525	-6.3	-54,000
2	4775	8.5	77,000
3	3500	-26.6	-176,000
4	5750	15.9	173,000
5	2500	13.3	63,000
6	3775	-1.2	-9,000
7	3000	1.3	7,000
8	4375	-14.6	-121,000
9	5100	-44.7	-431,000

Budget assumptions

The following assumptions were made during formulation of the sediment budget:

(1) Net longshore transport is from the west to the east, and is supported by the long-term evolution of the shoreline planform.

(2) As sediment has just recently begun bypassing the TRP, it may be expected that sand volume lost from the erosion of the salients is conserved in the downdrift compartment, impounded against the TRP.

(3) Volume gains not accounted for by losses updrift should be considered a result of sediment fluxes into the system across Caminada Pass at the west boundary.

(4) Given (1) and (3), net gains of sediment in the budget may be solved for by increasing the flux in at the west boundary.

These assumptions were applied to the calculated volume changes in Table 2-4, results are given in Table 2-5. Error was estimated at 25 percent. Values should be considered approximations.

Results show that net volume change within these (cells 1-4 and 6-7) is minimal compared to the gross change, and values are within the possible error in the calculation. Therefore, sediment in these compartments is conserved within the error. Given this, the volume change in cells 2-5 is considered the rate of sediment bypassing at the TRP, approximately of 63,000 cu yd ± 25 percent. This bypassing rate may be used to calculate sediment flux into the west boundary of Grand Isle (cell 1). The magnitude of the Q_{in} at the west boundary should satisfy the following condition for cell 5: sediment supply is adequate to support the present rate of accretion while the flux out (into the project area) is minimal. An optimal value of 83,000 cu yd/year for Q_{in} at cell 1 was determined to meet this condition. All calculated fluxes are given in Table 2-6.

Table 2-5 Volume Conservation in Combined Cells				
Cell #	**Vol Change (cu yd/year)**	**Net Volume Change (cu yd/year)**	**Gross Volume Change (cu yd/year)**	**25% of Gross (Error)**
1	-54,000	20,000	479,000	120,000
2	77,000			
3	-176,000			
4	173,000			
5	63,000	63,000	63,000	16,000
6	-9,000	-1,000	16,000	4,000
7	7,000			
8	-121,000	-551,000	551,000	138,000
9	-431,000			

Table 2-6 Sediment Fluxes*		
Cell #	**Volume Change (cu yd/year)**	**Q_{out} (cu yd/year)**
1	-54,000	137,000
2	77,000	60,000
3	-176,000	236,000
4	173,000	63,000
5	63,000	500
6	-9,000	9,000
7	7,000	2,000
8	-121,000	122,000
9	-431,000	553,000
*assuming 83,000 cu yd/year flux into the system		

Updrift sediment sources

Erosion of the Caminada-Moreau headland is the updrift sediment source for Grand Isle (Figure 2-17). Penland, Conner and Beall (2004) report a reduction in the erosion rate of the Caminada-Moreau headland for the period 1988-2002 to an average of –9 ft/year, compared to historical rates ranging from -31 to -52 ft/year (1884-1988). The reduction in shoreline retreat rates for the Caminada-Moreau headland has direct implications in the supply of material into the sediment budget for Grand Isle. The equilibrium profile model (Equation 2-2) was applied to estimate the potential loss of volume input into the system.

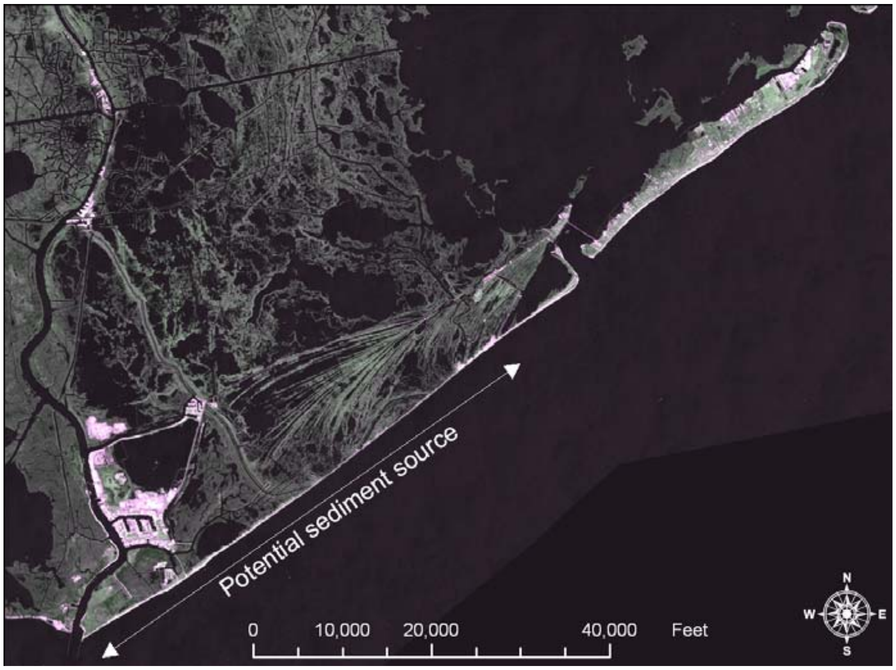

Figure 2-17. Caminada-Morea Headland. Potential sources of littoral material are derived from erosion along the headland

Assumptions. Alongshore distance of the section of the Caminada-Moreau headland contributing to sediment input is approximately 57,400 ft.

(1) The active depth is 17 ft and is relatively constant along the headland. This value was determined for the Grand Isle and may not properly represent processes along Caminada-Moreau Headland.

(2) One third of the volume is beach suitable sediment (List et al. 1991).

(3) Shoreline rate of change for 1884 to 1988 is approximately –42 ft/year, for 1988 to 2002 the rate of change is –9 ft/year.

The active depth used in this calculation was determined for Grand Isle and may not be representative of processes along the Caminada-Moreau Headland. Application of assumptions 1-3 to the historical period (1884-1988) give a potential sediment input of 4.5 million cu yd/year calculations for the modern period (1988-2002) resulted in a rate of 900,000 cu yd/year. This represents a potential deficit of 3.6 million cu yd/year in the regional sediment budget and may have implications into the future stability of Grand Isle.

Borrow Area Evaluation

Sediment grain size data from two borrow areas on the Barataria Pass ebb shoal were evaluated to determine the availability of suitable fill material. Overfill ratios were calculated for borrow areas containing material finer than the native beach. Potential borrow sites for future projects are also discussed.

Given the proximity to the project area, prior use, and availability of detailed data on underlying sediments, two borrow areas on the Barataria ebb shoal were evaluated for potential use (Figure 2-18). Sediment size distributions were examined for each core sample. Potential sand volume was evaluated for core sections containing sand percentages above 80 percent with a median grain size (d_{50}) greater than 0.10 mm. These guidelines were chosen to provide a fill material with similar characteristics to native sediments, in addition to minimizing overfill volume. Total volume was estimated from the bed surface to the depth where suitable sediments were located via a triangulated irregular network (TIN). Volume estimates were limited to the extent of the borrow area (East Barataria) or to the extent of the core data (West Barataria).

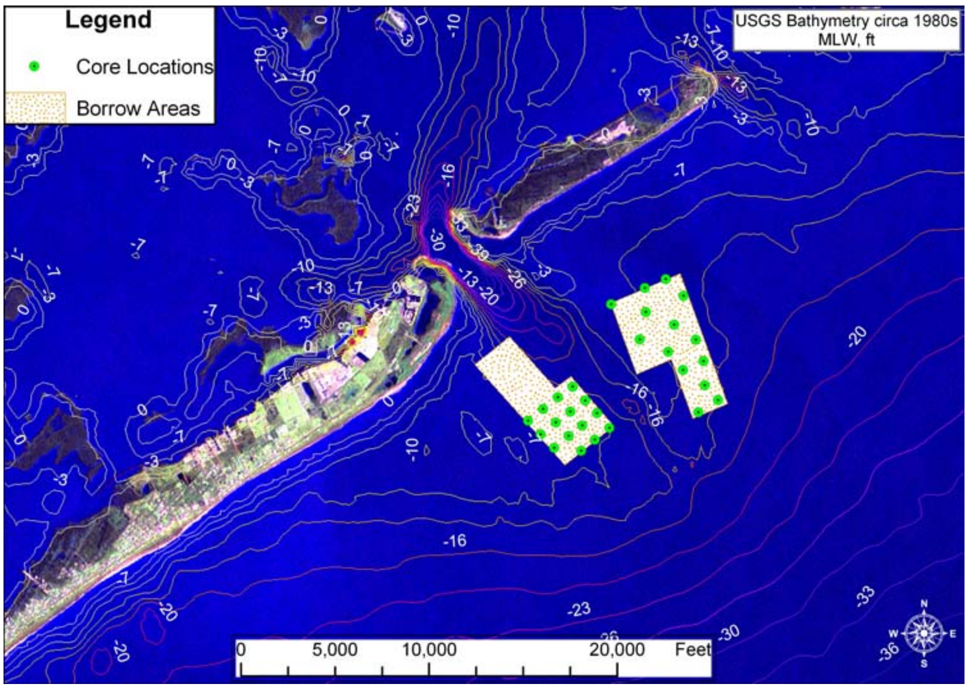

Figure 2-18. Borrow area and core locations on the Barataria Pass ebb shoal

Results from the analysis suggest that the West Barataria borrow area is more suitable for dredging. Core data indicated suitable sediments to an average depth of approximately 5 ft below the bed through the area, with little variability in thickness (Figure 2-18). Volume in this area (0.4 square mile) was estimated at 2 million cu yd. In contrast, the thickness of suitable material in the eastern area was variable, with the thickest portion constrained to a limited area. Volume in the eastern borrow area was similar to the west borrow site, except distributed over approximately twice the area (0.8 sq mi). Grain size was finer than native beach sediment in both areas, with an average median grain diameter of 0.113 and 0.116 mm for West Barataria and East Barataria, respectively. Less variability in grain size was observed in the western area.

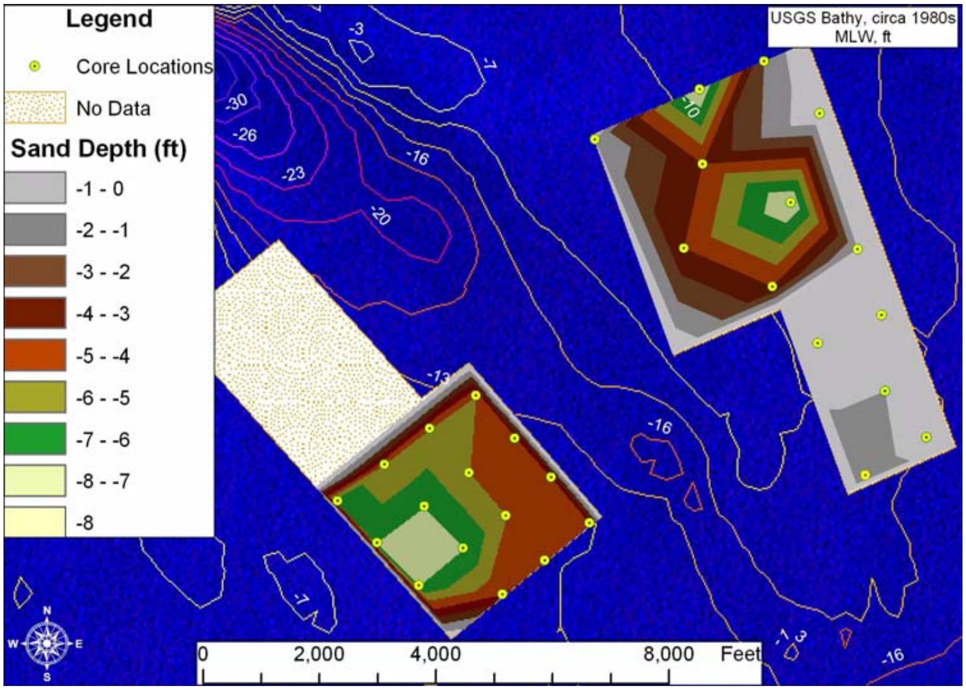

Figure 2-19. Thickness of suitable fill located in the east and west borrow areas on Barataria ebb shoal

Overfill ratios

Sediment grain size for both East and West Barataria Borrow areas was found to be finer than native beach sediments. Due to this condition, it was necessary to determine the additional material that will be required to fulfill the design template (overfill). The overfill ratio describes the volume of borrow material that is equivalent to one unit of native material. Here, the overfill ratio is evaluated by two methods: the equilibrium profile method and the overfill factor (R_A). Total volume requirements to meet the design fill template will be discussed in Chapter 4.

Equilibrium profile method

The equilibrium profile method is the standard engineering practice for evaluating the overfill ratio (USACE 2002). This method estimates the overfill ratio by comparing the volume of the EBP associated with the native material to the volume associated with the EBP of the fill material. Previously, the EBP solution for the native beach estimated native grain size at 0.13 mm. This value will be used for comparison of fill materials. Assuming fill material has a native grain size of 0.13 mm, the required volume of the profile per unit length of shoreline is estimated by Eq 2-4:

$$V = (B + H) \bullet Y \tag{2-4}$$

Where V is volume of sediment per unit length of shoreline, B is the Berm elevation, H is the depth of closure, and Y is the increase in beach width by the fill design template. Values of *B*, *H*, and *Y* were 6, 12, and 131 ft (an average added width developed from the shoreline change modeling), respectively.

The volume required to meet the design template by a finer borrow grain diameter was determined by equation 2-5 (non-intersecting profile):

$$V = YB + \frac{3}{5}H^{5/2}\left\{\left[\frac{Y}{H^{3/2}} + \left(\frac{1}{A_F}\right)^{3/2}\right]^{5/3} A_N - \left(\frac{1}{A_F}\right)^{3/2}\right\} \quad (2\text{-}5)$$

Where A_F and A_N are the A-parameter values for the fill and nourishment mean grain sizes, respectively. Values of the A-parameter were converted from borrow area mean grain size using methods outlined in the Coastal Engineering Technical Note CETN II-32. Average mean grain sizes for the east and west borrow areas were similar, 0.116 and 0.113 mm, respectively. The required volume of borrow material is then divided by the volume computed assuming the native material was used, to determine the overfill ratio. Results of this analysis are given in Table 2-7.

Table 2-7
EBP Overfill Ratios

Borrow Area	A (m**1/3)	Volume (cu yd/ft)	Overfill Ratio
Native (EBP)	0.0789	87	
East Barataria	0.0747	109	1.26
West Barataria	0.0737	115	1.32

Overfill factor

While the equilibrium profile method is standard practice, the overfill factor (R_A) considers both mean sediment diameter and sorting values of the native beach and borrow sediments in phi (φ) units ($-\log_2(\text{mm})$). This estimate is often employed as a check for the EBP method, as it provides a more conservative estimate of the overfill ratio due to sorting. R_A was determined for native beach using both mean grain size as determined by the EBP method (0.13 mm).

Grain size distributions of core samples from both borrow sites were evaluated by equations 2-6 and 2-7 to determine the mean sediment diameter (M_φ) and the standard deviation or sorting of the material (σ_φ). M_φ and σ_φ were determined for suitable sections of each core, then averaged for the core. An average value was then determined for each potential borrow area. These values were input into the Automated Coastal Engineering System (ACES) version 2.01G (USACE 1992) to determine R_A. Values for R_A are listed in Table 2-8.

$$M_\varphi = \frac{(\varphi_{16} + \varphi_{50} + \varphi_{84})}{3} \quad (2\text{-}6)$$

$$\sigma_{\varphi} = \left[\frac{(\varphi_{84} - \varphi_{16})}{4} + \frac{(\varphi_{95} - \varphi_{5})}{6} \right] \tag{2-7}$$

Table 2-8
Overfill Factor

	M_{φ}	σ_{φ}	R_A
Native	2.54	-0.419	
East Barataria Shoal	3.11	-0.924	1.26
West Barataria Shoal	3.16	-0.15	1.29

Overfill ratios calculated by the overfill factor (R_A) were similar to those determined through the EBP method. An overfill ratio of 1.3 is recommended for estimating sand volume borrowed from either borrow site. We recommend using the West Barataria Shoal borrow site in light of the proximity of the area to the project on Grand Isle, the increased homogeneity of the borrow material, and the possibility that sand from the East borrow area might be needed for placement beaches in the downdrift direction, (i.e., to the east), such as Grand Terre.

Sand Management Recommendations

Structures

Recent shoreline change rates (erosion) were observed to increase near both jetties, especially in the vicinity of the east jetty. The west jetty is in better condition, with the seaward portion beginning to loosen and fall apart. The east jetty is in an exteme state of disrepair. The shoreline west of the east jetty has experienced increasing erosion rates in the last ten years. Reduction of beach width in the vicinity of the structure has resulted in exposure of older sections that are in an extreme state of disrepair. Longshore transport along this reach of coast is high, due to the change in shoreline orientation and augmentation by tidal currents through Barataria Pass. It is likely that some volume of material is passing through this structure and being lost to the Barataria Pass shoal complex, especially during storm events. Impoundment of material updrift of the east jetty has historically occurred in large quantities (Combe and Soileau 1987). Sediment bypassing around the TRP was recently reestablished and should begin to provide some quantities of material into the eastern reach of the island. Insuring that the eastern terminal groin can retain material should be high priority, as impounded material was historically used for back-passing to nourish updrift beaches. Repair of this structure would help retain this material in the future and provide an inexpensive source of beach quality sediments.

Other potential borrow sources

Borrow areas located on the Barataria Pass ebb shoal contain adequate amounts of sediment for the present project. However, in the future, additional resources may be required to nourish the Grand Isle coast. A regional sand resources study was recently undertaken by the U.S. Geological Survey and it identified an additional potential borrow area on the Caminada Pass ebb shoal (Kindinger et al 2001). Core logs were available from the east and west flanks of the shoal. Sample BSS00_209 showed suitable material to a bed depth of 5 ft (extent of recovered material). Sand percentages were over 90 percent at this depth, with a median grain size of 0.09 mm (on the fine side of what would be most desirable for placement on Grand Isle). Sample BSS00_208 on the west flank of the shoal was less promising, with suitable sediment to a depth of 3 ft. The Caminada ebb shoal is the likely pathway for sand leaving the Caminada-Moreau headland and entering the Grand Isle littoral zone. A more comprehensive survey of these resources should be conducted in the future to accurately assess the volume and characteristics of suitable material in this area.

In addition to the Caminada ebb shoal, the Caminada spit should also be considered a possible source for future sediment needs. The recurved portion of the spit should contain quantities of suitable sediments for nourishment and should be assessed for potential mining.

Summary

Shoreline change rates, the sediment budget, structural conditions and borrow areas were investigated for Grand Isle, Louisiana. Historical shoreline evolution demonstrated that net longshore transport along Grand Isle was in the northeasterly direction. Recently constructed shore protection structures have had an influence on the evolution of the shoreline planform. The Town Rock Project, constructed in 1989, has impounded sediment along the western reach of the island at the expense of the eastern reach of the island. This has decreased shoreline erosion rates relative to the long-term trend in updrift areas, provided stability to the western reach of the island, but induced the opposite effect on the east side of the island. However, evidence from the sediment budget indicates that material is now bypassing the TRP at an approximate rate of 63,000 cu yd/year. The flux into the Grand Isle littoral cell across Caminada Pass was estimated at 83,000 cu yd/year.

Along the eastern reach of the island, elevated shoreline erosion rates appear to be a result of reduction of sediment input due to the TRP and reduction in alongshore sand transport associated with the DBWs. Sediment pathways to the east reach of the island have recently been reestablished. Restoration of sediment input (past the TRP) into the system and rehabilitation of the east jetty should begin to mitigate erosion in these areas. However, if beach nourishment is not undertaken, the critical erosion zone is expected to simply migrate to the east over the short term. Since the breakwater field reduces the longshore transport potential updrift of the severe erosion areas, it may take some time before this portion of the island re-equilibrates. The increasing tidal prism and transporting power of tidal currents at Barataria Pass may also limit recovery along the shoreline east of the east jetty.

Borrow areas in the vicinity of Grand Isle were evaluated for volume of suitable beach-type sediments. The West Barataria borrow area was found to contain approximately 2 million cu yd of suitable material. Sediments in this borrow area are better sorted than those in the East Barataria borrow site. Limited core data was available to investigate potential borrow areas on the Caminada Pass ebb shoal. The eastern flank of Caminada ebb shoal was promising, and a more comprehensive assessment of sediment resources should be conducted.

A detailed history of Grand Isle's shore protection is summarized in Table 2-9.

Table 2-9
Shore Protection History

Date	Project Details
1904	Bayou Lafourche closed for flood control purposes. Further reduction of sediment input into region.
1951 (1)	14 timber groins constructed to protect highway. Four 500 ft groins near the west end, approx. 3,000-5,700 ft from end of island. 10 groins toward center of island, 11,500-18,500 ft from east end of island. Spacing: western four 500 ft, eastern six 250 ft.
1954 (2)	Beach fill project, total 1,150,000 cu yd.
	800,000 cu yd pumped to fill groin compartments in center of island.
	350,000 cu yd pumped to fill western groin compartments.
	No design given, referred to as "stockpiles".
	Borrow areas immediately north of island.
	Material had slightly smaller median diameter, deemed compatible.
1956 (1)	Humble Oil and Refining Company: timber groin 6,800 ft from east end. Material placed on west side of groin from offshore source.
1956 (2)	Hurricane Flossy.
1957	Sand dune restoration after Hurricane Flossy. 140,000 cu yd for 4 1/2 mi of beach.
1958	1200 ft rock jetty constructed at east end. Within 4 years ~1 million cu yd trapped, approx. 30 acres east of jetty completely lost.
1961	Hurricane Carla. Significant damage.
1961-1962	350,000 cu yd placed within ten groins in center of island in response to damage from Hurricane Carla.
1964	East jetty extended 1400 ft to total length 2600 ft.
1965	Hurrican Betsy, storm surge of 8.8 ft MSL, GI inundated. Damage to structures in "millions of dollars".
1966	550,000 cu yd borrowed from accretion fillet to restore dunes. Eastern Jetty repaired 400 ft inshore section lost during Betsy, when constructed, the contractor had run out of timbers and used shell material for underlayment in this section.

