APPENDIX H: Sediment Transport Model

Grand Isle and Vicinity, Breakwater Design

Caminada Pass Borrow Source Analysis (Task 3, Amendment 2)

August 2020

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Grand Isle and Vicinity, Breakwater Design

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Executive summary

The goals of the Caminada Pass Borrow Source Analysis are to:

1. Evaluate the potential changes to sediment bypassing and wave transformation induced by dredging a sediment borrow source on the eastern lobe of Caminada Pass ebb shoal

2. Develop a sediment borrow source geometry on Caminada Pass Ebb Shoal that minimizes impacts on sediment bypassing from Caminada Headlands to Grand Isle while providing sufficient sediment to complete a proposed beach nourishment.

Numerical modeling was conducted to evaluate the potential impacts of various Caminada Pass ebb shoal borrow source configurations. Results show that a borrow source with a minimum volume of 750,000 cy located on eastern side of the shoal would have minimal impacts on sediment bypassing from Caminada Headland to Grand Isle and minimal changes on nearshore wave heights on Grand Isle or Elmer's Island.

Borrow source geometries were developed and tested to minimize changes to the sediment bypassing and wave heights in the nearshore. The proposed borrow sources caused a reduction to sediment bypassing from Caminada Headlands to Grand Isle of about 2 to 10%. When the borrow source geometry is aligned along the ebb shoal contours smaller changes occur to sediment bypassing from Caminada Headland to Grand Isle compared to dredge cuts that do not follow the ebb shoal contours.

Similar to the sediment bypassing results, changes to wave height near the Grand Isle and Elmer's Island shorelines are small (less than 0.4 ft), even for large cold-front storms. When comparing the different borrow source geometries, the wave height changes and gradient along the shoreline is smallest for borrow sources with the geometry aligned with the ebb shoal contours.

All borrow sources evaluated in this study were designed using the latest 2018 bathymetric surface; thus, all borrow sources need confirmation of top of cut elevations. We recommend verifying adequacy of borrow source volumes using an updated hydrographic surface.

1 Project Background

1.1 Introduction

This report discusses work completed under CPRA Contract Number 4400012419, Task Order 3, Amendment 2 for Grand Isle and Vicinity – Breakwater Design. The purpose of the overall Grand Isle Levee Dune Beach Stabilization and Beach Nourishment Project is to develop a design that stabilizes the western end of Grand Isle, protecting the Levee Dune and landward infrastructure, while maintaining a recreational beach. The project is needed to address the ongoing shoreline erosion and diminished protection against storm surge.

1.2 Project Goals

The west end of Grand Isle levee-dune and beach has experience chronic erosion and was severely damaged by Tropical Storm Cristobal. A beach and dune nourishment has been proposed to protect the integrity of the levee-dune. The borrow source for the beach nourishment is proposed on the eastern lobe of the Caminada Pass Ebb Shoal. The goals the analyses discussed in this report are to:

1. Evaluate the potential changes to sediment bypassing and wave transformation induced by dredging a sediment borrow source on the eastern lobe of Caminada Pass ebb shoal.

2. Develop a sediment borrow source geometry on Caminada Pass Ebb Shoal that minimizes impacts on sediment bypassing from Caminada Headlands to Grand Isle while providing sufficient sediment to complete a proposed beach nourishment.

1.3 Summary of Previous Coastal Engineering Analysis

1.3.1 Hot Spot Erosion on Grand Isle's West End

In May 2017 Mott MacDonald completed a Coastal Processes Analysis and Alternatives Development for CPRA, referred to as Phase 1 of the project. The objective of the study was to understand the causes of the erosion in the southwest end of the Island, and to develop solutions to stabilize the shoreline and protect the levee-dune system. The proposed solution was a breakwater field on the westernmost end of the Island in combination with beach nourishment.

Longshore transport (LST) patterns were evaluated using numerical wave modeling, shown in Figure 1. Net sediment transport for the Island shoreline was found to be predominantly from the south to the north. However, wave modeling indicated that the Caminada Pass ebb shoal modifies the wave transformation near the west end of the Island, producing a divergent node in longshore sediment transport. This results in a sediment transport reversal at the west end of Grand Isle. The divergent node creates an erosional hot spot which has generated the chronic erosion observed on the west end. Shoreline change analysis confirmed the presence of the erosional hot spot.

An evaluation of the morphology of the Caminada Pass ebb shoal shown in Figure 2 illustrates the seaward migration of the Caminada Pass ebb shoal. As the Barataria Bay tidal prism increases, sediment deposition on the ebb shoal increases, which results in an increase in the Caminada Pass ebb shoal volume and a seaward migration of the ebb shoal. As the ebb shoal grows, the waves refract on the evolving bathymetry resulting in a concentration of wave energy which has led to divergent LST and resulting erosional hot spot.

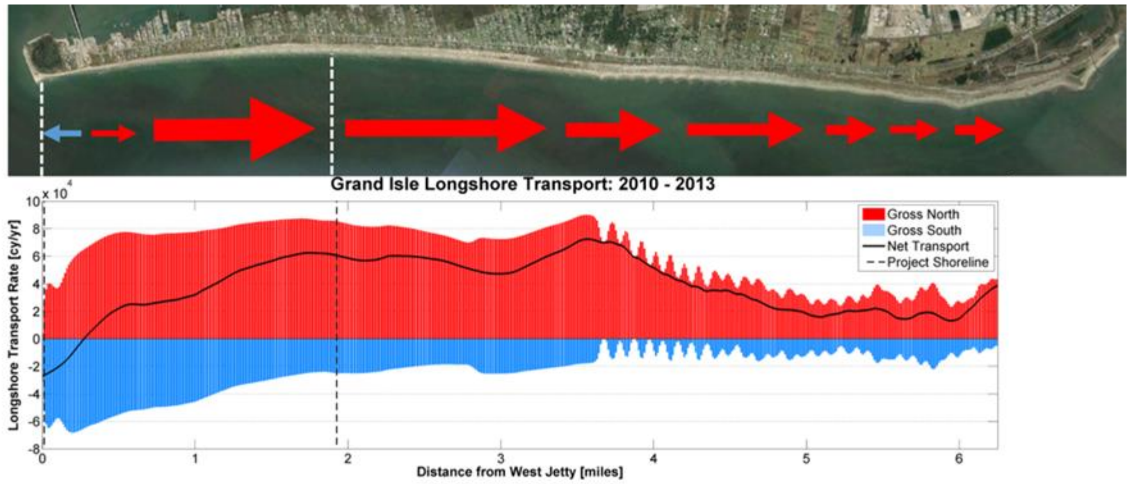

Figure 1. Computed LST rates from 2010 to 2013. Gross transport directed toward the southwest is shown with blue bars, gross transport directed toward the northeast is shown in red bars, and the thick black line shows the net longshore transport rate.