1967	1,000 ft concrete revetment constructed on north side of east end of island to protect LORAN station. 900 ft failed by end of year due to overtopping, uplift and loss of foundation.
1969	Performance of East Jetty. Trapped a large amount of material, excellent source for replenishment. Establishment of State Park resulted in limited use as borrow area. Noted that groin field has caused problems in past. Groins now deteriorated at seaward ends, continued deterioration desired to "equalize littoral drift".
1970	USACE installed a 1,400 ft rubble mound revetment around east end of island tying into the eastern jetty.
1971-1972	West jetty constructed in response to severe erosion along west reach of island. 2,600 ft long, elevation 4 ft MSL, crown 6 ft wide.
1974	Hurricane Carmen. Destroyed dune.
1975-1976	6 mile project area, fill placed. Longard Tubes used for fill retention and groins at 400 ft spacing. Dune design to 7' MSL 1:10 to toe @ 4' MSL, 1:25 to shoreline. "Two critically eroding areas". 1 at western end near Caminda Pass. Other? Not identified.
1983-1984	Hurricane Protection Project, total 2,800,000 cu yd. Dune design to 11.5 ft NGVD, 1:5 slope to 8.5 toe, 1:33 berm to 0 NGVD. Offshore borrow area, approx 3,000 ft offshore, 9,000x1,000 extent. Excavated to depth of 20 ft at ends, 10 ft at center. Dumbell shape, centriods of ends approx 4,500 ft apart, considerably deeper than center. 2 Cuspate bars (salients) formed by Feb 1985 (project completed by Sept 1984). Borrow areas identified through aerial survey as diffracting waves.
1985, 14-15 August	Hurricane Danny (not intense). est. 70,000 cu yd loss.
1985, 8/30-9/2	Hurricane Elena (erratic, wanders). est. 40,000 cu yd loss, erosion to toe of dune adjacent to salients.
1985, 10/28-31	Hurricane Juan (erratic, wanders). est. 370,000 cu yd loss. 6000 ft of dune leveled, over 14,000 ft of dune scarped.
1985, 11/16-11/22	Hurricane Kate.
1986, Dec	Borrow pits have shown "trend" of infilling.
1987, Oct-1988 Feb	Jetty extensions, East jetty extended 200 ft, West jetty extended 500 ft. Cuspate bar removed from east end state park and used to restore beach and dune inside of the state park, placement in the vicinity of the fishing pier.
1988, 9/1-9/19	Hurricanes Florence & Gilbert.
1989, 7/30-8/3	Hurricane Chantal.
1989, Oct	180 ft long, 3 ft high seawall constructed in front of motel complex.

1989, Nov	Town Rock Project. Detached breakwater and groin project. Two 300 ft long groins, 700 ft long seawall, four 70 ft long breakwaters w/70 ft gaps. Breakwaters ~ 350 ft offshore at elevation 6-8 ft MSL.
1990-1991	Beach renourished, sand dunes rebuilt w/ clay core, sand-tube breakwaters installed. 600,000 cu yd placed. Beach design has dune @ 11.5-13.5 ft elevations (NGVD) w/ 10 ft crown, toe @ 7.5-8.5 ft. Borrow area 2,700 ft offshore @ eastern end of the island. 1991 DNR lists placement on all of island, no real specifics to sand placement.
1992, 8/24-28	Hurricane Andrew. High stage of 3.57 ft NGVD. Estimated erosion 250,000 cu yd. Dune suffered approx 1 mile damage w/ 1,000 ft heavy damage.
Dec 1994-May 1995	23 Detached breakwaters built by USACE. Fill: Local sponser attempted to fill by truck haul, w/ little success, aborted and money spent on rock-(per comm. J. Combe, NO District).
1995, 9/26-10/5	Hurricane Opal.
1997, 7/16-20	Hurricane Danny.
1998, 8/30-10/1	Hurricanes Earl, Frances, Hermine, Georges.
1999?	13 Detached breakwaters built by LADOT.
2002, 8/4-9	Tropical Storm Bertha .
	high stage of 2.16 ft NGVD (average annual 3ft NGVD), not significant.
2002, 9/12-14	Tropical Storm Hanna. high stage of 2.76 ft NGVD, Not significant.
2002, 9/14-24	Tropical Storm Isidore. high stage of 4.86 ft NGVD.
2002, 9/21-10/4	Hurricane Lili. high stage of 4.5 ft NGVD, Category 4 24 b/f landfall. Damage Summary, Isidore & Lili. Combined had an impact that exceeded design storm. beach between sta 208+00 to 255+00 eroded an average to half the width, dune partially destroyed or breached in project area, remainder of island dune in excellent shape. Clay dike constructed between storms, clay wrapped in geotextile 4 Storm summary: ~9 weeks of near tropical storm conditions, incl 2 intense storms w/ high water levels at end.
2003, 6/30-7/01	Tropical Storm Bill. Prior to storm, emergency embankments installed. Approx 2,275 ft of 3-ft clay core covered w/ geotextile and broken concrete to a 6 ft elevation.

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3 Cross-Shore Sediment Transport Modeling

This chapter summarizes the procedures and results of storm-induced cross-shore sediment transport analysis using SBEACH as described in Task 2 of the study objectives. The main components of Task 2 are to gather storm wave and water level data for storms that have occurred since 1985, apply SBEACH to profiles of varying berm width to evaluate adequacy of the current berm and current dune design, develop recommendations for any modifications to the design cross-section, and particularly to define the desired berm width to be maintained in the critically eroded area.

Overview of SBEACH

Cross-shore sediment transport modeling was performed using the Storm-induced BEAch Change (SBEACH) model (Larson and Kraus 1989; Larson, Kraus, and Byrnes 1990). SBEACH is an empirical geomorphic-based model developed to simulate two-dimensional cross-shore beach change. Model development was based on extensive analysis of beach profile change produced in large wave tanks and in the field; and extensive validation was performed and documented for severe extratropical and tropical storms events at a number of locations around the country. Breaking waves and varying water level are the major driving agents in beach profile change and cross-shore sediment transport. SBEACH is intended to predict and analyze short-term, storm-induced erosion. A fundamental assumption of the SBEACH model is that profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no net gain or loss of material. Longshore processes are considered to be uniform and neglected in calculating profile change. In areas where strong alongshore transport gradients produce erosion, SBEACH would tend to underpredict beach recession.

Primary data input includes profile, storm, and sediment data. Storm data include time histories of wave height, wave period, and total water elevation (tide plus surge). Median grain size of the beach material is the primary sediment data required. Wave direction and wind speed and direction are options that can be specified, but were not included in the SBEACH simulations reported here. It was assumed that waves approached shore-normal to the beach at the SBEACH input location (water depth of about 16 ft), which is a slightly conservative assumption. Wave direction was considered in estimating wave conditions at the SBEACH input location.

Storm Data

Wave and water level data for storms having an impact on Grand Isle from 1985 to present were assembled. The primary extreme storm events experienced at Grand Isle are hurricanes and intense tropical storms. Storm selection was based primarily on landfall location, but also on wave height and water level elevations associated with the storm. Landfall locations were obtained from the UNISYS re-analysis of Atlantic Hurricane storm track data (http://weather.unisys.com/hurricane/atlantic/). Wave data were obtained from three sources: Wave Information Study (WIS) hindcasts, a special WIS Mississippi Delta (MS-Delta) hindcast using a finer grid, and the National Buoy Data Center (NDBC) sta 42041. Water level data were obtained from the New Orleans District Gage 88410 located at the Coast Guard Station on the bayside of Grand Isle. Supplementary water level data were obtained from NOAA/NOS CO-OPS sta 8761724 also located at the Coast Guard Station on Grand Isle.

Waves were transformed from the offshore depth of the data source to the offshore boundary of SBEACH (16-ft depth plus storm surge). The PHASE III transformation program was used to perform the transformation, and it considers refraction, shoaling, sheltering if appropriate, and spectral evolution, and provides information on nearshore wave height, wave period, and wave direction. Data from the NDBC buoy did not contain directional information; therefore, it was assumed that wave direction was shore-normal. The hindcast data do contain directional information, and it was utilized. The major storms that impacted Grand Isle since 1985 are listed in Table 3-1. The table lists the peak water level relative to NAVD 88, significant wave height H_s at the offshore boundary of SBEACH, and the peak period T_p associated with the peak wave height. Storms are included up to 2002 (no wave data were available for 2003).

Five storms were selected from Table 3-1 to use as input to the SBEACH analysis: Juan (1985), Andrew (1982), Danny (1997), Isisdore (2002), and Lili (2002). They were judged to be the most severe from an erosion standpoint due to their high peak water levels, storm duration, and wave characteristics. In terms of measured peak water level only, these four events are representative of storms that occur every 5 to 7 years, based on the stage-frequency curve shown in the Grand Isle and Vicinity Louisiana Phase 1 General Design Memorandum (GDM) (1979). Isidore and Lili occurred approximately one week apart and were combined as a single storm as input to SBEACH. Figures 3-1 thru 3-4 show storm hydrographs of the selected storms in which the blue lines are the offshore wave height, orange lines are transformed waves defined at the offshore boundary of the SBEACH domain and red lines are water level elevations.

Additionally, a hypothetical design hurricane condition was considered, which was created based on information contained in the GDM (1979) for the event: a 50-year peak water level event with a storm surge of 9.5 ft NAVD 88 (8.0 ft MSL), and a 7.7-ft significant wave height, and 7.3-sec period. A storm hydrograph was generated for the design hurricane, based on the hydrograph of Hurricane Danny, as input to SBEACH (Figure 3-5). The shape of the Hurricane

Danny hydrograph was assumed to be typical, and was scaled to match the peak value.

Table 3-1
Storms Impacting Grand Isle, LA (1985-2002)

Storm	Date	Peak Water Level ft, NAVD 88	Peak H_s ft	T_p at Peak H_s sec
Danny	Aug 1985	2.9	9.7	12.0
Elena	Sep 1985	2.0	4.4	5.0
Juan	Oct – Nov 1985	4.8	10.0	13.0
Kate	Nov 1985	2.0	5.8	5.0
Florence	Sep 1988	2.3	6.8	13.0
Chantel	Jul – Aug 1989	2.8	5.2	9.0
Andrew	Aug 1992	3.8	12.9	13.0
Opal	Sep – Oct 1995	3.4	8.6	14.0
Danny	Jul 1997	4.5	7.3	6.2
Earl	Aug –Sep 1998	3.2	7.2	10.0
Frances	Sep 1998	3.3	4.5	7.3
Hermine	Sep 1998	2.9	4.7	8.4
Georges	Sep – Oct 1998	2.9	7.2	14.0
Isidore	Sep 2002	5.1	7.7	12.9
Lili	Oct 2002	4.8	12.9	10.3

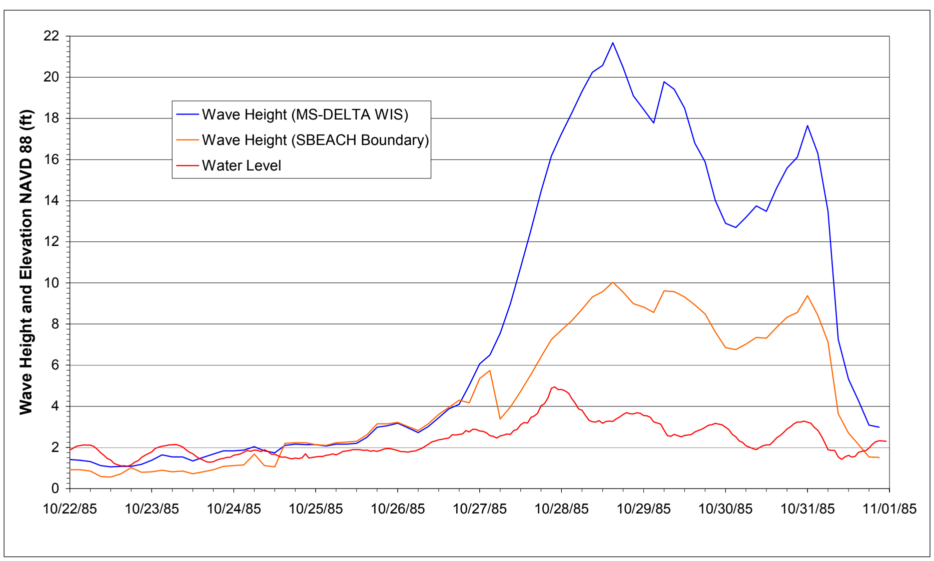

Figure 3-1. Hydrograph of Hurricane Juan

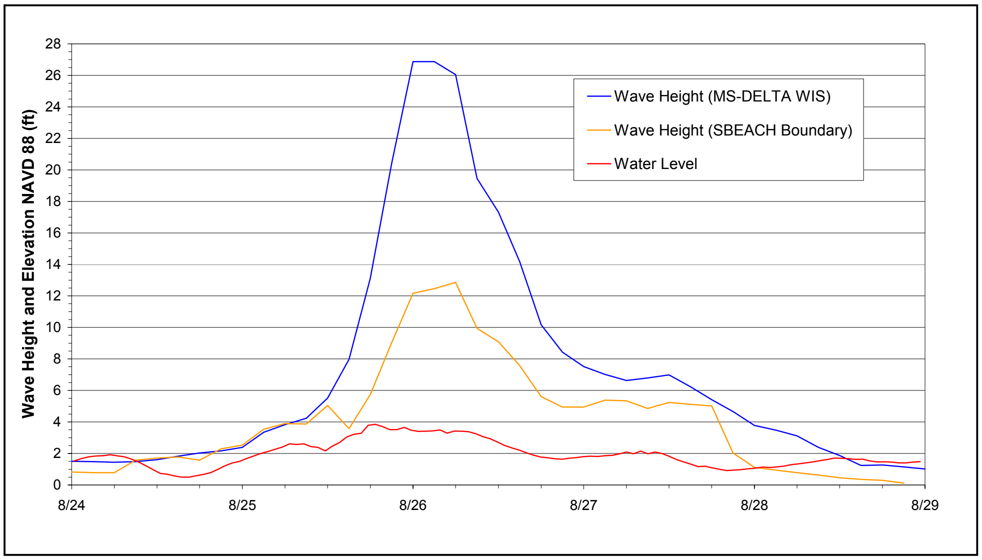

Figure 3-2. Hydrograph of Hurricane Andrew

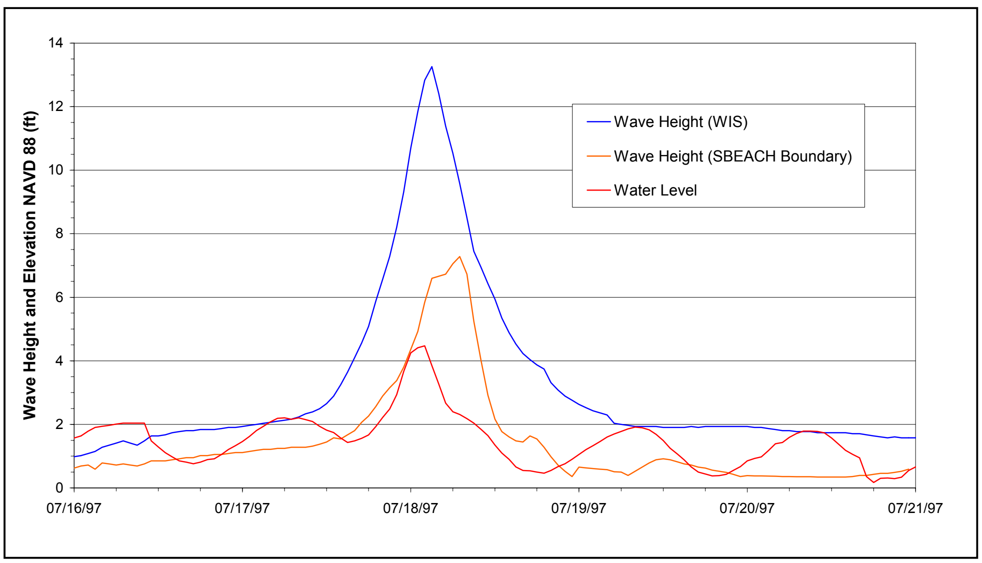

Figure 3-3. Hydrograph of Hurricane Danny

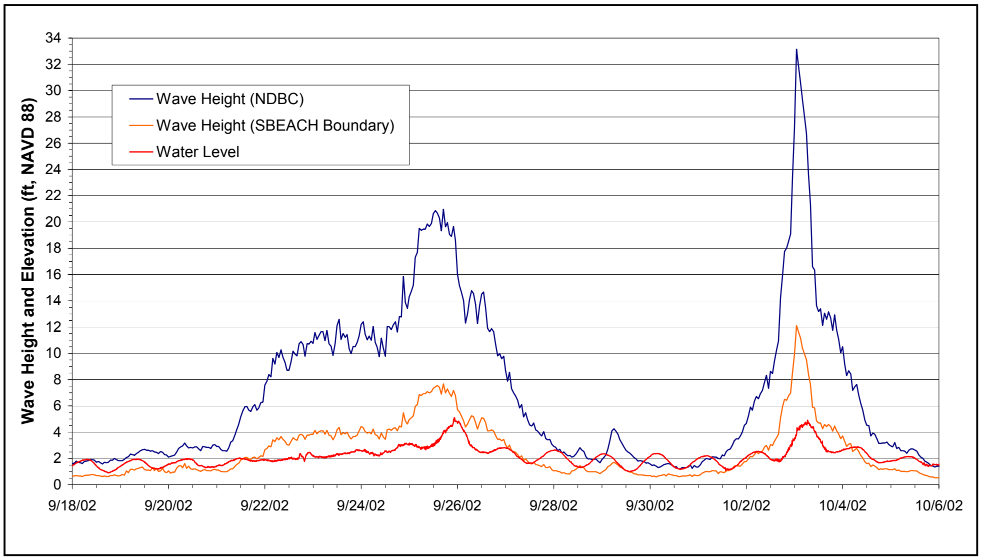

Figure 3-4. Hydrograph of Hurricanes Isidore and Lili

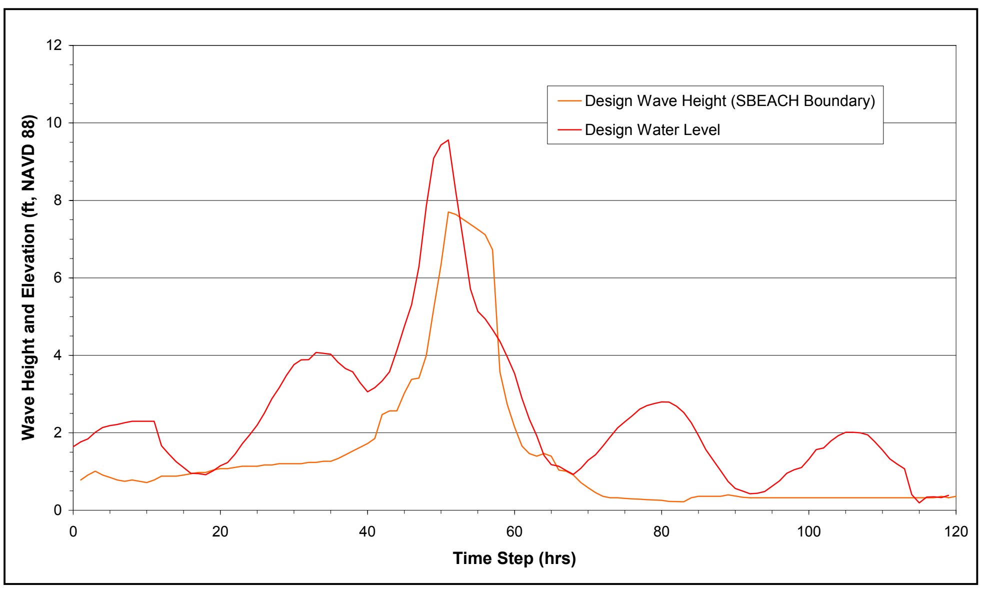

Figure 3-5. Hydrograph of Design Hurricane

SBEACH verification

Prior to performing analysis on beach fills, SBEACH was applied to eight profiles from a March 1994 survey of the island. The resulting profiles were compared to profiles from the subsequent year (March 1995). The profiles evaluated were located at sta 140+00, 160+00, 200+00, 220+00, 240+00, 280+00, and 320+00 (Figure 2-8). The purpose of the runs was to evaluate the default coefficients in SBEACH and adjust them if necessary. A series of winter storms occurring over 45 days between November 1994 and January 1995 were used as input to SBEACH. The hydrograph of the winter storm sequence is shown in Figure 3-6. The data set is not ideal for model evaluation because of the considerable length of time between pre- and post-storm surveys; but these were the best available data.