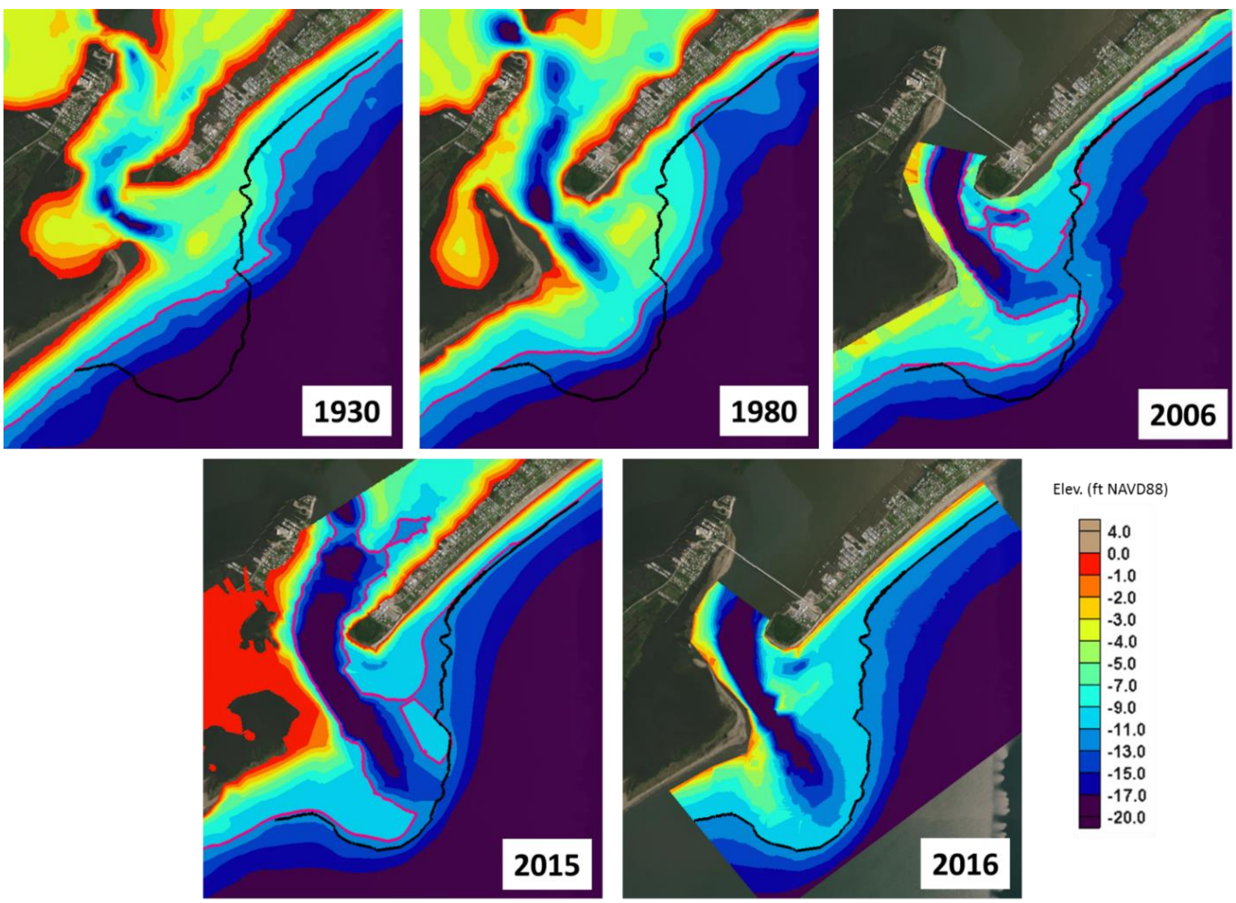

Figure 2. Western end of Grand Isle bathymetric surfaces for 1930, 1980, 2006, 2015, and 2016. Pink line represents 11-ft contour for the given year; black line represents the 2016 11-ft contour. Scale applies to all plots.

The erosional hotspot present along the western end of the Grand Isle shoreline has impacted the Federal projects with erosion rates higher than the planned maintenance rate. The GI-01C project (revetment) was successful in protecting the levee-dune in its immediate lee but does not alleviate erosion adjacent to the structure. The proposed alternative, shown in Figure 3 was designed to provide sand and rock structures to retain the sand at the site.

Figure 3. Preferred alternative site plan: Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill.

1.3.2 Breakwater Field Optimization and Caminada Pass Borrow Source Analysis (Mott MacDonald, 2019)

Project Stakeholders advocated to build the breakwater field along much of the western end of the project shoreline. However, breakwaters extending further westward than proposed in the concept presented in Figure 3 may prevent the sediment bypassed across Caminada Pass from interacting with the nearshore beach, resulting in erosion to the Grand Isle shoreline. As a result, in March 2019 Mott MacDonald completed a breakwater analysis for CPRA, referred to as Phase 2 of the project. The objective of the study was twofold: (1) to evaluate the length of the breakwater field proposed on Phase 1 to reduce interference with the natural bypassing of sand from the Caminada Headland onto Grand Isle, and (2) to assess the potential impacts of using the Caminada Pass ebb shoal as a borrow source for beach fill on the Grand Isle shoreline.