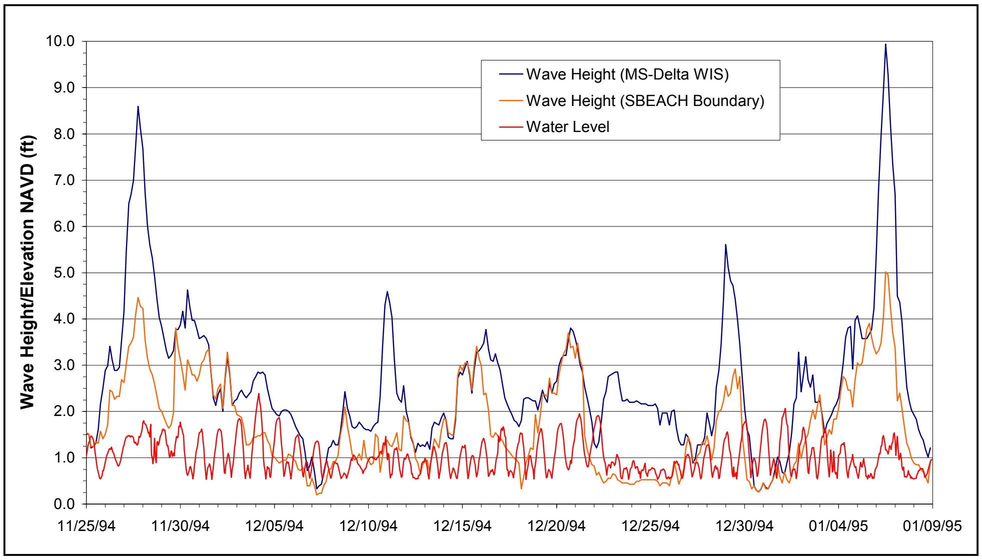

Figure 3-6. Hydrograph of 1994-1995 winter storms

Two examples of the verification are shown in Figures 3-7 and 3-8. Figure 3-7 shows the resulting SBEACH profile applied to the 1994 profile at sta 180+00, which is located approximately 1800 ft west of the Town Rock Project. Predicted recession at the mean lower low water (MLLW) and mean high water (MHW) compare well to the 1995 profile. The SBEACH profile also compares well to the 1995 profile. Station 180+00 is located in the stable portion of the island and little erosion is observed between 1994 and 1995. Reasonable agreement was observed for other profiles in more stable areas.

Comparison of SBEACH results to the measured profile at sta 220+00 is shown in Figure 3-8. Station 220+00 is in the critical area (approximately 2,200 ft east of the Town Rock Project). Between 1994 and 1995, greater shoreline retreat is observed at sta 220+00 than at sta 180+00. SBEACH does not predict the magnitude of erosion shown by the 1995 survey at this location. Based on the location of sta 220+00, part of the discrepancy may be due to loss of sediment due to longshore effects (i.e., a gradient in longshore transport induced by the presence of the TRP that produces persistent erosion as has been observed in this area, see Chapter 4). SBEACH is a two-dimensional model and does not account for gradients in longshore transport. As sand begins to bypass the TRP, and enter the critical zone, or especially if beach fill is placed, the role of longshore gradients in determining storm-induced beach response will be reduced. Therefore the SBEACH results are a reasonable indicator of beach erosion with the fill in place.

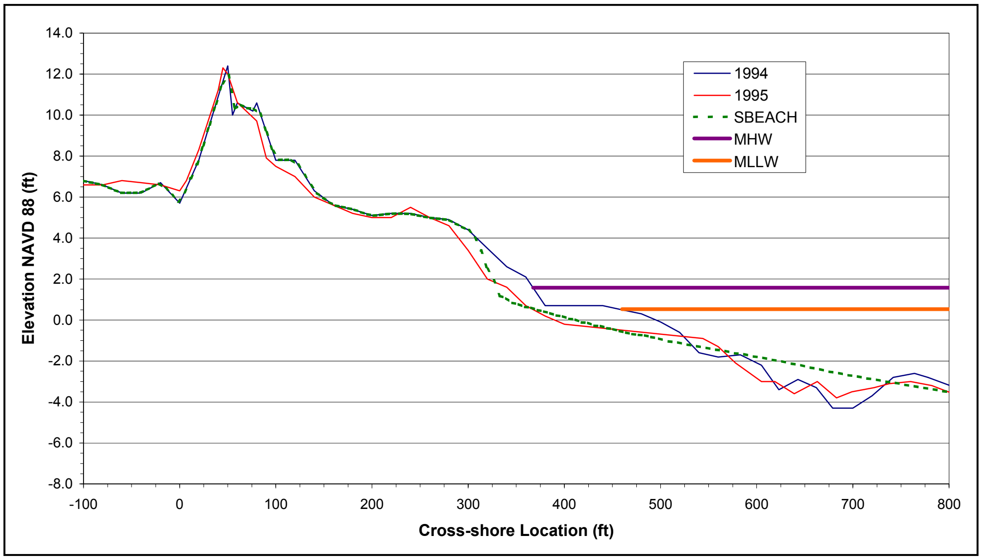

Figure 3-7. SBEACH results at sta 180+00

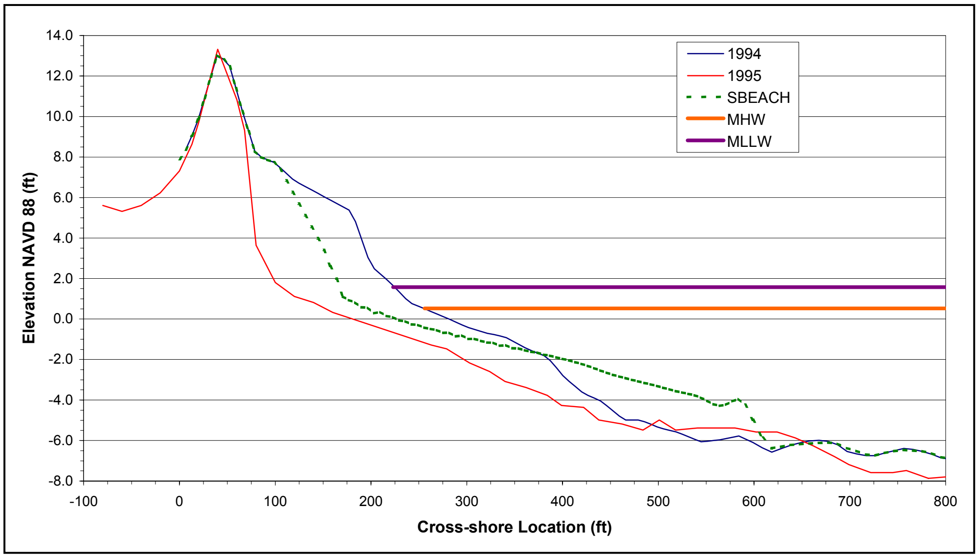

Figure 3-8. SBEACH results at sta 220+00

Figure 3-9 shows predicted SBEACH erosion volumes versus measured erosion between MLLW and the berm crest for the eight profiles. Most of the points fall near the line of perfect prediction, slightly below, with the exception of the result for sta 220+00. Considering that the time between surveys is one year (less than ideal from a model validation standpoint), and the likely role of alongshore transport gradients at certain locations, the comparison is considered reasonable. Predicted versus measured recession at MLLW and MHW are shown in Figure 3-10. There is considerable scatter in the results. It was concluded from the verification attempt that the default coefficients in SBEACH are reasonable for use in performing beach fill analysis on Grand Isle, and that no changes to the defaults are justifiable.

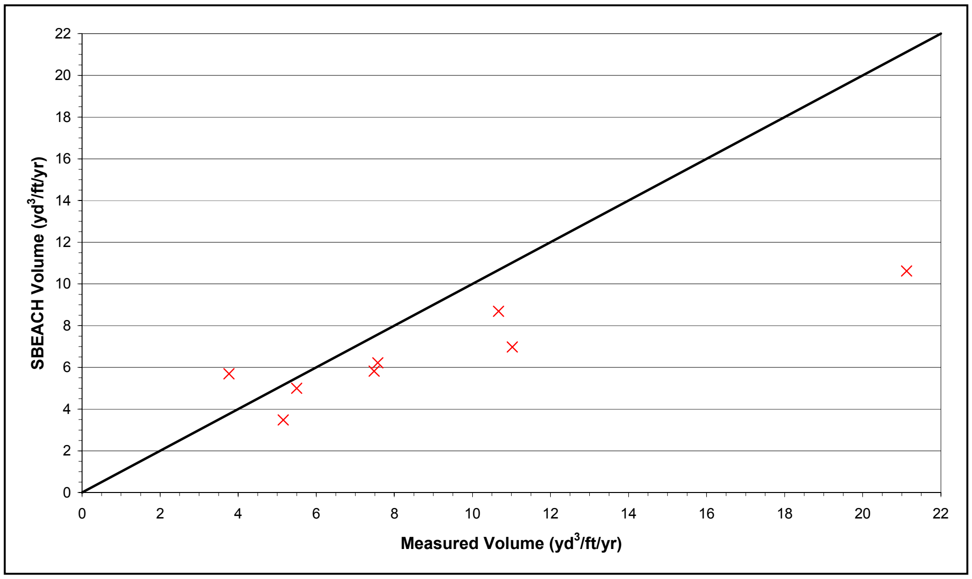

Figure 3-9. Predicted versus measured erosion volume

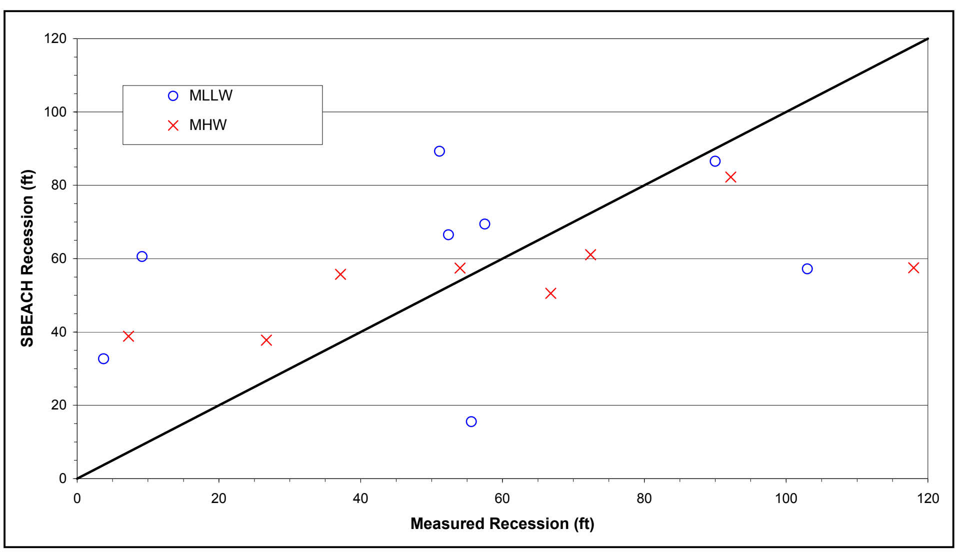

Figure 3-10. Predicted versus measured recession at MLLW and MHW

SBEACH analysis on existing profiles

SBEACH analysis was conducted on the existing conditions in the critical area based on 2003 surveys. Six profiles at stas 210+00 through 260+00 (Figure 2-8) were subjected to the four historical storms shown in Figures 3-1 through 3-4. SBEACH results are shown in Figures 3-11 through 3-16. Between sta 210+00 and 250+00, the combined storms of Isidore and Lili completely eroded the existing dune. The dune elevation at sta 230+00 (Figure 3-13) is significantly lower than elevations at the other profiles in the project area, and was severely impacted by all the storms. Hurricane Juan eroded the dune at sta 220+00 (Figure 3-12) and significantly lowered the dune elevation at the other profiles between sta 210+00 and 250+00. This storm produced high erosion due to its long duration. The dune at sta 210+00 (Figure 3-11) suffered little erosion due to Hurricanes Andrew and Danny; however, erosion of the beach fronting the dune was observed. In general, erosion caused by Andrew and Danny was less severe than the other two storms. The results are consistent with the observed response of the critical area to recent severe storms.

Station 260+00 (Figure 3-16) is located on the eastern end of the project area and consists of a wide beach fronting the dune. Although erosion by all storms was evident, the dune maintained its elevation. This result reveals the importance of having a wide berm landward of the dune, to “absorb” the erosive effects of the higher water levels and wave conditions associated with severe storms.

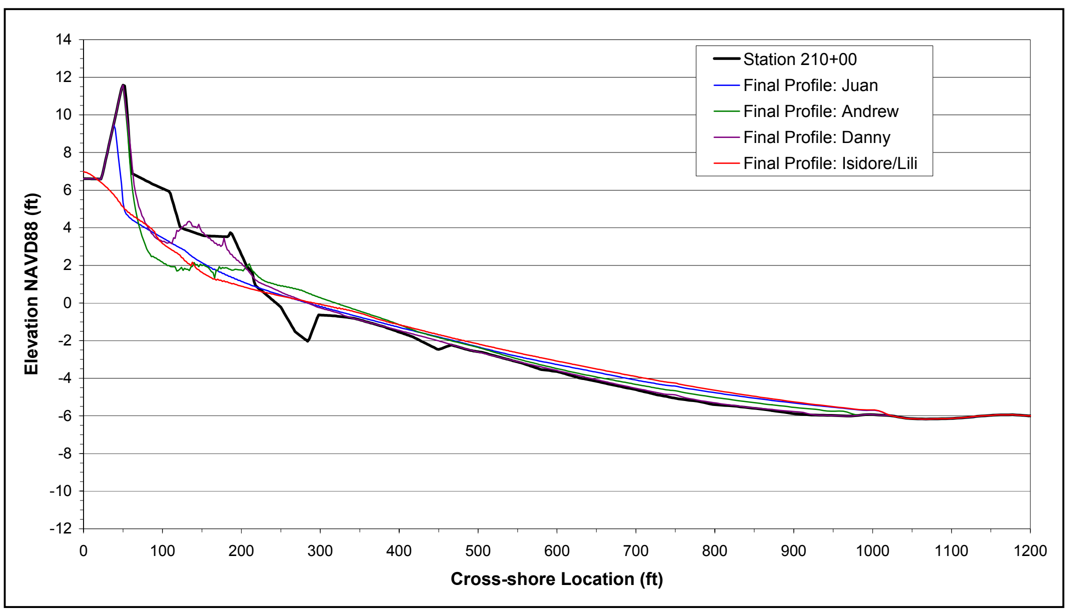

Figure 3-11. SBEACH results at sta 210+00 for existing conditions

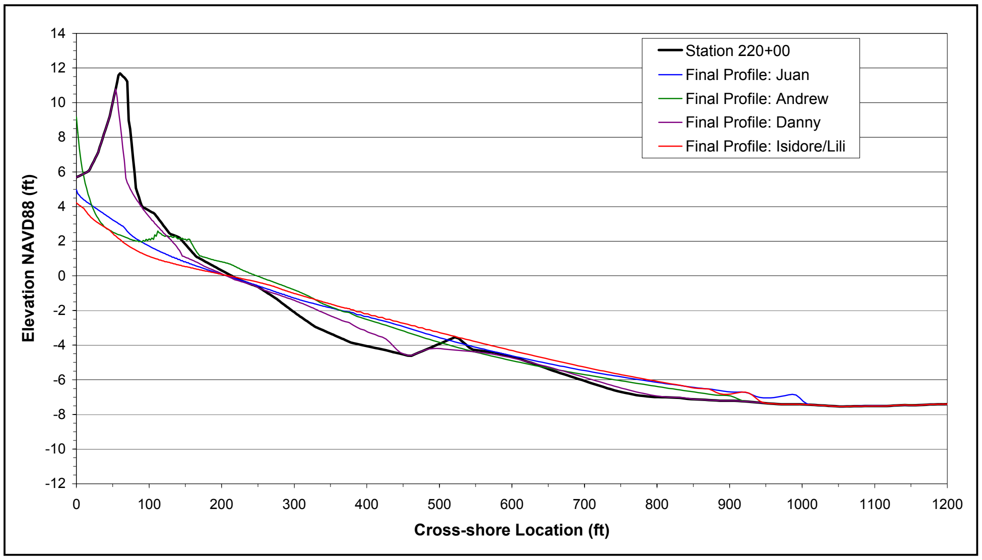

Figure 3-12. SBEACH Results at sta 220+00 for existing conditions

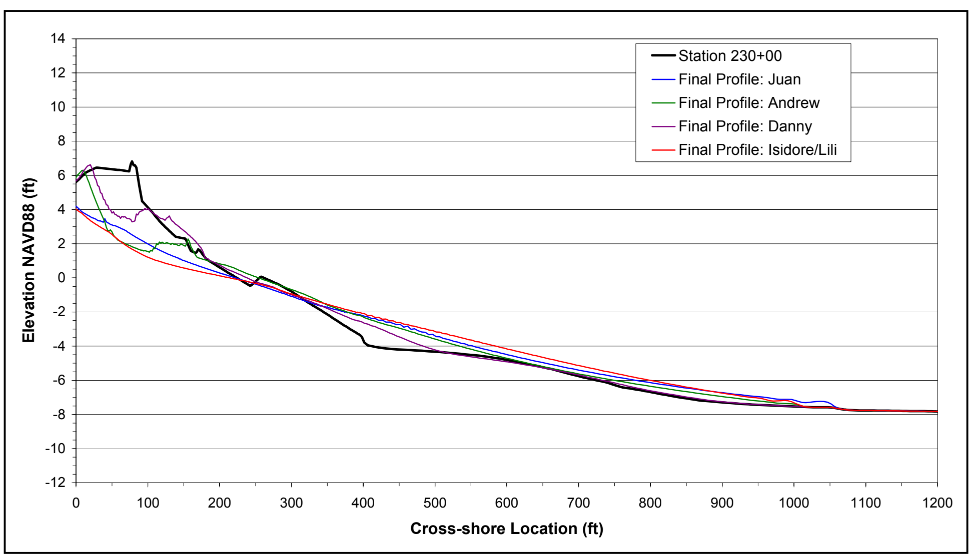

Figure 3-13. SBEACH Results at sta 230+00 for existing conditions

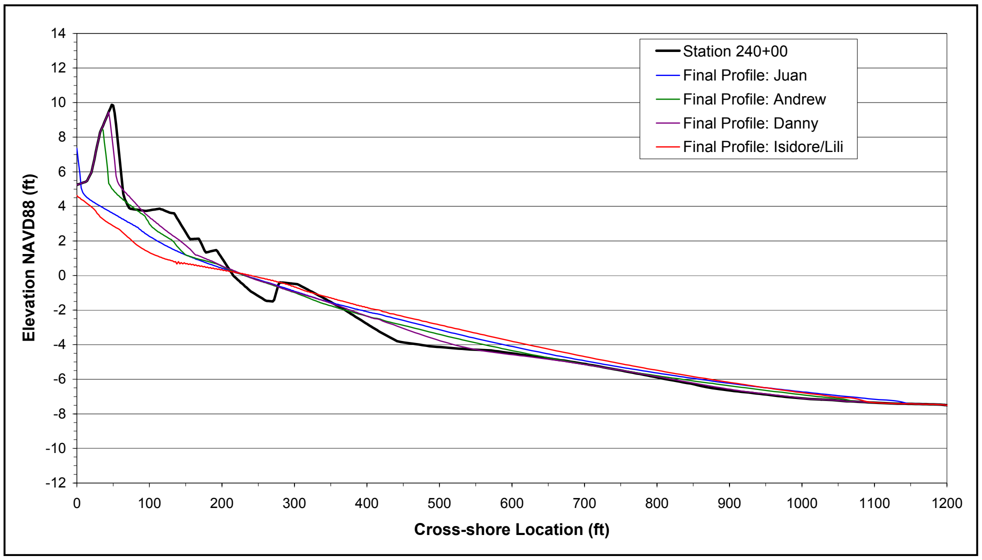

Figure 3-14. SBEACH results at sta 240+00 for existing conditions

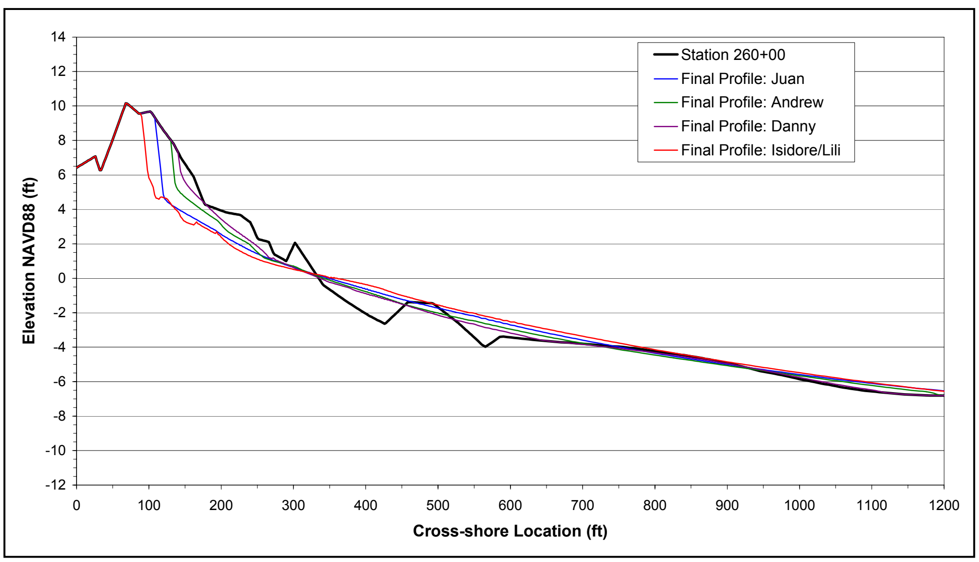

Figure 3-15. SBEACH results at sta 250+00 for existing conditions

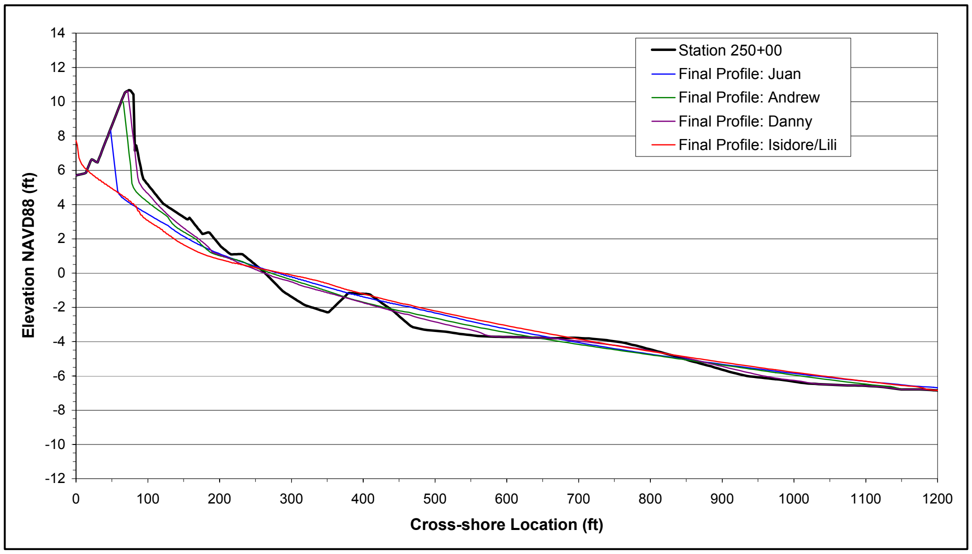

Figure 3-16. SBEACH results at sta 260+00 for existing conditions

Design Profiles

SBEACH results are consistent with past observations of beach and dune erosion at the stite, and they show that Grand Isle is at risk of flooding from storms of the magnitude encountered since 1985. To assess the requirements for providing protection from these types of events, SBEACH simulations were conducted for added beach widths and higher dune configurations.