To simulate the Caminada Pass shoal dynamics and the associated sediment bypassing, a Delft3D process-based numerical model was developed. The model included coupled circulation, waves, sediment transport, and morphology for a three-year simulation from 2015 through 2018. The model was used to compare the relative impacts of breakwaters and dredge borrow pits to existing conditions.

The results of the existing condition model indicated net sediment transport directed toward the northeast with increasing sediment transport in the center portion of the island, which matches well with previous observations and analyses. Model results in Figure 4 show the mean total sediment transport, and illustrate bypassing from Elmer's Island over the Caminada Pass ebb shoal onto Grand Isle. The analysis also indicated the presence of a divergent node on the western end of Grand Isle resulting in an erosional hot spot consistent with Phase 1 results. The erosional area extends between 0 mi to approximately 0.6 mi from the jetty where the Grand Isle shoreline stabilizes. It has been noted the 0.6 mi location matches the eastern end of the 2017 revetment at station 51+00. Figure 5 illustrates the realtionship between the GI-01A project stationing and cross-shore transects along the Grand Isle shoreline.

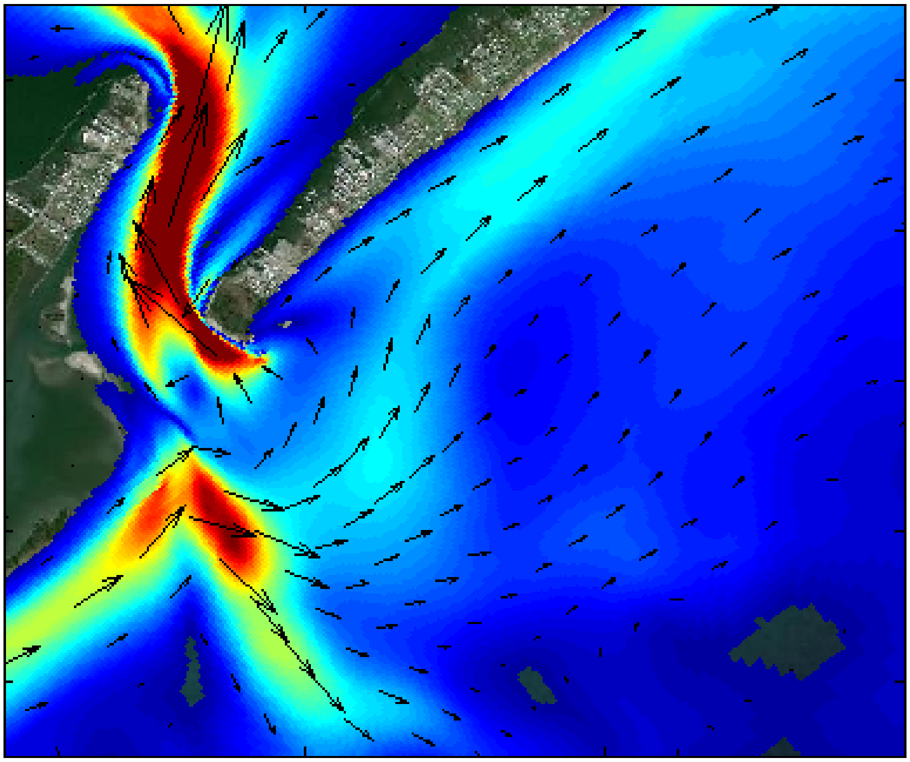

Figure 4. Sediment transport vectors at Caminada Pass ebb shoal.