Selection of a berm height was determined by analyzing profiles from a sediment rich portion of the island. Profiles in a highly eroded state may not show the signature of a natural berm. The western end of Grand Isle is stable and profiles between sta 60+00 to 90+00 were examined. To determine a representative berm height, historical profiles at the stations were horizontally aligned so that all are superimposed at mean sea level (MSL). As an example, Figures 3-17 and 3-18 show horizontally aligned historical profiles from sta 60+00 and 80+00, respectively (survey dates are given in the legend). The figures indicate that the higher natural berm crest is approximately +5 ft MSL, or about +6 ft NAVD 88. Therefore, the berm elevation used for beach fill analysis and subsequent modeling was selected as +6 ft NAVD 88. The dune configuration recommended in the GDM (1979) was used for beach fill analysis. The recommended dune consists of a +12 ft NGVD dune elevation, a 10-ft-wide dune crest, and 1 vertical to 5 horizontal side slopes (note that NAVD 88 is 0.28 ft below NGVD 29).

Input profiles to SBEACH were constructed by placing the design dune configuration above the existing dune with the seaward slope extending to +6 ft NAVD. The existing profile below +6 ft NAVD 88 was translated a given distance, and tied to the base of the dune. An example profile is shown in Figure 3-19 for sta 240+00 in which the existing profile has been translated seaward by 75 ft.

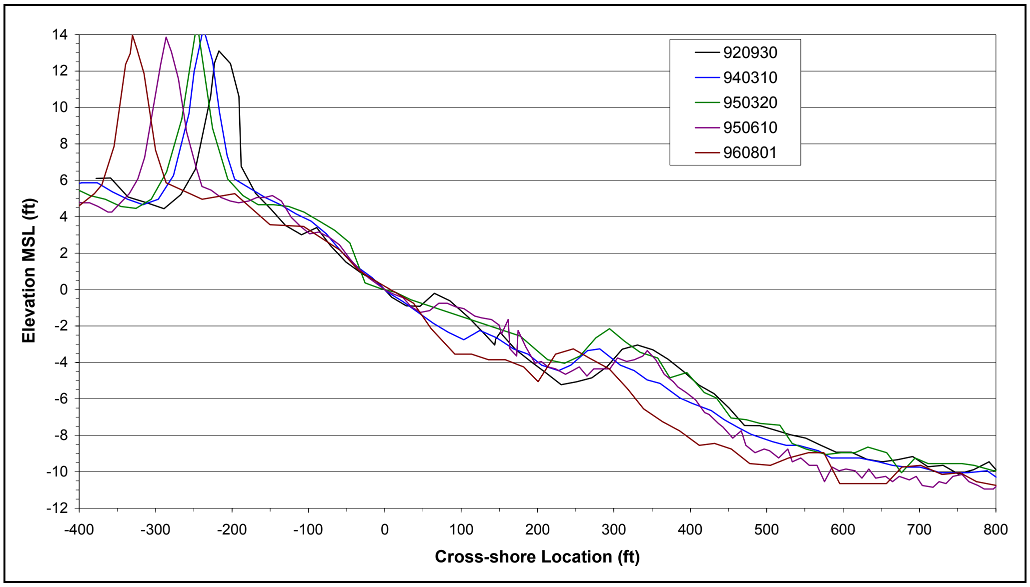

Figure 3-17. Horizontally aligned profiles at sta 60+00

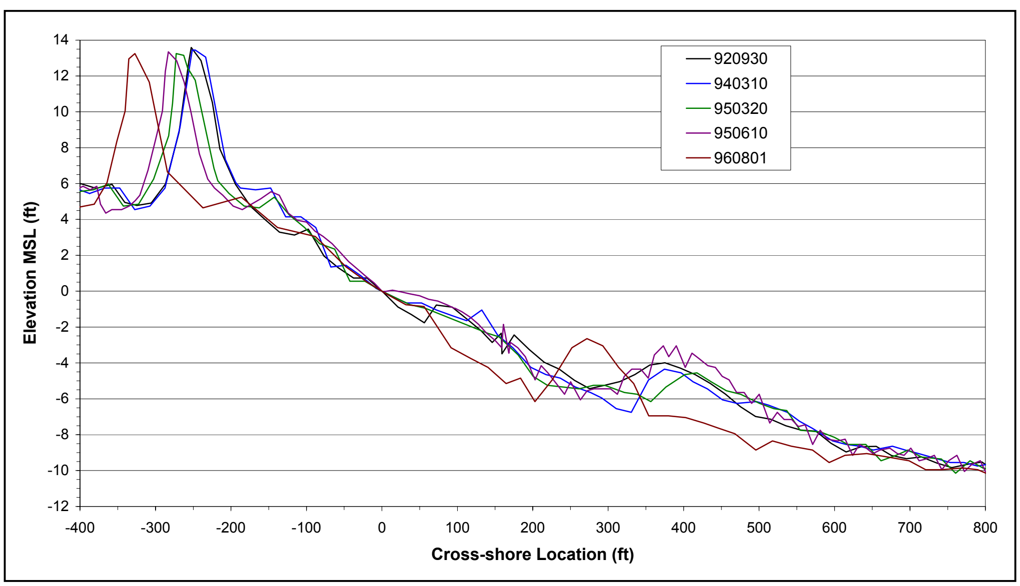

Figure 3-18. Horizontally aligned profiles at sta 80+00

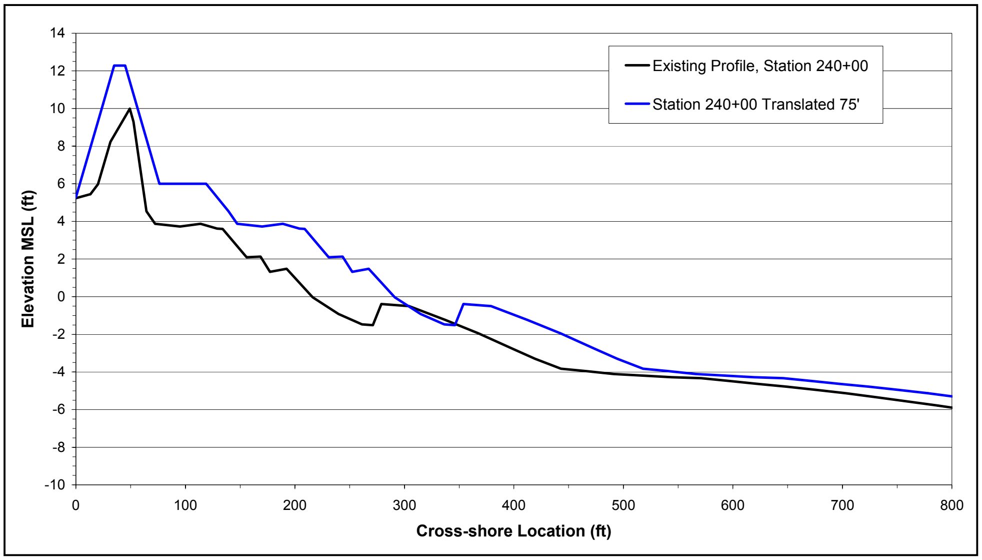

Figure 3-19. Existing profile and 75 ft translated profile at sta 240+00

SBEACH analysis: Historical storms

Profiles in the project area were examined for varying beach widths to determine the necessary width to protect Grand Isle from the four historical storms. To optimize beach width, profiles were subjected to the four historical storms in SBEACH and the translated portion of the beach was adjusted. A successful beach width was determined if the dune maintained its elevation for each of the storms.

Computed post-storm beach profiles determined from SBEACH for sta 220+00, 240+00, and 260+00 are shown in Figures 3-20, 3-21, and 3-22, respectively. Station 220+00 (Figure 3-20) required an added width of 125 ft. The beach was eroded to the dune for Hurricanes Juan and Isidore/Lili; however, the dune elevation was maintained. Figure 3-21 shows the resulting profile at sta 240+00 with 75 ft of added beach width. Hurricanes Juan and Isidore/Lili eroded the beach to the dune, and the dune elevation was slightly lowered by Isidore/Lili. The lower elevation was slight (0.1 ft) and was considered successful. Only the new dune configuration was added to the profile at sta 260+00 and no added beach width was required (Figure 3-22). Table 3-2 lists the added beach widths required for each station in the project area based on SBEACH analysis. In addition, the table includes maximum runup elevation estimated by SBEACH.

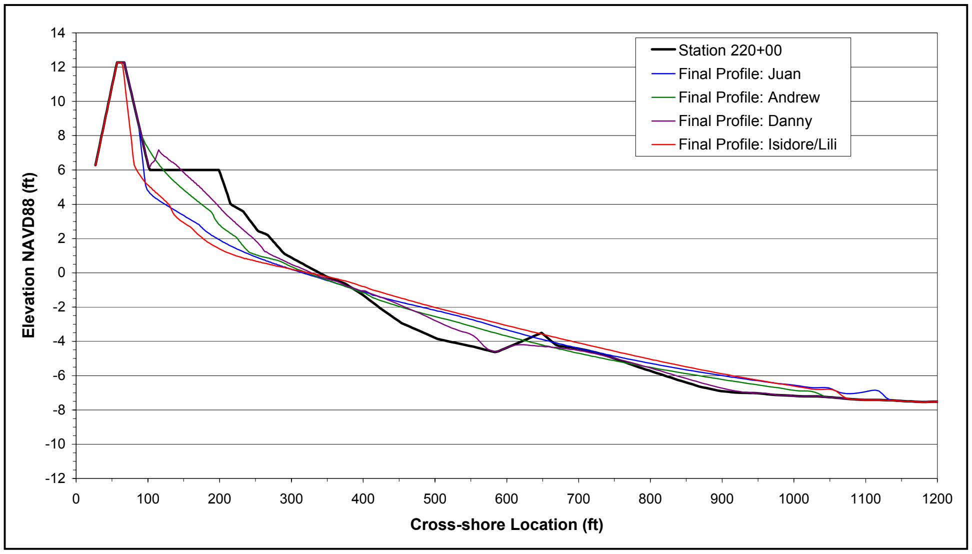

Figure 3-20. SBEACH results at sta 220+00 with 125 ft added beach width

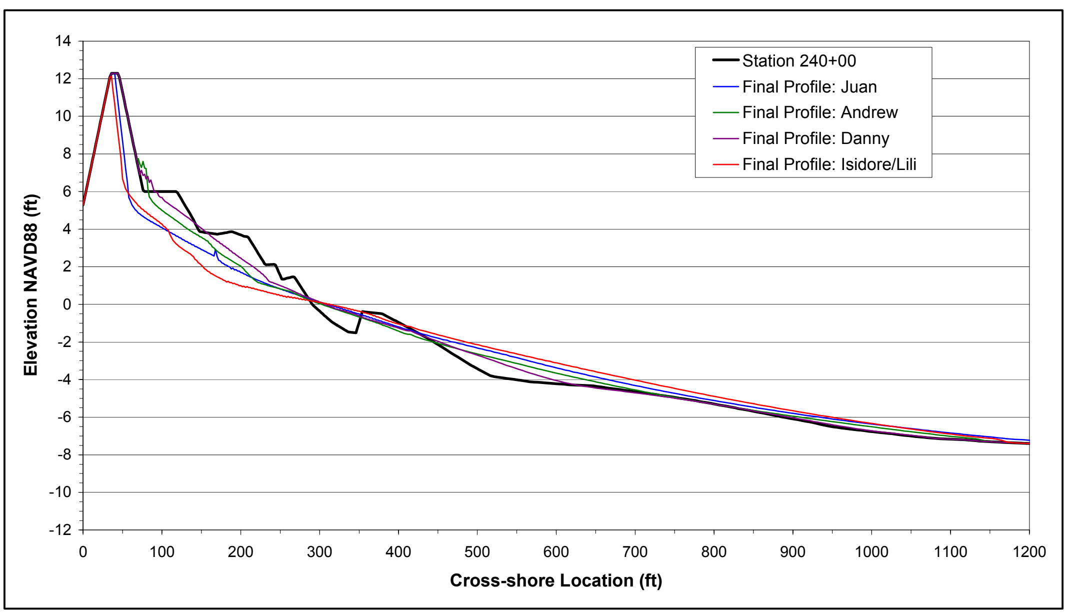

Figure 3-21. SBEACH results at sta 240+00 with 75 ft added beach width

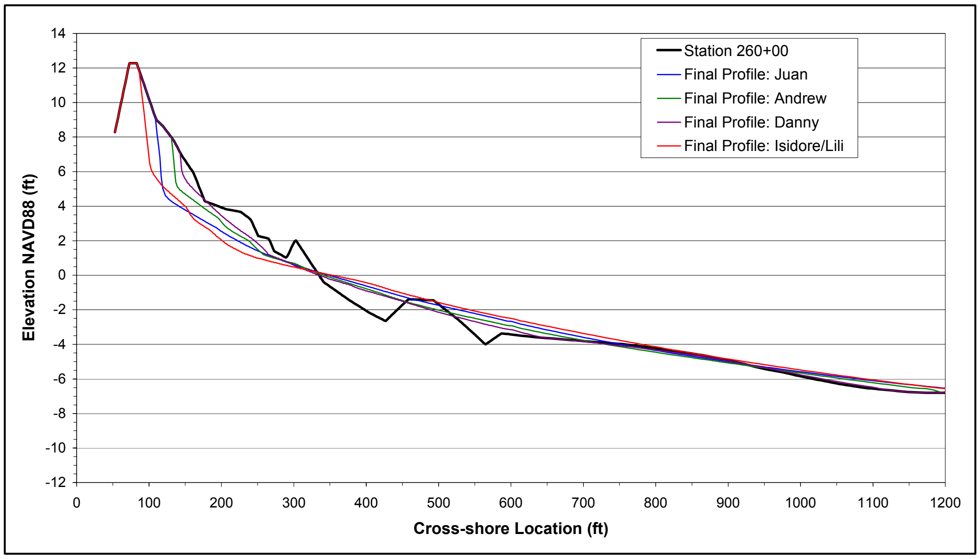

Figure 3-22. SBEACH results at sta 260+00

Table 3-2
Minimum Added Beach Widths

Station	Added Beach Width ft	Maximum Runup Elevation ft, NAVD 88	Storm with Max Runup
210+00	50	8.2	Juan
220+00	125	9.4	Isidore/Lili
230+00	75	9.6	Isidore/Lili
240+00	75	8.4	Isidore/Lili
250+00	75	8.1	Isidore/Lili
260+00	0	8.5	Isidore/Lili

SBEACH was applied to the profiles in the project area to examine the influence of the detached breakwaters on storm-induced erosion. The breakwaters were simulated as a hard bottom in SBEACH. The hard bottom does not allow transmission through the structure and the structure must remain submerged during the simulation. However, the breakwaters at Grand Isle were designed to allow wave transmission through the structure. The breakwaters have a rather low crest elevation. During very high water levels, the breakwaters are submerged, and high water and waves still impact the dune. The breakwaters also aid in dissipating energy to a degree, but their dissipation effectiveness diminishes with increasing water level. Additionally, wave energy is transmitted three-dimensionally through the gaps between the breakwaters, which isn't simulated in the two-dimensional model of SBEACH. For the crest elevations of the Grand Isle breakwaters, only a portion of the historical storms time series would submerge the breakwater and allow wave energy to reach the shoreline.

In an attempt to allow more transmission energy of the storms to more appropriately simulate real world conditions, the breakwater crest was lowered in the

SBEACH application. Without calibration data, it was difficult to determine the appropriate crest elevation that would accurately model the Grand Isle breakwaters. It was felt that an elevation approximately equal to MSL would allow energy to be transmitted, but would provide reduction in wave height in the manner of the breakwaters. The minimum water elevation of the Juan and Isidore/Lili storms, the storms which cause the greatest beach change, was +0.91 ft NAVD 88, which slightly less than MSL (+1.07 ft NAVD 88). Because the difference in elevations is small, the breakwater crest was lowered to +0.91 ft NAVD 88, which allowed simulation of the entire Juan and Isidore/Lili storms.

SBEACH was applied to profiles between sta 210+00 and 250+00 (sta 260+00 required no added beach width). Computed post-storm profiles from sta 220+00 and 240+00 are shown in Figures 3-23 and 3-24, respectively. Results at sta 220+00 /Lili indicate the dune remains nearly stable with a 100-ft beach width (erosion of the dune face occurs during Isidore/Lili). Without the breakwater, the profile was stable for a 125-ft added beach width. At sta 240+00, the dune was stable for a 75-ft added beach width. The added beach width is the same as required without the breakwater, and results look similar to those without the breakwater (Figure 3-21). Results from the other stations indicated that the beach widths required for a stable profile at sta 210+00, 240+00, and 250+00 were the same with or without a breakwater, and the added beach width at sta 230+00 could be reduced by 25 ft if fronted by a breakwater. The conclusion from these tests is that the breakwaters do not provide significant value for beach and dune protection for severe storms. Therefore, the design was done using results from simulations that excluded the breakwater.

Figure 3-23. SBEACH results at sta 220+00 with breakwater and 100 ft added beach width

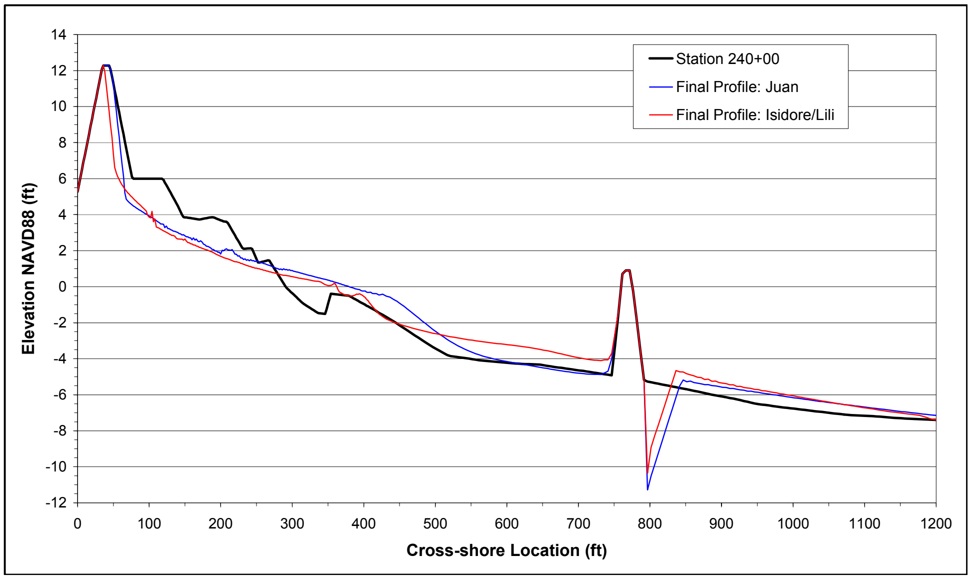

Figure 3-24. SBEACH results at sta 240+00 with breakwater and 75 ft added beach width

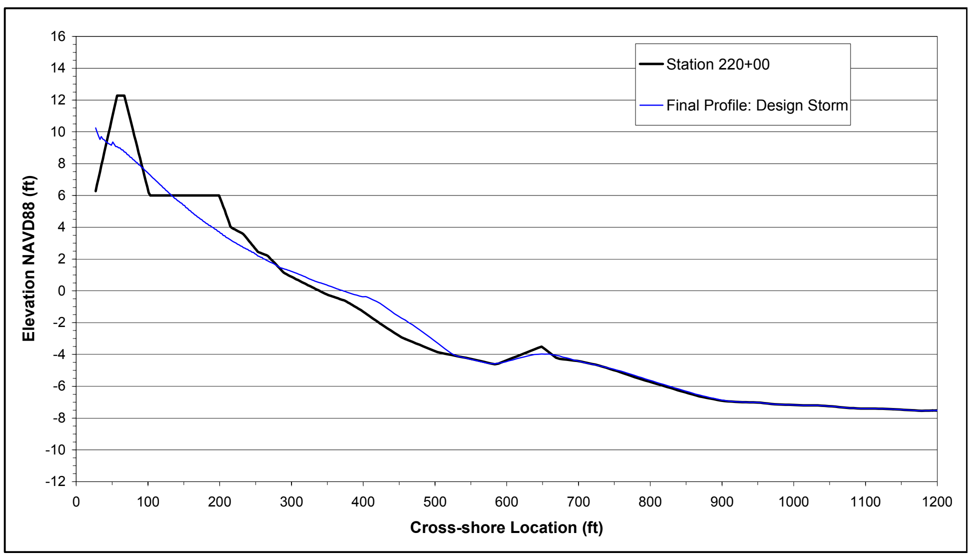

Figure 3-25. SBEACH results for design storm at sta 220+00 with 125 ft added beach width

Note that the calculated erosion at the seaward toe of the structure is probably overstated, and the process is not accurately modeled. Sand is likely to be transported offshore through the gaps between breakwaters during a severe storm. This influx of sand would act to counter the effects of scour formation at the toe of the breakwater.