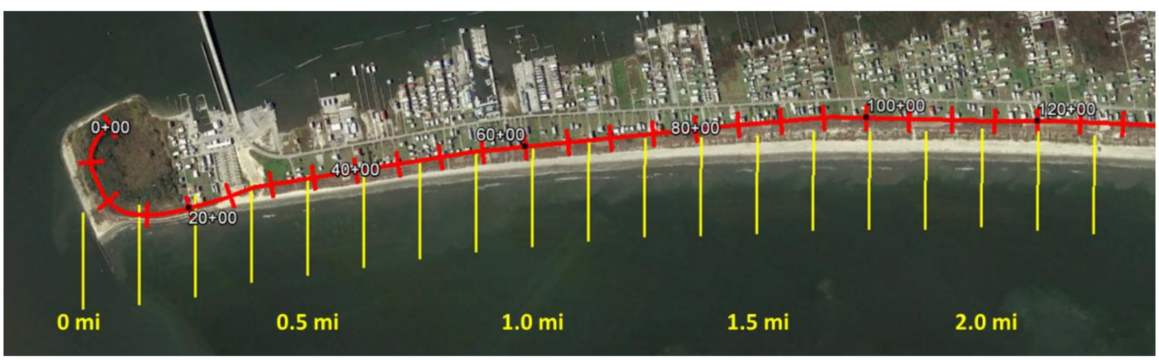

Figure 5. GI-01A project stationing shown in red and cross-shore transects shown in yellow with distance from the jetty on miles along the Grand Ilse shoreline.

A 5-breakwater field was evaluated using the Delft3D model. The results (Figure 6) showed that the 5-breakwater field has no negative impacts on sediment bypassing from the Caminada Headlands to Grand Isle. Delft3D results indicated the 5-breakwater field does not reach the location where the sediment bypassing attaches onto the Grand Isle shoreline, and it showed benefits of reducing erosion between 0 mi to 0.6 mi from the jetty.

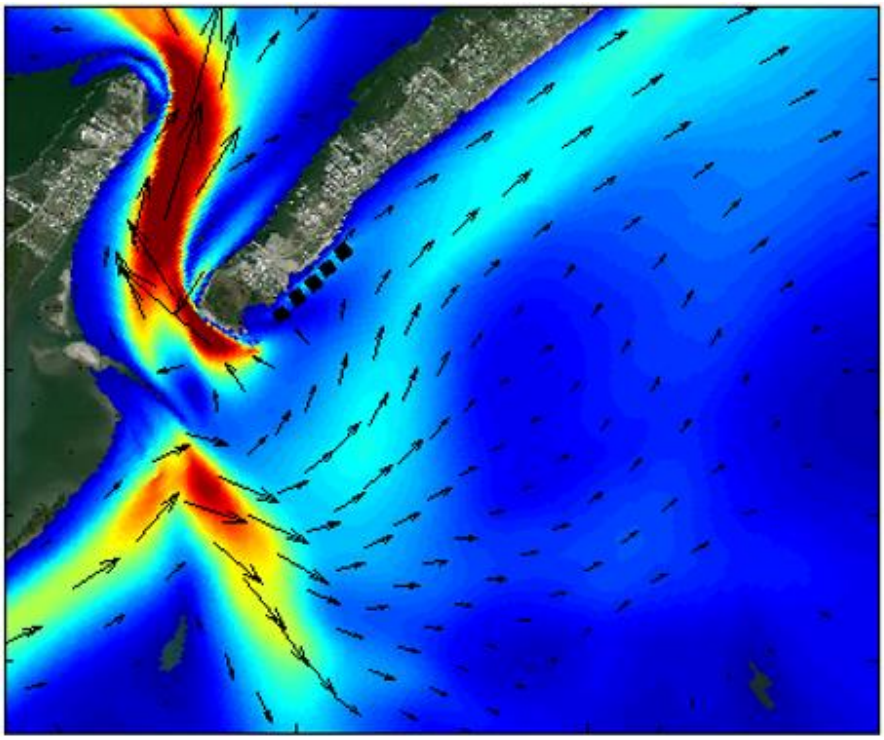

Figure 6. Sediment transport vectors at Caminada Pass ebb shoal with breakwaters.

These results were further quantified using the shoreline morphology model Gencade. The Gencade analysis showed that with the 5-breakwater field the shoreline position for the entire western end of Grand Isle is seaward of the future without project (FWOP) shoreline and has no negative impact when compared to the FWOP. At year 5, for the 5-breakwater field, the beach is at or seaward of the initial shoreline position for the area of interest.