SBEACH analysis: Design storm

The profile at sta 220+00 was subjected to the design storm, which was based on storm characteristics defined in the GDM (1979). The added beach width of 125 ft that provided protection for the historical storms was used as the initial profile in SBEACH. The resulting profile is shown in Figure 3-25 and shows considerable erosion of the dune. Maximum runup elevation estimated in SBEACH was +13.6 ft NAVD 88. Because of the high water elevation associated with the design storm (surge plus set-up plus run-up), increasing the beach width added little protection to the dune. Therefore, the dune configuration was altered and analyzed. Figure 3-26 shows the resulting profile of a dune configuration that survived the design storm. The profile consists of 125 ft added width, a dune elevation of +14 ft NAVD 88, and a dune crest width of 20 ft. The added dune volume is computed to be just enough to withstand the design storm, although had a longer wave period and higher wave height been used to define the design event (higher values for peak height and/or period are seen in Table 3-1), even this dune configuration may experience erosion and overtopping.

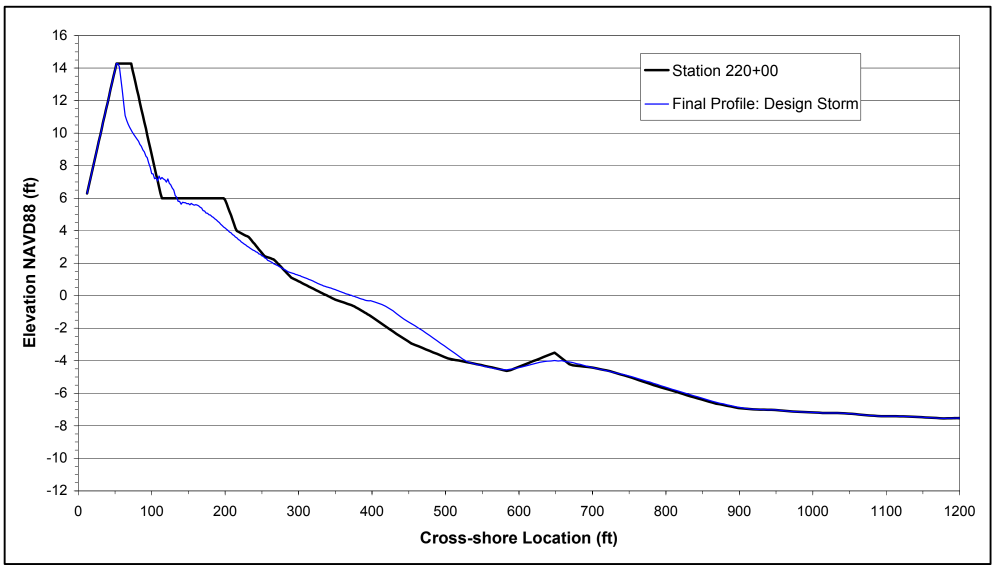

Figure 3-26. SBEACH results for design storm at sta 220+00 with 125 ft added beach width and modified dune

Conclusions

Historical storm wave and water level data were gathered for severe storms that have impacted Grand Isle since 1985. Based on the historical data, four storm conditions were used as input into SBEACH: Juan, Andrew, Danny, and a combined storm of Isidore and Lili. Additionally, a design storm created from information in the GDM (1979) was used as SBEACH input. SBEACH results indicated that the existing beach and dune in the critical erosion area do not provide adequate protection from the historical storms.

SBEACH was run using the historical storms on design profiles consisting of a +6 ft NAVD 88 berm height, and varying dune width. The dune configuration given in the GDM (1979), which consists of a +12 ft NGVD elevation, 10 ft crest width, and 1 vertical on 5 horizontal side slopes, was used for the simulations. Beach widths that provided adequate protection were determined for the six profiles in the project area. The necessary beach widths varied by station location and are given in Table 3-2. An SBEACH simulation was performed at sta 220+00 with an offshore breakwater installed. The results indicated that the breakwater does not significantly improve storm protection compared to the cases simulated without the breakwater.

The dune configuration from the GDM (1979) did not withstand the design storm and it was necessary to raise the dune crest to +14 ft NAVD 88 and widen the dune crest to 20 ft to provide protection against this hypothetical event.

References

Larson, M., and Kraus, N. C. (1989). "SBEACH: Numerical model for simulating storm-induced beach change; Report 1: Empirical foundation and model development," Technical Report CERC-89-9, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Larson, M., Kraus, N. C., and Byrnes, M. R. (1990). "BEACH: Numerical model for simulating storm-induced beach change; Report 2: Numerical formulation and model tests," Technical Report CERC-89-9, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

4 Longshore Transport and Shoreline Change Modeling

Introduction

This chapter summarizes the procedures and results described in Task 3 of the study objectives. The long term behavior of various beach fill designs was modeled using GENESIS (GENEralized model for SImulating Shoreline change), a numerical program that predicts longshore sand transport and resulting cross-shore change in shoreline position. Additionally, the model was used to examine plan-form evolution and volume losses from the project area, to estimate the periodic re-nourishment requirements, and to evaluate the performance of the detached breakwaters.

The main GENESIS input is wave information near the seaward edge of the surf zone (the breaker line.) Wave data for this study were available in the form of WIS hindcasts several miles offshore. So first, the numerical model, STWAVE (STeady-state spectral WAVE model), was used to transform these offshore waves to a near-breaking depth. The wave information was then handed off to GENESIS, which calculated the breaking wave parameters, the longshore sediment transport, and the resulting shoreline change.

STWAVE is a computationally intense, steady state spectral wave model that uses a two-dimensional uniform rectilinear grid to transform waves from the offshore region to a near-breaking depth. It solves the complete radiative transfer equation that includes both propagation effects (refraction, shoaling, diffraction, and wave-current interactions) and source-term effects (wave breaking, wind inputs, and nonlinear wave-wave interactions). In this application, refraction, shoaling and breaking were the primary processes that were simulated with STWAVE. As input, the model requires some basic configuration data, a uniform rectilinear bathymetry grid, and directional wave spectra at the seaward boundary of the grid.

GENESIS is a shoreline change model used to simulate longshore sand transport and the resulting cross-shore change in shoreline position. A basic GENESIS assumption is that when erosion or accretion occurs, the entire profile shifts landward or seaward, without changing profile shape, so that only one cross-shore point at each grid cell needs to be tracked. Thus, it belongs to a class of models known as one-line models. At each alongshore grid cell, the model uses the transformed wave data supplied by STWAVE to calculate breaking wave heights and angles, and uses this information to calculate the temporally varying local longshore sediment transport rate. Others inputs include configuration data, shoreline positions, and structure and beach fill locations. Wave diffraction behind the detached breakwaters and wave transmission through the structures were simulated within the wave module internal to GENESIS.

STWAVE Modeling

Two data sets of offshore waves were available for use as input to STWAVE, a 1976-1995 special MS Delta WIS (Wave Information Study) hindcast and a 1990-1999 WIS hindcast. The special MS Delta hindcast was chosen primarily because of its better grid resolution along this section of coastline, which is highly sheltered by the mouth of the Mississippi River. Characteristics of the offshore wave climate for MS Delta WIS station 12, which is in 13 meters of water off Grand Isle, are given in Figure 4-1. Wave angles in this figure are referenced to shore-normal (azimuth of 147^{o}). Straight onshore is zero degrees. Waves approaching the coast from the left (for a person standing on the beach looking offshore) are positive. About a third of the time the waves are traveling offshore. Statistics for these offshore-traveling waves are not included in Figure 4-1.

Offshore bathymetry data collected in the 1980's were obtained from NOAA. These were combined with 1996 profile data collected off Grand Isle ("long lines" discussed in Chapter 2) and are presented in Figure 4-2. This figure shows the boundaries of the STWAVE grid which has 117 cells in the cross-shore direction and 172 cells in the along shore direction, each 250 ft apart. The model propagates the spectral wave energy from the offshore edge of the grid to the save stations in near breaking water depths. Two depths were considered for the save stations, as shown in Figure 4-2. Though the largest waves in the data record would already have broken at the shallower station (8 ft depth), there are relatively few of these (Figure 4-1), and the shallower depth allows more of the complex wave refraction, particularly over the large Barataria ebb shoal, (the left-hand feature in Figure 4-2) to be more accurately represented in the model. Therefore, the shallower save stations were used in this modeling effort.

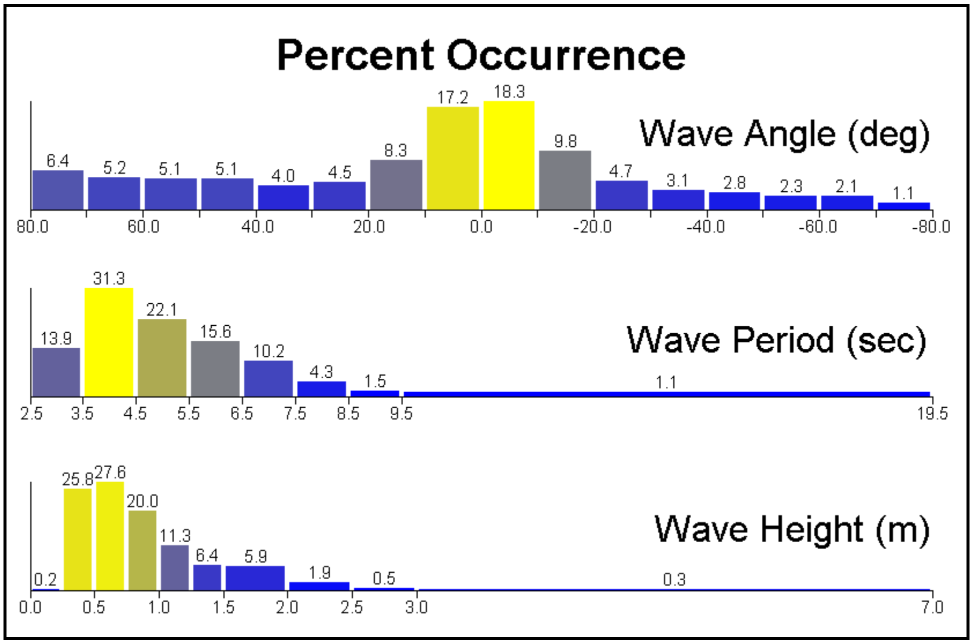

Figure 4-1. Distribution of wave heights, periods, and directions from MS Delta WIS station 12 off Grand Isle

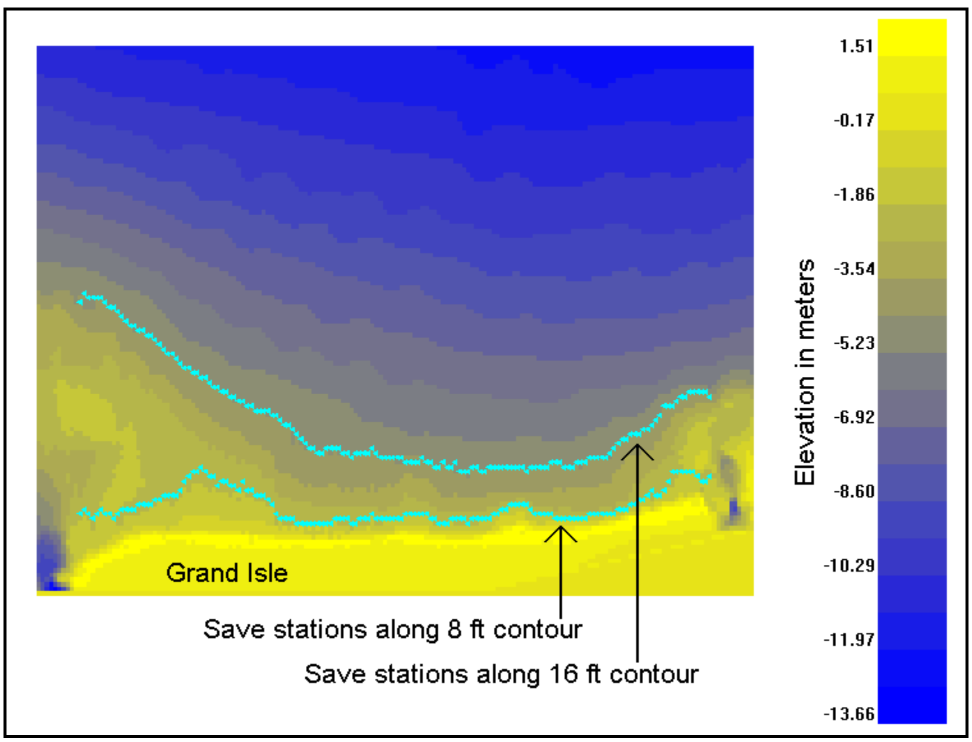

Figure 4-2. STWAVE bathymetry grid and save station locations

The STWAVE output is summarized in Figure 4-3. Panel A shows the locations of the output save stations, as the line of dots seaward of (above) the beach (and seaward of the detached breakwaters) that approximately follows the 8-ft depth contour. Panels B and C show the average save station wave heights and angles, respectively. These panels were produced by averaging the STWAVE output wave heights and angles at each save station over the entire 20-year data set.

Within GENESIS, the longshore transport is typically most strongly dependent upon wave heights, wave angles, and shoreline orientations. In locations where the shoreline is straight and the average wave angles do not vary, variations in the average wave height (Panel B) will give a relative indication of the gross transport rate. In locations were the shoreline is straight and the wave heights are constant, wave angle lines (Panel C) that slope upward to the right (for the sign convention used here) are indicative of areas of erosion, and conversely, lines that slope upward to the left are indicative of areas of accretion. Bumps in both lines in the vicinity of 20,000 and 25,000 ft are the results of dredge holes in the offshore bathymetry that produced the large salients seen in the beach at these locations in the 1980s. The influence of inlet ebb shoals at both ends of the island, particularly the Barataria shoal at the eastern end, are also clearly evident.

At Grand Isle, the shoreline is not straight (particularly at the state park at the eastern end of the island, and the transport rate is substantially influenced by the man-made groins and breakwaters, so this analysis of the save station wave data gives an incomplete picture of the longshore transport. What it does give is an important indication of how the offshore bathymetry is trying to drive the sediment transport. In this respect, it is seen that the trends indicated in panel C, erosion on the western end of the island and accretion on the eastern end, are in general agreement with the long-term shoreline trends for Grand Isle, as discussed in Chapter 2 (Figure 2-11). Comparable panel B and C plots of the STWAVE output from the 16-ft save stations showed much flatter lines, indicating that important nearshore refraction effects, particularly over the Barataria ebb shoal, would be lost if the deeper stations were used.

GENESIS Setup

The GENESIS grid was set up on the eastern half of Grand Isle with 749 cells at a 25-ft spacing between the east jetty and the Town Rock Project. Figure 4-4 shows the east end island shoreline (green), the breakwaters and groins (red), and the spacing of every tenth grid cell (blue vertical lines). The fine grid resolution was necessary to resolve the diffraction behind the breakwaters. The GENESIS grid was aligned with the STWAVE grid so that each STWAVE save station corresponds to every tenth GENESIS grid cell.

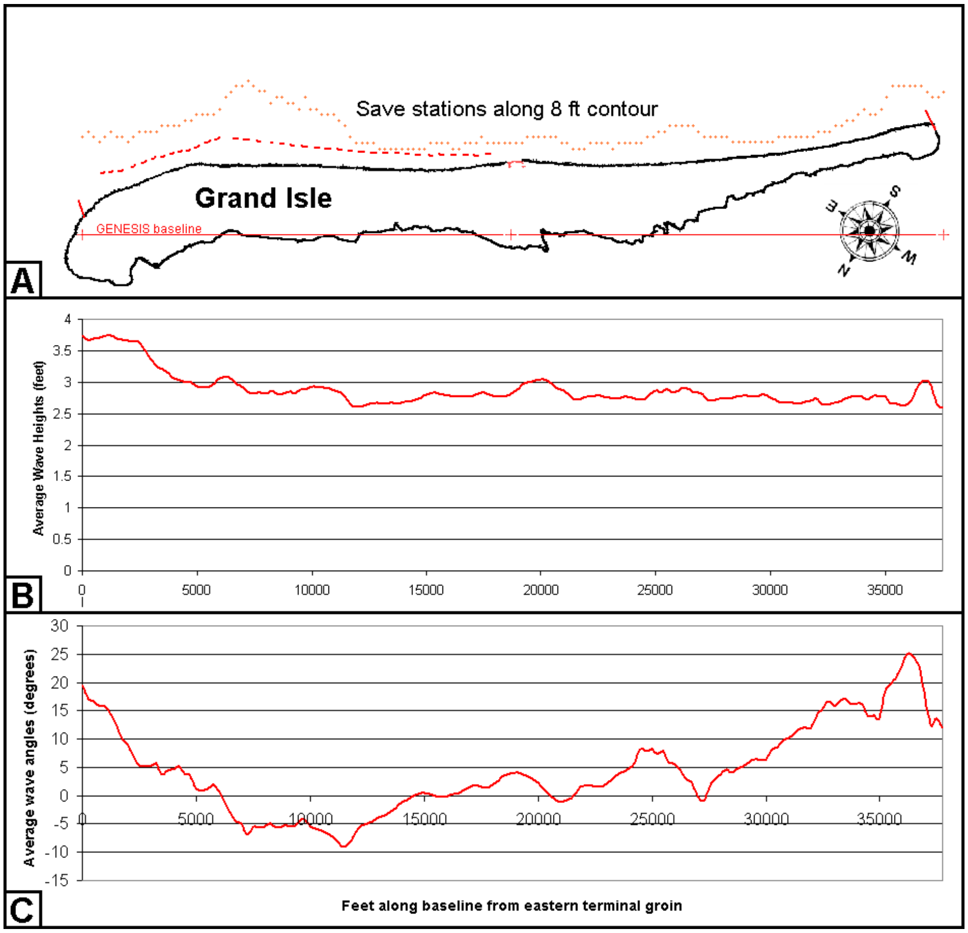

Figure 4-3. Average wave heights and angles at STWAVE save stations.

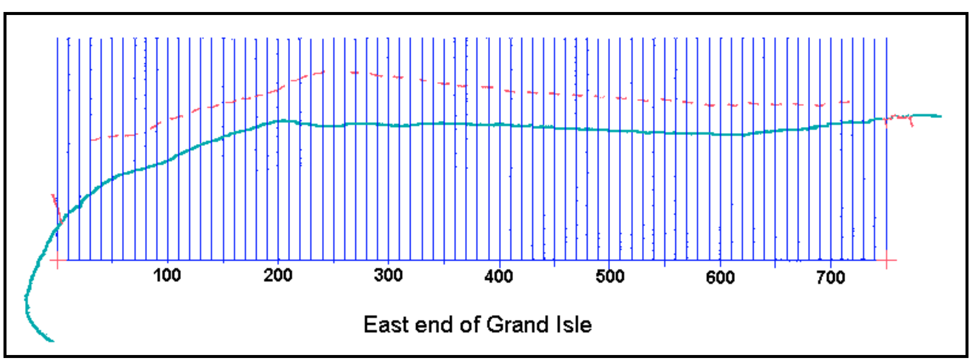

Figure 4-4. GENESIS grid on eastern end of Grand Isle, LA.

GENESIS Calibration and Verification

Calibration

Calibration of GENESIS consists of running the model over a time interval from the date of one measured shoreline to the date of a later one. The model uses the earlier shoreline as the initial condition and parameters are adjusted so that the model output matches the later measured shoreline as closely as possible. To model the effects of the Federal breakwaters, post-1995 shorelines were used for calibration. The shorelines listed in Chapter 2 are of two types, those derived from profile measurements and those derived from aerial photographs. Since the profiles were spaced too far apart to resolve the salients that formed behind the breakwaters, the available calibration shorelines were limited to those derived from aerial photographs dated 12/9/1996, 2/23/1998, 11/7/2002, and 7/11/2003. The calibration period chosen was between 12/9/1996 and 11/7/2002. This selection spans most of the available time period and the two shorelines occur at approximately the same time of year, which helps reduce shoreline variations caused by seasonal trends in cross-shore processes. For the calibration, the STWAVE application used the average wave climate as the driving force.

The calibration parameters used in this model are listed in Table 4-1. These are all fairly typical values. A comparison of the measured and predicted shoreline change rates are shown in Figure 4-5. The measured shoreline change rate is shown in blue and the GENESIS predicted rate is shown in black. There is reasonably good agreement along the western half of the grid, which contains the critical area (shown by the red bar). There is reasonable agreement along the eastern portion, which is within the direct influence of the ebb shoal at Barataria Pass. The modeling done here only considers wave-driven longshore sand transport, and neglects the role of tidal currents as a transporting process. Consideration of those processes was beyond the scope of work, and neglecting them was not thought to adversely influence model results in the vicinity of the critical area. Of course, with increasing proximity to the inlet, the role of tidal currents is expected to increase. In the eastern portion, the general structure of shoreline change is replicated, but model results depart significantly from measured rates at certain locations. A discussion of the effectiveness of the breakwaters and the influence of Barataria inlet and ebb shoal on the eastern end of Grand Isle is presented in the next section. A detailed comparison of the predicted shoreline and the 11/7/2002 measured shoreline in the western portion of the grid is shown in Figure 4-6. Agreement is fairly good in the critical area. Note the 4:1 vertical distortion in this figure.

Table 4-1 **GENESIS Calibration Coefficients**	
Parameter	**Value**
K	0.1
K_2	0.05
D_{50} Grain Diameter	0.13 mm
Berm Height	6 ft
Depth of Closure	12 ft
Number of Grid Cells	749
Cell width	25 ft
Total Width of Grid	18725 ft
Left Boundary Condition Type	moving
Left Boundary Condition Value	-0.088 ft/day
Right Boundary Condition Type	moving
Right Boundary Condition Value	+0.003 ft/day
Initial Shoreline Aerial Photograph Date	12/9/1996
Final Shoreline Aerial Photograph Date	11/7/2002
Simulation Start Date	19961201
Simulation End Date	20021101
GENESIS Northing Origin (state plane 1702 ft)	85636.303
GENESIS Easting Origin (state plane 1702 ft)	1134358.277
Shoreline Rotation	147°
GENESIS version	CEDAS 2.01g

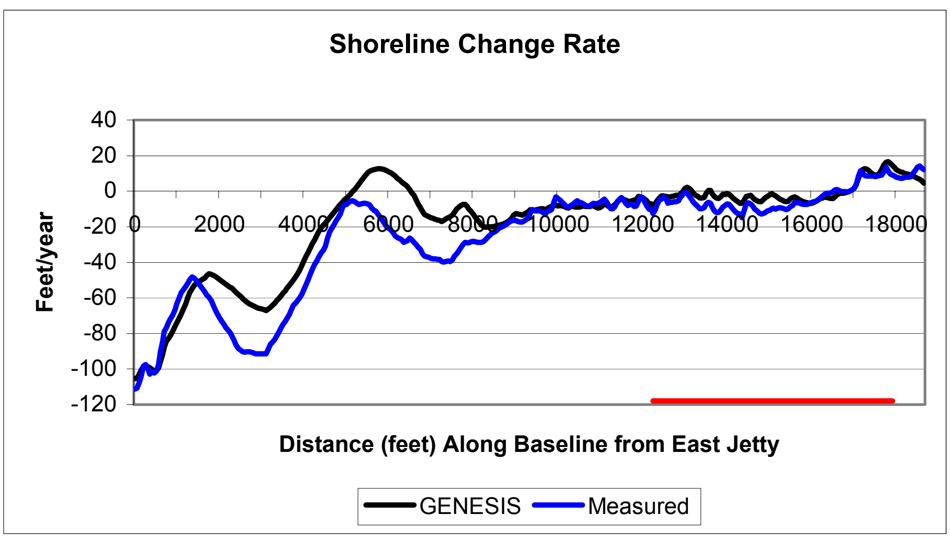

Figure 4-5. Comparison of 1996-2002 measured shoreline change rate along eastern portion of Grand Isle with GENESIS prediction

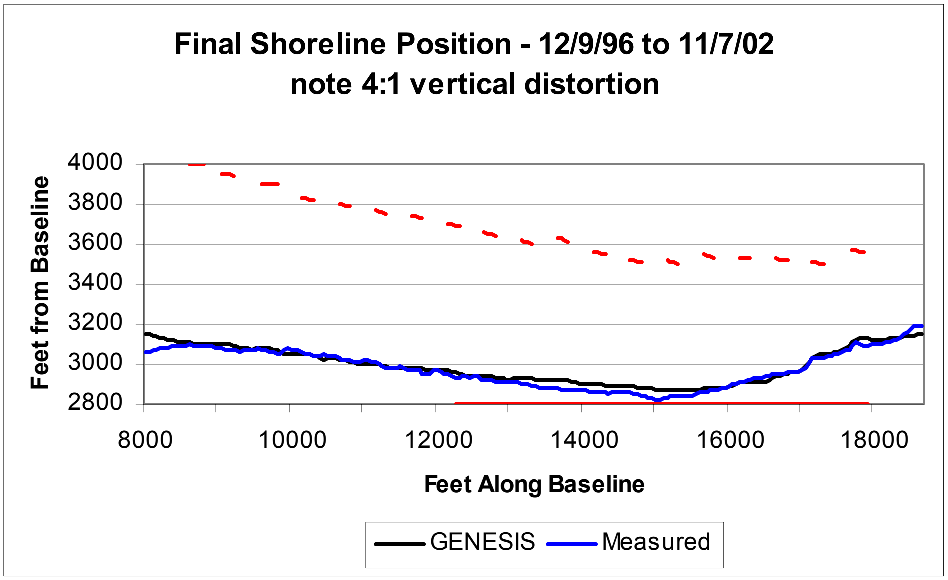

Figure 4-6. Comparison of 2002 measured shoreline position with calibrated GENESIS prediction; detail of western portion of GENESIS grid that includes the area of critical concern

Verification

Verification of the model could not be made during the time period when the detached breakwaters were in place because additional suitable measured shorelines were not available. Therefore, verification was made over a time period prior to their installation. The time period 12/15/1985 to 1/17/1989 was chosen from the available shorelines, because this time period avoided major man-made alterations to the beach except for the placement of the fill in 1984 and 1985. There were several significant tropical storms that impacted the area prior to the 1985 survey. Their impact on the verification result is uncertain. The hindcast wave conditions for this period of time were used as the wave forcing.

Figure 4-7 shows a comparison of measured and predicted shoreline change rates and Figure 4-8 shows a detailed comparison of the final shorelines over the western portion of the grid. In general, verification agreement over the eastern portion of the GENESIS grid is quite poor, but is much better over the western portion of the grid where the critical area is located. The pattern of shoreline change in the vicinity of the critical area is consistent with that for the calibration period. In the eastern portion of the domain, near the inlet and within the ebb shoal, the pattern of shoreline change during the verification period is completely different from that seen during the calibration period. The reasons for the discrepancy are unknown, but are believed to reflect inlet processes not simulated by the model with wave forcing alone. Nevertheless, the model was judged to have reasonable predictive skill in the critical area, and it was judged to be useful for examining alongshore spreading of a beach fill placed in this region.

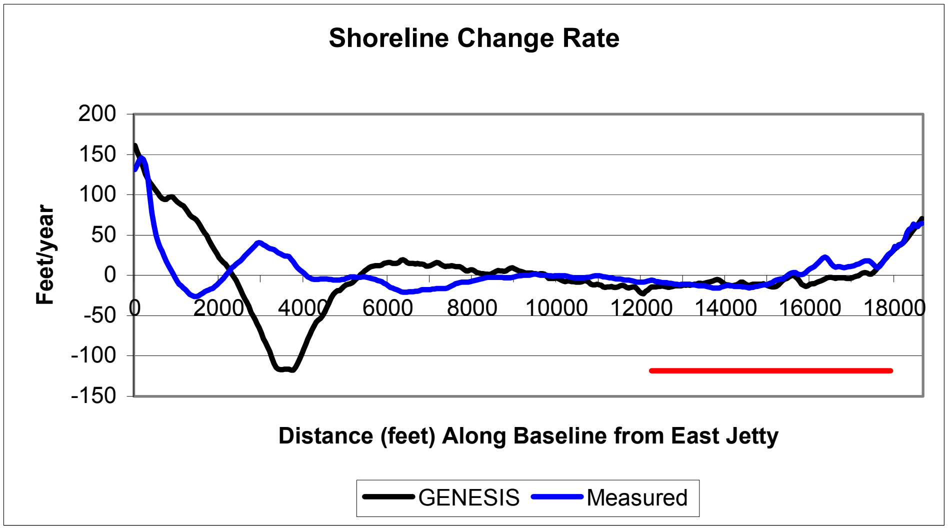

Figure 4-7. Comparison of 12/15/85-1/17/89 measured shoreline change rate along eastern portion of Grand Isle with GENESIS prediction

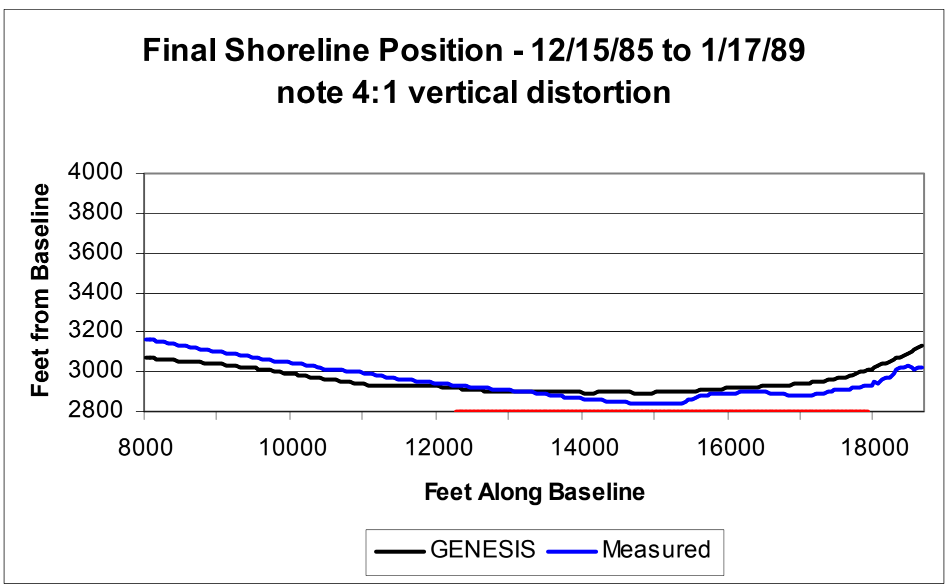

Figure 4-8. Comparison of 1989 measured shoreline position with calibrated GENESIS prediction; detail of western portion of GENESIS grid that includes area of critical concern

Longshore transport rates

The calculated net longshore transport rate is to the northeast along the Grand Isle shoreline. This is clearly seen in Figures 4-9 and 4-10. Figure 4-9 shows impoundment on the west side of numerous wooden groins (photograph taken in the 1950s). It is interesting to note that back in the 1950's the critical area was an area of concern that required shore protection measures at that time, as well. Figure 4-10 shows the 1,200 ft of seaward accretion that occurred next to the jetty, which was built in 1958. In Chapter 2, the net transport rate past the Town Rock Project was estimated to be 63,000 cu yd/year to the east. "CERC formula" calculations using the WIS offshore wave data discussed above indicate a whole-island average net transport rate of 90,000 cu yd/year to the east. Gravens and Rosati (1994) estimated the transport rate in the area east of the Town Rock Project to be 60,000 to 75,000 cu yd/year to the east.

Figure 4-9. 1950s (exact date unknown) Grand Isle aerial photograph of wooden groin field looking westward in area that is today of critical concern. Photograph by U.S. Army Corps of Engineers

The GENESIS predicted net transport rate prior to the construction of the breakwaters (12/15/1985 to 1/17/1989) is shown in Figure 4-11. These calculations indicate a net transport rate of 62,000 cu yd/year to the east at the western end of the grid (adjacent to the Town Rock Project) and transport rates behind what would become the Federal breakwaters of between 35,000 and 85,000 cu yd/year with an average of 61,000 cu yd/year to the east. The GENESIS transport rate calculations after breakwater construction (12/9/1996 to 11/7/2002) are shown in Figure 4-12. They indicate net transport rates (in cubic yards per year to the east) of 60,000 at the Town Rock Project, and a slightly lower post-construction average of 55,000 behind the Federal breakwaters. The range remained essentially the same as with pre-construction. The consistency of results from various sources is satisfying and matches the morphologic evidence.

Figure 4-10. Aerial photograph of the east end of Grand Isle, LA, and Barataria Pass taken in March 1968. Photograph by U.S. Army Corps of Engineers

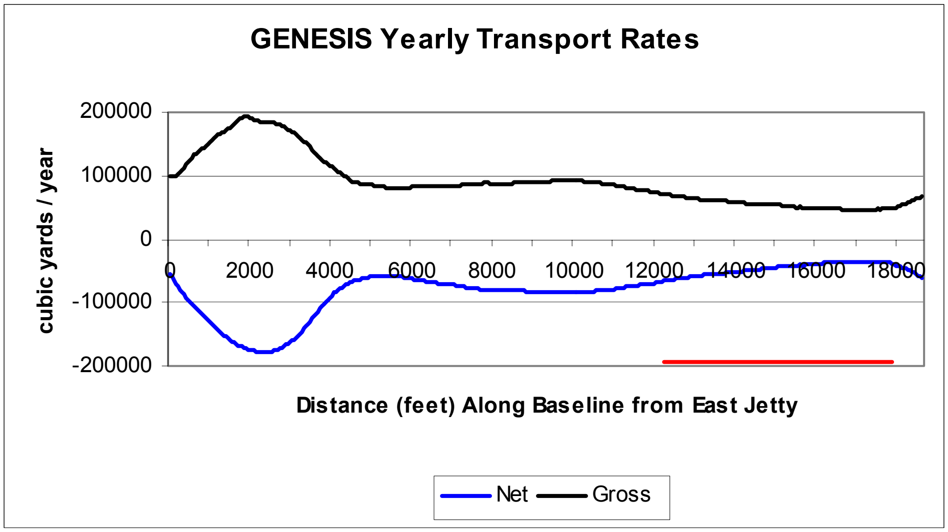

Figure 4-11. GENESIS calculated average gross and net yearly transport rates along the eastern half of Grand Isle between December 1985 and January 1989. Negative net values indicate eastward transport

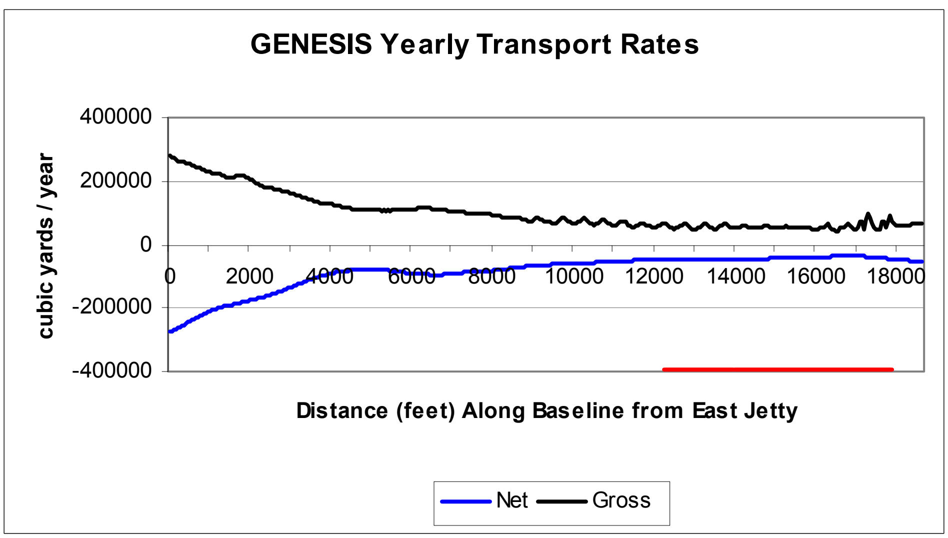

Figure 4-12. GENESIS calculated average gross and net yearly longshore transport rates along eastern half of Grand Isle between December 1996 and November 2002. Negative net values indicate eastward transport

Discussion of Breakwaters

The 23 western detached breakwaters (those on the right hand side of Figure 4-4) were constructed by the U.S. Army Corps of Engineers in early 1996, and are referred to in this report as the Federal breakwaters. The 13 eastern breakwaters were constructed by the Louisiana Department of Transportation in 1999, and are referred to in this report as the LADOT breakwaters. When individual breakwaters are discussed in this chapter, they are numbered from west to east (from right to left in Figure 4-4) with Federal breakwater 1 starting at grid cell 716, and LADOT breakwater 36 ending at cell 31.

LADOT breakwaters

The Barataria ebb shoal is a massive feature (compared to most other inlet ebb shoals around the United States) that ties into the beach in the region of the gap between the Federal and state detached breakwaters (Figures 4-2 and 4-4). As a rule of thumb, inlet ebb shoals form in the area where tidal forces and wave forces are approximately equal in their capacity to move beach sediments. Thus, the LADOT breakwaters can generally be considered to be inside the inlet, and the Federal breakwaters, outside the inlet.

A typical detached breakwater retards the longshore transport of sand by reducing wave heights and causing wave sheltering, diffraction and refraction in its lee. However, because of the position of the LADOT breakwaters relative to the inlet, the longshore transport on the adjacent beach may be greatly influenced by tide-generated currents in addition to wave-generated currents. Thus, while these breakwaters do function to reduce the wave climate in their lee, they probably have little effect on the tidal currents through the inlet. This is shown by the fact that salients do not appear to form behind them.

Before any modifications to the LADOT breakwaters are considered, it is recommended that a field data collection program be undertaken to measure the alongshore currents behind these structures along with other relevant parameters (waves and wind), and use the data to quantify the relative roles of tidal and wave-driven currents. A detailed analysis of these data would be necessary to determine the most effective solution to the erosion in this area. Modeling of tidal currents and calculations of sediment transport due to the combined action of waves and currents would also provide valuable information for designing a solution.

Federal breakwaters

The Federal breakwaters were designed to be highly transmissive structures. See HTNB (1993) and Gravens and Rosati (1994). That is, a large percentage of the incident wave energy passes through them. While this means that only small amounts of sand become trapped behind them, it also means that they do not cause major downdrift erosion problems.

Most of the Federal breakwaters appear to be largely operating as designed. GENESIS modeling of the shoreline shows that, depending upon the wave climate, over the course of days to weeks, small salients form and disappear behind various breakwaters. These salients are more evident behind the most westerly of the Federal breakwaters (the ones near the Town Rock Project). Like the LADOT breakwaters, the most easterly of the Federal breakwaters (the ones further offshore and nearer the Barataria ebb shoal) do not appear to form salients.

This is seen in Figures 4-13 and 4-14. These figures show portions of the 1996, 1998, 2002, and 2003 aerial photograph-derived shorelines. (These figures do not have a distorted vertical scale.) Figure 4-13 shows the shorelines behind the most westerly of the Federal breakwaters. Small undulations (salients) are seen in some of the shorelines that have positions and length scales that match that of the breakwaters. These salients do not appear in the shorelines behind the most easterly of the Federal breakwaters (Figure 4-14) or behind the LADOT breakwaters. It is interesting to note that as the shoreline has advanced seaward in response to the increased transport of sand around the TRP, the presence of salients has become more evident. A recent site inspection confirmed the presence of salients formed in the lee of the first few breakwaters.

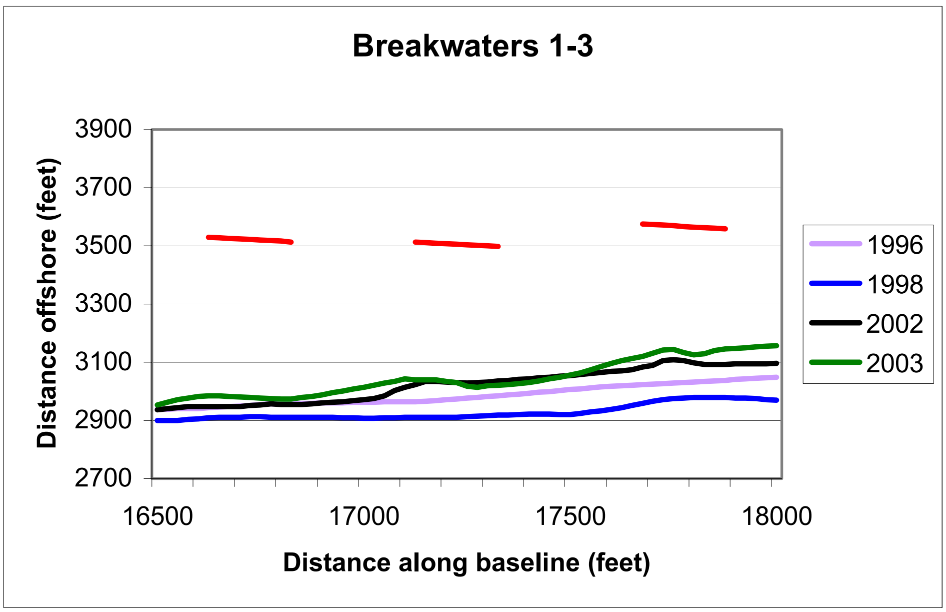

Figure 4-13. Shoreline positions behind breakwaters 1-3

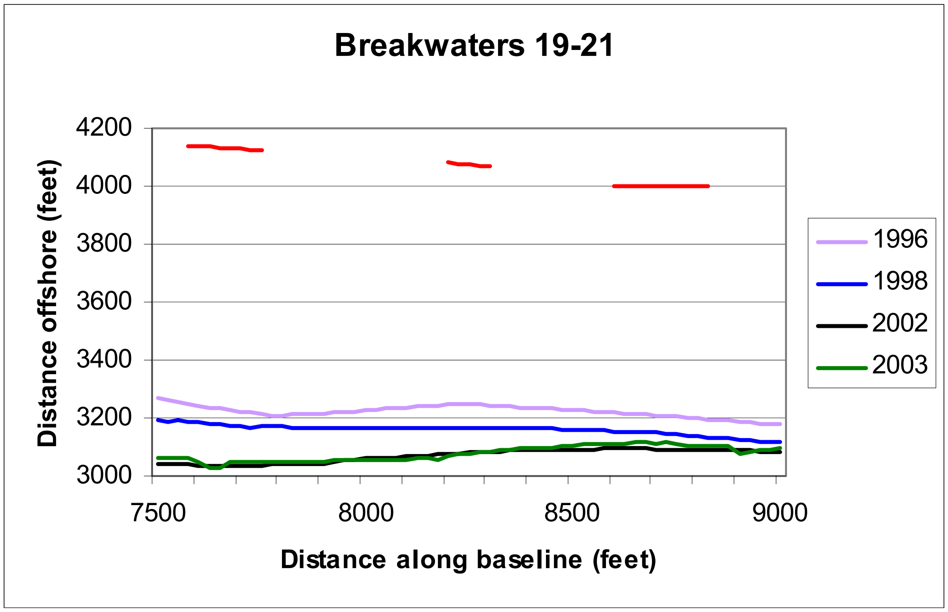

Figure 4-14. Shoreline positions behind breakwaters 19-21

Breakwater transmission coefficients

The HTNB (1993) breakwater design parameters are listed in Table 4-2. A transmission coefficient ($K_{T,}$ is the ratio of the wave height behind the breakwater to the wave height seaward of it) is needed for each breakwater to run GENESIS. A theoretical value of $K_T = 0.9$ was calculated from the values in Table 4-2 using the Ahrens equation (Ahrens 2001; Wamsley and Ahrens 2003). Using values of this order of magnitude, GENESIS results produced salients with amplitudes and spacings that reasonably matched those seen in the aerial photograph shorelines.