Figure 7. Shoreline change computed by Gencade model at year 5 after construction for FWOP (red line), 5-breakwater (blue line) and 10-breakwater (green line). Positive (negative) means seaward (landward) of initial shoreline position.

The 5-breakwater field construction was completed on July 2020. For further details on the breakwater field study see Appendix B (Mott MacDonald, 2017).

A preliminarily proposed borrow pit (Pit 1) located on the eastern lobe at Caminada Pass (Figure 8) was defined, and its impacts on sediment bypassing were evaluated using the Delft3D model. The model results, shown in Figure 9, indicate that the Pit 1 will reduce the sand bypassing to Grand Isle by 18%.

Generally, the model results indicate that the impacts of dredging the Caminada Pass ebb shoal may be mild to moderate and the Caminada Pass is a feasible borrow source that should be considered with further evaluation.

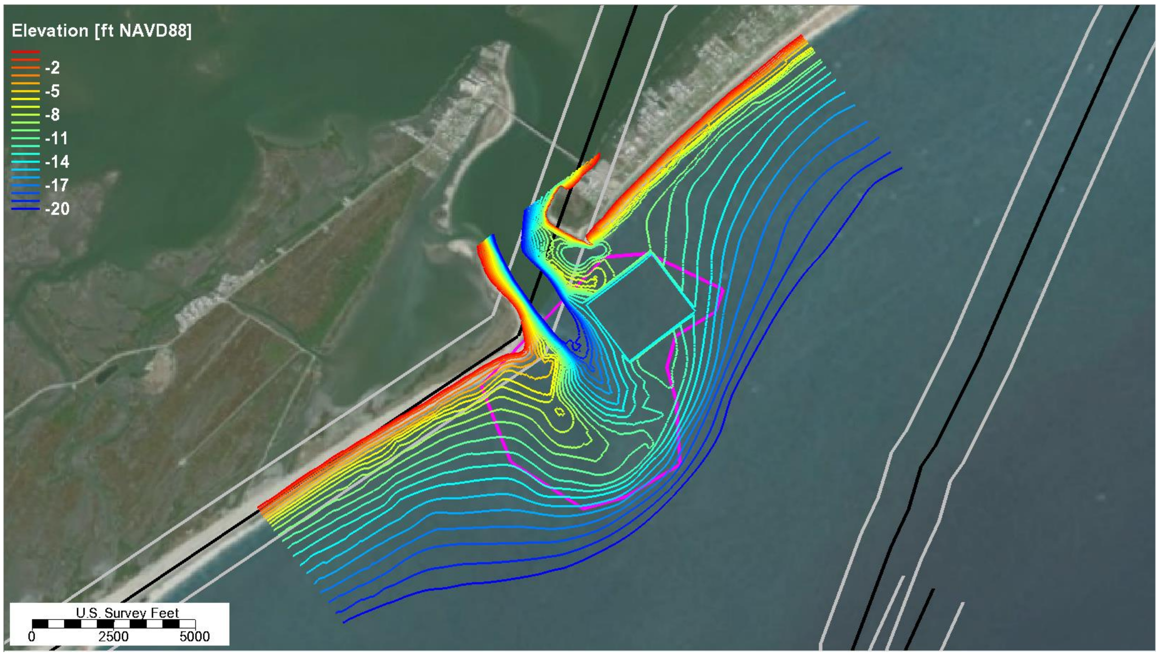

Figure 8. Plan view of Borrow source 1 bathymetry contours; "Caminada Sand Body" in magenta; known existing pipelines in black with corresponding buffers in grey.