Visual inspection of the breakwaters suggests that some are even more transmissive than their design value. The crest height of the breakwater is an important parameter in limiting the amount of wave overtopping that occurs. Figure 4-15 is a photograph of breakwater 5 taken when the tide was below mid-point. In this photograph, the average crest elevation certainly appears to be less than the design value of 4 ft MSL. If so, the breakwater should have a higher K_T value. The K_T values of individual breakwaters were slightly tuned to more closely match the observed shoreline salients and the general shoreline trends. The final K_T calibration results are given in Table 4-3.

Table 4-2 Breakwater Design Parameters	
Parameter	**Value**
Crest Height	4 ft MSL
Design Wave Ht (H)	4 ft
Design Wave Period (T)	5.75 sec
Design Depth (d)	6 ft
Wavelength (L)	77 ft
Distance Offshore (X)	450 ft
Structure Length (Ls)	225 ft
Gap Length (Lg)	300 ft
Crest Width (B)	10 ft
Side Slope Ratio	3:1
Riprap diameter (D50)	8 ft
Stone Size (W)	3500 lbs

Table 4-3
Breakwater Transmission Coefficients

Breakwater	K_T	Breakwater	K_T	Breakwater	K_T	Breakwater	K_T
1 (Federal)	0.85	10 (Federal)	0.90	19 (Federal)	0.99	28 (State)	0.99
2 (Federal)	0.85	11 (Federal)	0.90	20 (Federal)	0.99	29 (State)	0.99
3 (Federal)	0.90	12 (Federal)	0.90	21 (Federal)	0.99	30 (State)	0.99
4 (Federal)	0.90	13 (Federal)	0.90	22 (Federal)	0.99	31 (State)	0.99
5 (Federal)	0.97	14 (Federal)	0.90	23 (Federal)	0.99	32 (State)	0.99
6 (Federal)	0.97	15 (Federal)	0.90	24 (State)	0.99	33 (State)	0.99
7 (Federal)	0.95	16 (Federal)	0.90	25 (State)	0.99	34 (State)	0.99
8 (Federal)	0.95	17 (Federal)	0.90	26 (State)	0.99	35 (State)	0.99
9 (Federal)	0.90	18 (Federal)	0.90	27 (State)	0.99	36 (State)	0.99

Breakwater modifications

GENESIS was used to examine three types of breakwater modifications. The first was to make the breakwaters less transmissive, which could be done to the physical structures by rebuilding them with a core of small stones or (more cheaply) by increasing their crest elevation. Decreasing the transmission coefficients created more pronounced salients, and in some cases, tombolos. The second modification was to lengthen the breakwaters. This generally produced only small changes in shoreline positions and transport rates. The third type of modification was to combine breakwaters to form a smaller number of longer breakwaters. When the breakwater lengths became substantially longer than their distances offshore, the salients became much more permanent in nature, with permanent erosion features at their downdrift end and erosion between salients.

In general, longer breakwaters produced salients of greater seaward extent and downdrift erosion areas of greater landward extent. An example of this is shown in Figure 4-16, which compares GENESIS predictions of the 2002 shoreline for the present set of breakwaters versus what the shoreline would have looked like if breakwaters 1-6, 7-12, and 13-18 had been built in 1996 as three segments.

Figure 4-15. Photograph of breakwater 5 taken 4 November 2003

No type of breakwater modification that was modeled provided substantial benefit to the area of the shoreline of critical concern without causing substantial erosion problems elsewhere. The general conclusion from this study is that periodic beach renourishment is a better alternative to addressing the problems of beach erosion on Grand Isle than breakwater modification.

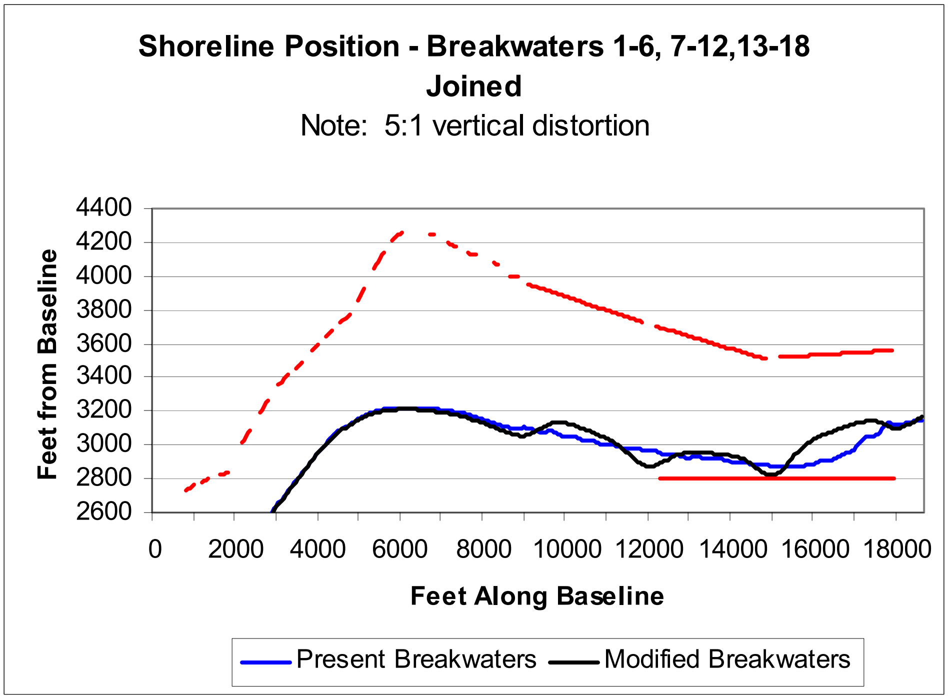

Figure 4-16. Comparison of GENESIS predicted 2002 shorelines with present breakwater configuration versus modified breakwater configuration

Beach Fill Design

Initial fill

Numerous beach fill designs were investigated using the GENESIS model. The criterion was to find the minimum initial fill volume that would maintain the desired beach width over a 4-year renourishment cycle. Thus, the beach fill is designed to withstand the effects of typical wave conditions for 4 years, and then still provide protection for the types of storm events modeled in Chapter 3.

The 6,000-ft long beach fill selected is given in Table 4-4. This fill will extend the shoreline 225 ft seaward of the present shoreline (the July 2003 shoreline was used for design) between Grand Isle baseline stations 214+00 and 221+00 and 125 ft seaward between sta 232+00 and 250+00. The fill is tapered on each end and between the 125 and 225-ft sections. The expected behavior of this fill is shown in Figure 4-17. In this figure the initial beach fill shoreline is shown in pink (Table 4-4), the GENESIS prediction of the beach fill shoreline after 4 years in black, the minimum acceptable beach width in orange (derived from Chapter 3), the present shoreline in dark blue, and the present dune line in brown. For clarity, this figure has a 4:1 vertical to horizontal distortion. It is seen that throughout the study area, the 4-year beach fill shoreline (black) does not move landward of the minimal acceptable beach width (orange).

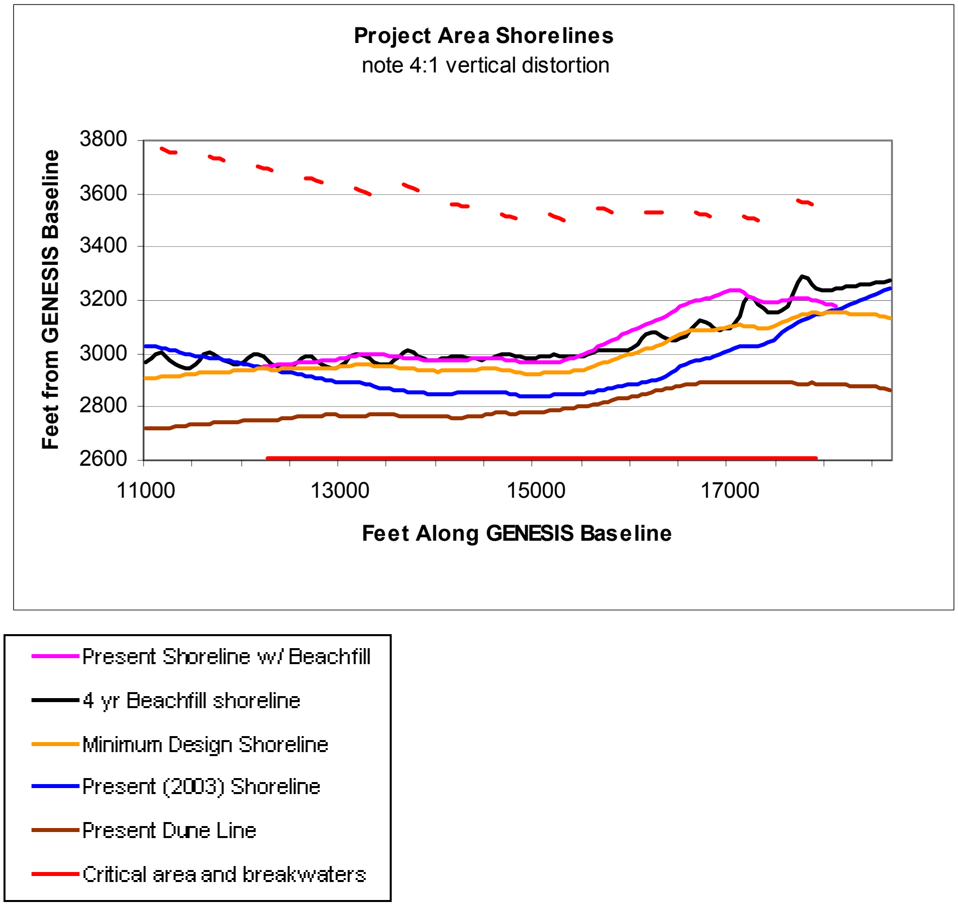

Figure 4-17. Project area shorelines

Table 4-4
Beach Fill Location

Range Line	Breakwater Position	GENESIS Cell	GENESIS Baseline Distance	Seaward Extent of Fill From Present (7/03) Shoreline
202+00	W of 1	729	18200	0
214+00	between 2 & 3	681	17000	225
221+00	W end of 4	653	16300	225
232+00	6	609	15200	125
250+00	near W end of 10	537	13400	125
262+00	12	489	12200	0

Assuming a 6 ft average berm elevation and a 12 ft depth of closure (Chapter 3), the beach fill volume for this design will be 520,000 cu yd. This volume includes both the amount required to create the design cross-section and that needed for advance nourishment, which is designed to retain the design cross-section during the first renourishment interval. Assuming an overfill ratio of 1.3 (derived in Chapter 2), the needed beach fill volume would be 680,000 cu yd. For a further discussion of overfill ratios to determine the quantity of beach fill needed (see Chapter 2).

Performance and renourishment

The model indicates that over a 4-year interval, the beach fill shoreline will retreat back to near the minimum acceptable beach width (Figure 4-17). This eroded material will be spread along the beach to the east and west, providing sand to other areas outside the project area. Thus, the beach will require periodic renourishment on a 4-year cycle. Expected renourishment volumes, given in Table 4-5, are roughly a fourth of the initial fill volume. These renourishment volumes are based upon near-normal wave activity over the 4-year interval. Major storms impacting Grand Isle would be expected to significantly increase these renourishment volumes above those given in Table 4-5 and may need to be dealt with on an emergency basis.

Dune volumes

The above volume estimates do not include changes to the present dune line. Results from SBEACH (Chapter 3) showed that a dune having a crest elevation of +12 ft NGVD, crest width of 10 ft, and 1:5 vertical to horizontal side slopes withstood the historical storms that have occurred since 1985 if adequate beach width was present. The average additional volume needed to increase the dunes to these dimensions is 5.7 cu yd/ft or 35,000 cu yd for the beach fill project area. This volume would be in addition the volume discussed, increasing the total initial volume required to 715,000 cu yd.

Table 4-5
Renourishment Volumes

Yearly Wave Statistics	No Overfill	Overfill Ratio 1.3
1 St Dev above average	160,000 cu yd/year	210,000 cu yd/year
Average	130,000 cu yd/year	170,000 cu yd/year
1 St Dev below average	100,000 cu yd/year	130,000 cu yd/year

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5 Summary, Conclusions, and Recommendations

A shore protection history for Grand Isle was compiled. Available aerial photographs, shoreline position, bathymetric survey, beach profile, grain size, and other data were acquired and processed, carefully examined for quality, and then analyzed to support this study. Spatial data were geo-referenced to a consistent set of datum controls; vertical (NAVD 1988), and horizontal (Louisiana State Plane South, 1702, NAD 1983). Certain spatial data sets (aerial photos, bathymetric survey and shorelines) were integrated into an ArcMap geographic information system (GIS) to support geomorphic analyses and interpretation. Beach profile data were organized into the Beach Morphology Analysis package (BMAP) to support analyses of this special type of spatial "transect" data. Processed and analyzed data were also used to support set-up and application of numerical models to calculate storm-induced erosion (using the SBEACH model) and shoreline planform evolution (using the GENESIS model). These models were also used to re-evaluate the project design. Wave climate information required for the modeling was derived from available measurements, and from a special USACE Wave Information Study hindcast wave climate used as input to the STWAVE nearshore wave transformation model. Water level data were acquired from existing measurements.

Overall Assessment of the Project's Current Condition

The condition of the 2003 Grand Isle shoreline was evaluated against the design beach template from the General Design Memorandum (GDM) for Grand Isle (USACE 1979). It should be noted that the GDM design template would represent a best-case scenario for shoreline position, as post-nourishment readjustment of the fill would reduce the design beach width. The originally constructed beach would have adjusted substantially following placement, resulting in much less added dry beach width than indicated in the original design profiles.

Even assuming this best-case scenario, overall, the analysis showed the island to be in good condition compared to the GDM design. However, four areas did not meet the design template and they deserve closer inspection. Area 1 is adjacent to the west-end terminal jetty, and is likely induced by the influence of the Caminada ebb shoal and the degraded condition of the structure. Even here, recent visual observation indicates the beach fronting the dune seems rather healthy. Area 2 begins approximately at range 208+00 and ends at range 261+00. This area defines the extent of the critical problem area for this project, and needs immediate attention. Area 3 is located in the vicinity of the State Park between ranges 320+00 and 330+00. While the beach width fronting the dune is rather healthy, this local erosion hot-spot is interfering with park usage. This hotspot appears to be induced by wave sheltering, diffraction and refraction processes where the detached breakwater field changes orientation. Area 4 is located just east of Area 3, extending from range 340+00 to 370+00. This area is experiencing significant erosion and is of concern.

Influence of Coastal Structures on the Alongshore Movement of Sand

Regional longshore sand transport regime

In order to understand the influence of the various coastal structures, which have been built along the Gulf coast of Grand Isle, on adjacent beaches, knowledge of the regional longshore sand transport regime is needed. The net long-term rate and direction of net longshore sand movement are critical coastal process parameters. Information about them is also needed to design and maintain a sound engineering solution to erosion problems that have been and are being experienced at Grand Isle. Sand moves in both directions along the island, as evidenced by impoundment on the east side of the west-end terminal groin and impoundment on the west side of the east-end terminal groin. The transport magnitude and direction can change with changing event-scale, seasonal and annual shifts in wave climate. Local erosion can occur in response to spatial variations in the longshore net sand transport rate.

At Grand Isle, the long-term net direction of longshore sand transport is from southwest to northeast, and with a variable rate depending on position along the island. This direction is determined based on both physical evidence and engineering computations. The following physical evidence from geomorphic analyses supports this net sand transport direction: the historical morphologic evolution of Grand Isle itself which has rotated between 1884 and 1945, accreting on the northeast end and eroding on the southwest end; the rapid rate of accretion that has been observed at the east-end terminal groin between 1945 and 1985 (also believed to be due in part to the evolution of the ebb shoal); evidence of re-curved sand spits on the northeast end of the island; the degree of sand accumulation on the west side of the terminal groin at the eastern end of the island, compared to the rather minor accumulation that has occurred on the eastern side of the terminal groin at the west end of the island; extensive impoundment of sand that has occurred west of the Town Rock Project (TRP) since its construction, and the accompanying erosion pattern that has been experienced east of the TRP; and the extensive sand spit that has formed on the western interior side of Caminada Pass. All these physical features point to a net longshore sand transport direction from southwest to northeast. Also, as part of this study, longshore sand transport rates were calculated based on the hindcast wave climate and nearshore wave transformation modeling. Calculated transport rates also indicate a net transport direction to the northeast, which is consistent with the physical morphologic change evidence.

Under this regional longshore sand transport regime, the origin of sand on Grand Isle must be from sources to the west, most likely the Caminada-Moreau Headland. The sand fraction of sediment eroded from the headland is transported within the surf zone to beaches to the west, where sand enters Caminada Pass and ends up in the interior spit mentioned above and in the ebb and flood shoal complexes. Some sand is transported around the Caminada ebb shoal and onto Grand Isle. The relatively small size of the Caminada ebb shoal, the shallow depths along the ebb bar, and the lack of a deep channel through the pass all seem conducive for natural bypassing of sand to occur across the Pass. The Caminada-Moreau Headland is a major long-term source of natural beach nourishment for Grand Isle.

In a natural state, without the presence of coastal structures and in a net sense, sand that moves to the northeast along Grand Isle would eventually enter Barataria Pass and end up in the shoal complexes of that pass. The deepness of the channel through this pass, and the deeper depths of the outer ebb bar suggest that sand bypasses Barataria Pass to Grand Terre much less easily than it bypasses Caminada Pass onto Grand Isle.

Town's Rock Project

Initially, construction of the Town's Rock Project, which was comprised of two rather long groins and several detached breakwaters, reduced the alongshore movement of sand, or blocked it completely. Since its construction, sand has accumulated between the groins and to the west of the Project. But the accretion has come at the expense of beaches to the east. In an area with a well-defined net direction of longshore sand transport, and in the absence of placing new sand, accumulation on one side of a groin structure (the updrift side) will be accompanied by erosion on the other side (the downdrift side), purely from the standpoint of continuity. In light of the southwest-to-northeast net transport direction along Grand Isle, at the TRP, the updrift side is to the west and the downdrift side is to the east. The positive aspect of the TRP has been its stabilizing influence on beaches in the central portion of the island west of it. However, as long as the Project reduces the longshore net transport of sand, it causes erosional pressure on the downdrift beaches, i.e., those beaches to the east of it. Initially, it appears that the blockage of longshore transport was nearly complete and erosional pressure on downdrift beaches was at its peak. Presently, the compartments behind the breakwaters and between the groins are completely filled, the detached breakwaters are virtually land-locked, there has been extensive accumulation west of the project, and sand now appears to bypassing the project, at a rate calculated to be approximately 63,000 cu yd/year during the recent era.

The critical erosion area is the area that has been most impacted by the obstruction of alongshore sand movement by the Project. Placement of sand in the critical area is needed to offset the sand deficit that has been created by the Project and by natural processes. It is important to note that this area has been reported to be an area of locally higher than normal erosion even before the Project was constructed (discussed in the next section). Once this sand deficit is made up, and the bypassing rate past the Project begins to approach pre-project levels, and sand is periodically added to the critical area as needed to make up the difference between the actual and natural bypassing rates, then the Project will become a completely positive factor in stabilizing the central portion of Grand Isle. Until the sand deficit is made up, the area of critical erosion can be expected to migrate slowly to the east, as the amount of sand that bypasses around the Project increases. The optimal engineering approach is to take advantage of the future positive impact of the Town Rock Project, and mitigate any negative impact through beach nourishment.

Federal breakwaters

As noted above, the area in the general vicinity of, and just to the east of, the Town Rock Project has been an area of higher-than-normal erosion for a long time, even before construction of the Town Rock Project. Results of wave transformation modeling performed as part of this study suggest that this erosion "hot-spot" may be due to the influence of the ebb shoal on local wave refraction and the resulting alongshore sand transport pattern. The Barataria ebb shoal is massive, and the southeasterly-oriented ebb shoal bathymetric contours merge with the generally shore-parallel bathymetric contours along central Grand Isle in this vicinity. It appears that the changing offshore bathymetric contour orientation produces a persistent alongshore variation in wave characteristics, which leads to an alongshore gradient in the longshore sand transport rate, producing a local erosion zone, or hot-spot.

In response to the persistence of the highly erosive area east of the Town Rock Project, the loss of protective dunes in certain places and intermittent wave overtopping, localized flooding, and sand overwash during storms, a series of segmented detached breakwaters were constructed by the Corps. The breakwaters were intended to slow the rate of longshore sand transport, reduce incident wave conditions that reach the shoreline, and stabilize the beaches east of the Town Rock Project. But in the absence of newly placed sand, the breakwaters can only partially solve the erosion problem; they can only reduce the rate at which erosion is occurring. The critical erosion area suffers from a sand deficit, and the deficit must be made up through beach nourishment.

Once the sand deficit is made up in the critical area, then any positive influence of the detached breakwaters can be realized, much like the benefit of the Town Rock Project. Based on an assessment of breakwater transmission characteristics, shoreline change data and shoreline model results, the detached breakwaters only slightly reduce the rate at which sand is removed from the beach behind them. The breakwaters were designed to be transmissive, and they are. Periodic renourishment of a fill placed behind the breakwaters probably will be needed.

State (LADOT) breakwaters

In response to erosional pressure experienced at the east end of the island and impacts to park usage, the Louisiana DOT constructed a series of breakwaters adjacent to the Federal breakwaters, extending the same general shore protection concept to the east. The eastern end of the island also has suffered the consequences of obstructions to the alongshore movement of sand. The Town Rock Project initially blocked sand that would have eventually moved to the eastern end of the island. The poor condition of the east-end terminal groin has also contributed to the loss of sand from the east end of the island. The capability of the terminal groin to retain sand has been severely degraded. The Federal breakwaters also appear to be highly transmissive and not slowing the rate at which sand reaches the east end of the island. The State breakwaters perform in a similar manner to the Federal Breakwaters; they don't eliminate the erosion problem; they only slightly slow the rate of erosion from behind the breakwaters. As is the case with the Federal breakwaters, beach fill is needed to make up any erosion-induced deficit; then the breakwaters can serve any positive role for which they were intended.

The increasing role of tidal currents in transporting sand along the beach probably also increases the rate of alongshore movement with increasing proximity to Barataria Pass, although that process was not considered in this study. This gradient in longshore transport rate, increasing east-bound transport rate from west to east behind the breakwaters, also suggests an erosional tendency. While the situation is not ideal, the issue of decreasing breakwater effectiveness can be addressed through periodically renourishing the beaches behind the breakwaters and placing sand in differing amounts where it is needed to mitigate any differences in loss rate. The east-end terminal groin should be sand-tightened and perhaps lengthened; and if this is done, the shoreline west of the terminal groin will advance seaward, thereby decreasing the distance from shoreline to detached breakwaters. A decreased distance offshore will render the breakwaters more effective in reducing the alongshore movement of sand, and helping to mitigate the erosional tendency behind the breakwaters.

There also appears to be localized erosion, a hot-spot, where the gap in the breakwater field is unusually large relative to adjacent breakwater gaps and the breakwaters change orientation. Where the gap is abnormally large, more wave energy reaches the beach causing a local longshore sand transport rate that is higher than the rate along adjacent beaches to either side. In response to the local gradient in transport rate (more sand leaving a local region than is entering it), the beach will experience a local erosion hot-spot. Adjacent to (downdrift of) the erosion hot-spot, more sand is entering the region from the erosion hot-spot than is leaving it, and a corresponding area of accumulation results. This pattern is apparent at the west end of the State breakwater field, between sta 320+00 and 330+00.

Stabilizing the Project Area

Beach fill placement in the critical area

The critical erosion area is defined as a stretch of beach between sta 208+00 and 261+00. Since original project construction, several attempts have been made to stabilize the critical area and the eastern end of the island, with structural measures in lieu of beach re-nourishment. The structures have not completely produced the intended result, primarily because no sand was placed in conjunction with them. Beach fill is needed in the critical area to offset the volume deficit that has arisen from the cumulative effect of interruptions to the alongshore flow of sand due to the various coastal structures. Periodic renourishment will be required in the future to maintain the initial beach fill, thus providing long-term protection against storms. With placement of the beach fill, the full value of the detached breakwaters and the Town Rock Project will be realized; and their presence will reduce the long-term re-nourishment requirement for the project, compared to what it would be if the structures were not in place.

Analyses of measured beach profile data provide valuable information about how a beach fill should be designed. Profile data from areas where the beach is healthy indicates that the natural berm elevation on Grand Isle is about +5 ft MSL, or about +6 ft NAVD 88. Analysis of the same data shows that the natural beach slope from the berm crest elevation of +6 ft NAVD 88 down to MSL is about 1:30. The theoretical equilibrium profile for a median grain size of 0.13 mm fits the measured beach profile shape very well in the critical area, over the depth range of 0 to -8 ft NAVD 88. This implied median grain diameter, 0.13 mm, is consistent with median diameter derived from sieve analyses of sediment samples collected along the beach and nearshore zone. Generally samples along the shoreline have median grain diameters that are slightly greater than 0.13 mm (0.15 to 0.17 mm), and samples collected from deeper nearshore waters out to 12 ft were slightly finer, with median grain diameters of about 0.09 or 0.1 mm. Borrow area sand having a median grain diameter of 0.13 mm was defined as being the most compatible with the native beach. Sediments having median grain diameter less than 0.1 mm should be avoided if at all possible as sources for beach nourishment at Grand Isle. Based on analysis of profile data collected between 1992 and 1996, a short-term "depth of closure" was identified as -9 ft NAVD 88. Based up on a comparison of these data with comparable profiles collected in 2003, a closure depth of -12 ft NAVD 88 was identified. An active depth of 18 ft was adopted for all shoreline and beach change modeling, and in computations of beach fill volume. This value was derived assuming a berm crest elevation of +6 ft and closure depth of -12 ft NAVD 88.

Storm erosion modeling work indicates that the dry beach berm and the entire active profile including the subaqueous beach down to depth of closure, must be advanced in the critical area by as much as 125 ft to provide protection against the types of hurricanes and tropical storms that have occurred since the project was originally constructed in 1985. Hurricanes Juan, Andrew, Danny, Isadore/Lili were the most severe storms between 1985 and 2003. Based solely on the peak water levels associated with these storms, and the stage-frequency curve contained in the original GDM, these four events are the types of events that are expected every 5 to 7 years. Storm erosion modeling suggests that a more sizable dune (one having a 20-ft-wide top crest width and a crest elevation of 14.3 ft NAVD 88, and side slopes of 1:5) is required to withstand the effects of the hypothetical design storm defined in the original GDM. It is important to point out that the GDM beach fill design, and the beach fill proposed here, will not protect against all storms, and damage during the life of the project should be expected.

Restoration of the project in the critical area will require the dune to be restored to design specifications (10 ft top crest width, crest elevation of 12.3 ft NAVD88, and 1:5 side slopes). Also the beach berm at the +6 ft NAVD 88 elevation, and the entire active profile seaward of the berm down to the depth of closure -12 ft NAVD88, will have to be advanced by amounts listed in Chapter 4 (Table 4-4). This level of beach advancement includes both the volume required to create the design cross-section, which will provide protection from the types of storms that have occurred since 1985, and the advance renourishment volume (which is equal to the planned periodic renourishment volume). The periodic renourishment volume is placed initially, in addition to the design beach volume, in an effort to ensure the design beach cross-section remains intact during the complete interval between initial construction and the first scheduled renourishment.

Sediments in the East and West Barataria ebb shoal borrow areas are similar, and use of either implies an overfill ratio of 1.3 (due to the fact that the borrow area sediments are finer than the beach sediments). Assuming this overfill ratio, the required initial construction volume estimated for this fill is 715,000 cu yd (680,000 cu yd for the storm protection berm and 35,000 cu yd for the dune reconstruction). The volume of 680,000 cu yd includes the design beach berm cross-section plus the advance nourishment volume. Our estimate of the periodic renourishment volume is, on average, 170,000 cu yd every 4 years.

To define the construction template, i.e., the target beach cross section for the contractor to construct, we recommend that the elevation of the beach berm is held constant at 6 ft NAVD 88 from the gulf-side base of the dune to the seaward terminus of the berm crest associated with the design berm and advance nourishment (this distance will vary within the critical area). Then from this point seaward, the berm elevation should be tapered from an elevation of 6 ft NAVD 88 to an elevation of 5 ft NAVD 88 at the seaward terminus of the constructed berm crest. This will minimize the degree of berm scarping that is expected during the initial beach adjustment process. Seaward of this point, natural slopes will be formed in response to wave and tide action. The natural slope of the beach face is about 1:30, and this slope should be the target slope for this zone of the placement operation.

It is important to communicate to the local sponsor(s) the process of beach adjustment following construction. During construction, sediment will be “stacked” into the nearshore zone, and the constructed beach slopes will be unnatural and in a relative state of disequilibrium. Wave action will cause the beach to adjust and the sediment will gradually move offshore to form more natural slopes. As this happens, beach widths will be reduced dramatically and this is to be expected. The beach width at construction is much greater than the design beach width plus any width associated with advance renourishment. As the project is exposed to increasingly more severe storms the fill material will adjust and move further seaward into deeper water. The beach fill design proposed here assumes the fill material will eventually readjust down to a depth of about -12 ft NAVD88. Very fine material, for example silts and very fine sands having median diameters of less than 0.09 mm, are expected to gradually move further offshore, to even greater depths.

Terminal groin rehabilitation

Terminal groins at both ends of the island are in need of repair, in order to improve their effectiveness in trapping sand. The terminal groins are designed to stabilize the island at its ends, but due to their degraded condition, their effectiveness has been compromised.

The east-end terminal groin, in particular, is in very poor condition. It is very low in places, including a very large gap at the shoreline where much sand moves alongshore, past it. Presently the east end of the island is experiencing very high erosion rates, and the poor condition of the east terminal groin is a contributor. Rehabilitation of the eastern terminal groin is recommended to improve its sand retention capability, and to help stabilize this highly eroding end of the island. Lengthening of the eastern terminal groin should also be examined to improve sand trapping and its stabilizing effect on the State Park area. Periodic extension of the east end terminal groin is consistent with the long-term growth trend for this end of the island, and should be expected over the life of the project.

An effective east-end terminal groin also provides a good method for trapping high quality beach sand that can be used for scheduled or emergency beach fill maintenance in the future, and for mitigation of local erosion hot-spots associated with the non-uniform gap spacing or other causes. Impoundment of high quality beach sand at the terminal groin prevents, or at least reduces, the likelihood that the sand will enter the inlet. Once it does, it becomes much more widely dispersed, mixing with finer, less suitable sediment. The mixing process effectively increases the cost to dredge a unit of high-quality beach sand; and makes identification of, and accessibility to, the higher quality sand less certain. In coastal Louisiana, where beach quality sand is an extremely valuable resource, it seems prudent to try and retain beach quality sand in a location and in a way that maximizes its availability for effective use in stabilizing beaches.

The west-end terminal groin is also in need of some maintenance, requiring restoration of the crest elevation and eliminating low spots to improve sand retention. The "leakiness" of the groin is evidenced by the low sunken spots in the beach immediate adjacent to the groin on its east side. The west end groin does not require lengthening.

Western half of island

West of the Town Rock Project, all the way to the west-end terminal groin, beaches are generally quite healthy and wide, with highly vegetated dunes, so little attention was given to them in this study. Along the western half of the island, there appears to be at least the 75-100 ft of dry beach berm in front of the dune that is required to provide protection from the types of storms that have occurred since the project was originally constructed in 1985.

Shoreline changes between 1985 and 2003 show the area to be accreting, although some of the apparent accretion maybe due to post-storm recovery from storms Danny, Elena, Juan, and Kate in 1985. Shoreline and volume changes between 1993 and 2003 indicate that this area has been generally stable. The only areas of erosion appear to be immediately at the west end of the island and in the area where the large shoreline salient has disappeared. Survey data indicate the dredged holes that induced the original salient to form have filled in and, as they have done so, the salient has disappeared; presumably since the causative factor, the dredged hole, has also disappeared. The 2003 shoreline shows no evidence of the salient. All sand from the salient has been redistributed to adjacent beaches.

In most places the beach is much wider that would be expected based on the amount of fill placed as part of the initial construction. It seems that the Town's Rock Project has trapped a considerable amount of sand and has been quite effective in stabilizing the shoreline west of the Project. This benefit is expected to continue.

The Town's maintenance practice of scraping beach debris and piling it at the foot of the dune to promote dune growth has worked well and should be continued.

We recommend that periodic monitoring though beach surveying be performed along the entire project domain, including the western half of the island, to assess whether or not beaches retain the design cross-sectional volume, width, and dune characteristics. It is likely that additional renourishment will be required sometime in the future on the west half of the project domain, but at the present time, no renourishment appears to be necessary.

The flux of sand to the Grand Isle littoral cell, across Caminada Pass, is estimated to be approximately 83,000 cu yd/year at present. But recent changes in measured erosion rates at Caminada-Moreau Headland, may have implications on future stability and project performance on Grand Isle. Published work by Penland, Conner, and Beall (2004) concludes that the recent erosion rate of the Caminada-Moreau Headland, -9 ft/year calculated for the period 1988 to 2002, has decreased dramatically compared to historic rates, -31 to -52 ft/year during the period 1884 to 1988. This suggests that the future rate at which sand is supplied to Grand Isle may be significantly reduced, leading to erosional pressure on the island, including the western half. For this reason long-term monitoring should be performed along the entire extent of the island, and particular attention should be given to developing a strategy for long-term maintenance of the project.

State Park region

Rehabilitation of the east-end terminal groin should be done as soon as possible to address the loss of sand from Grand Isle beaches and the high degree of erosion being experienced at the east end of the island. Lengthening should also be considered and studied to assess the effectiveness of this action. Following placement of beach fill sediment in the critical area, the sand should begin to move toward the east in a net sense through the breakwater field, and eventually be impounded at the rehabilitated east-end terminal groin. If the east-end groin is not repaired, then much of this sand will be lost into the inlet shoal system. The presence of the Federal and State breakwaters will slightly slow eastward sand movement from the critical area, so local hot-spots are probably best addressed over the short-term with introduction of new sand. As the critical area loses sand, and is periodically renourished, and as sand continues to bypass the Town Rock Project, an increased supply of sand should reach the east end, helping to mitigate any local hot-spots and the general erosion trend, again provided the east-end terminal groin is rehabilitated.

There may be aspects of ebb shoal evolution and tidal currents that contribute to the erosion problem in the State Park area, but they are more difficult to address. This region is heavily influenced by the massive ebb shoal and probably by tidal currents as well. Modeling work done here did not account for tidal currents, the focus was on the critical erosion area east of the TRP. Application of GENESIS-T (which includes a tidal current capability), a tidal circulation model to simulate tidal currents off Grand Isle, and sediment transport calculations or modeling on the ebb tidal delta are recommended to examine the effectiveness of different engineering solutions in this area. In the absence of further study, terminal groin rehabilitation and sand fill should be pursued first, then the situation should be re-evaluated to assess the degree of the problem that remains after these steps are taken. Concurrently, lengthening the terminal groin should be studied. As stated previously, lengthening the terminal groin would be one means for advancing the shoreline closer the detached breakwaters, making them more effective, which would improve stabilization of the island's east end, and provide a source of high quality sand for future beach maintenance activities.

Effectiveness of breakwater modifications

The detached breakwaters were designed to be transmissive, and they appear to be functioning that way, providing little benefit to retention of sand behind them. The storm erosion modeling suggested that they have minimal benefit in stemming dune erosion during high wave and water level conditions. The GENESIS model was used to examine three types of breakwater modifications. The first was to make the breakwaters less transmissive, which could be done to the physical structures by rebuilding them with a core of small stone or (more cheaply) by increasing their crest elevation. Decreasing the transmission coefficients created more pronounced salients, and in some cases, tombolos. The second modification was to lengthen each of the breakwaters. This generally produced only small changes in shoreline positions and transport rates because the structures are so transmissive. The third type of modification was to combine breakwaters to form a smaller number of much longer breakwaters. When the breakwater lengths became substantially longer than their distances offshore, the salients became much more permanent in nature, with permanent erosion features at their downdrift end and erosion between salients. This would be undesirable in the critical area. In general, longer breakwaters produced salients of greater seaward extent and downdrift erosion areas of greater landward extent. The local erosion effect of modified breakwaters could be mitigated by placing beach fill in conjunction with structural modifications. Instead, we recommend pursuing placement of beach fill to overcome the current sand deficit, and subsequent periodic renourishment. We recommend any structural measures that are taken be focused on sand-tightening and lengthening of the east end terminal groin.

Strategies for long-term beach renourishment

There are a number of strategies than should be considered for long-term renourishment of the Grand Isle beaches. One strategy, that is adopted on many Federal projects, is to locate and use offshore sources of beach-compatible sand. The lack of suitable offshore sand sources in close proximity to the project site, and the high cost associated with sand mining from a distant site, appear to make this a less viable alternative for Grand Isle. The approach of dredging from shallow nearshore regions seaward of the depth of closure, at least in the manner as was done initially to construct the project, should be avoided if at all possible for two reasons: first, the salient formation that can and did accompany nearshore dredging close to shore will cause both local accretion and erosion which may compromise the design cross-section in places, and secondly, the material will generally be much finer that the active beach sand and therefore be much less suitable as a fill material. A much greater volume of fine sand will be required to construct a beach fill, if the borrow source is finer than the native beach sand.

Other strategies might involve bypassing, where sand is take from updrift sources and introduced to Grand Isle. This would seem to be a natural approach, working in concert with natural sand movement and bypassing processes. Possible updrift sources are the Caminada ebb tidal shoal and the large interior spit on the western side of Caminada Pass.

The Caminada ebb shoal appears to be in the likely pathway for sand entering the Grand Isle littoral cell, naturally. Presumably the ebb shoal at Caminada is comprised of beach quality material that originates in the Caminada-Moreau Headland. The exact manner and rate at which sand is bypassed across the ebb shoal to Grand Isle is unclear. The average annual rate for the recent era appears to be about $83,000 cu yd/year, but the relative steadiness or episodic nature of this mode of bypassing is unknown. Borrowing from Caminada ebb shoal might best be done by shaving off the seaward face of the ebb shoal, so as to minimize disruption to the natural bypassing process. Mining from the ebb shoal can alter the rate at which sand might naturally bypass Caminada Pass, but that might be acceptable as long as sand is placed on Grand Isle. A more detailed investigation into the grain size characteristics of the Caminada ebb shoal, and assessment of this site as a potential borrow area, is recommended.

The interior spit on the east side of Caminada Pass would also seem to be an accessible source of beach quality sand. Mining of this interior spit as a potential borrow source should be explored.

Other re-nourishment strategies involve back-passing, where sand from downdrift sources is used for renourishment. This approach is being pursued for placement of fill in the critical area. Two downdrift borrow sources are being considered, one area each on the east and west sides of the Barataria ebb shoal complex. Both sites have similar sediments, although the west site is more homogenous, closer to Grand Isle, and therefore the preferable site from these standpoints. The highest quality beach sediments are most likely to be found in the shallow, most energetic, areas of the ebb shoal. We recommend that mining of the ebb shoal sediments from the existing designated borrow areas be done down to a depth not to exceed 5 or 6 ft lower than ambient depth, in an attempt to capture the highest quality of sand and minimize alterations to wave transformation over the ebb shoal.

Another aspect of a back-passing strategy would involve use of material from the deposit impounded by the east-end terminal groin. In light of the long-term morphologic evolution of Grand Isle, and the direction of net longshore sand transport, use of the impounded material at the terminal groin would seem to provide the best source of beach quality sand, at least for dune repairs and dune construction, and mitigation of local hot-spots. Of course this is probably only possible provided the east end erosion problems are mitigated first, and the beach is stabilized at this point. Borrowing from the impoundment zone would also have to be done in a manner that does not compromise beach stability in the State Park region.

In light of the need for long-term renourishment of the Grand Isle project, and the scarcity of high-quality beach sand, we recommend that all potential borrow sites be thoroughly examined as part of work to develop a long-term strategy for maintaining the Federal project. Because, sand is such a limited and valuable commodity in the region, formulation of such a plan should involve all stakeholders in the region, and be considered in the context of other sediment needs of stakeholders in the region (Grand Terre, for example) and in the context of the existing operations and maintenance programs for nearby channels and harbors.

Future Studies

The eastern end of Grande Isle is strongly influenced by Barataria Pass and the massive ebb tidal shoal. The entrance appears to be ebb dominant, with a considerable volume of water moving through the entrance. For this type of inlet, flows along adjacent beaches are often toward the inlet on both ebb and flood tides, and sediment is transported toward the inlet in a net sense along the adjacent beach. The effect of tidal currents was not considered in this project, because the focus was on a beach fill for the critical area just east of the Town Rock Project. The primary study area is located outside the confines of the ebb shoal, and forcing is dominated by waves. The models have greater predictive skill in the critical area that in areas within the ebb shoal and along the inlet throat. The shoreline change model, with wave forcing alone, demonstrated limited skill within the confines of the ebb shoal.

To examine a solution to the high erosion at the east end, additional studies are recommended: acquisition of field data to assess the dominant wave and current forces that characterize the area, application of Coastal Inlet Research Program and Dredging Operation and Environmental Research Program technology (linked wave and circulation models and sediment transport and morphology change calculations) to examine the complex wave-current-sediment transport regime in this region, application of the GENESIS-T model (which can consider the role of tidal currents, where currents are provided by a circulation model or measurements), and assessment of the likelihood of success of various engineering solutions to the east end problem. If no further studies are pursued, then we recommend sand-tightening and possibly extension of the terminal groin.

We recommend continuation of a monitoring program for the entire Grand Isle beach, having the following components: periodic long beach profile surveys encompassing the dune and extending to a point beyond the active depth assumed for this project -12 ft NAVD88, aerial photography (end of summer photos are recommended), and a period of grain size collection and analysis following placement of the fill along the beach (see the CEM, 2002 for monitoring guidance). We recommend periodic bathymetric surveys and sediment sample collection and analysis in the borrow area, to assess its infilling rate and suitability for use in the future.