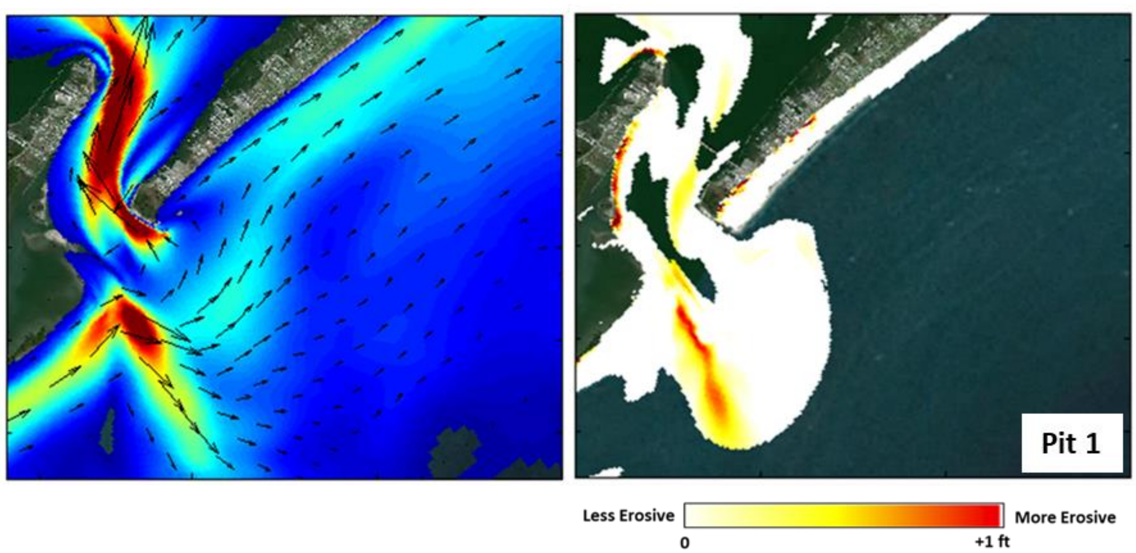

Figure 9. Sediment transport vector field over Caminada Pass ebb shoal for Pit 1 (left) and morphology changes resulting from Pit 1 (right).

2 Caminada Pass Borrow Source Design

The goal of this section is to define a borrow source on the eastern lobe of Caminada Pass ebb shoal within the NEPA bounds provided by the USACE and based on all geotechnical data available. The proposed borrow source impacts on Caminada Pass sediment bypassing are discussed on Section 3. This section references data provided by USACE and CPRA.

2.1 Available Data and Existing Design Constraints

Required Volume

Based on the post Tropical Storm Cristobal hydrographic survey conducted by HydroTerra on June 2020, the in-place sand fill volume for the beach nourishment would be 375,500 cy (Byland, 2020), with a cut to fill ratio of 1.75 (USACE, 2020). For a conservative estimate a total volume of sediment required to be dredged was established at 750,000 cy (CPRA, 2020).

Bathymetry

The latest bathymetric surface on Caminada Pass is based on the hydrographic survey data collected by the USACE on November 2018. Therefore, all the borrow source volumes presented on this report are based on the 2018 hydrographic survey. The 2018 bathymetric surface is shown in Figure 10.

Spatial Constraints

All the borrow sources presented in this report fall within the NEPA regulatory bounds established and provided by the USACE (USACE, 2020). In addition, a cultural resource remote-sensing investigation of a portion of the Caminada Pass area was conducted by Coastal Environments Inc. Overall, three anomalies of interest were identified within the NEPA regulatory bounds in the remote-sensing data collected. It is not possible to determine from the remote sensing data alone if these anomalies are related to cultural resources that meet National Register of Historic Places criteria (Coastal Environments Inc. , 2020). Therefore, these areas were avoided with a 164 ft (50 m) radius buffer from the outside edge of the anomaly[*]. The USACE NEPA bounds and the anomalies found in the cultural resource remote sensing survey are shown in Figure 10.

Geotechnical Data

The USACE collected three borings within the NEPA regulatory bounds; the boring locations are shown on Figure 10 and the associated results on Figure 11. The USACE borings indicate suitable beach fill material at a depth of -20 ft NAVD88 with a sand content of approximately 90% or higher.

Due to the limited spatial coverage of the geotechnical data provided by the USACE, CPRA collected and processed four vibracore samples with a maximum depth of 10 ft within the southern region of the NEPA regulatory bounds to gain confidence on the quality and suitability of the material in the Caminada Pass Ebb Shoal; the CPRA sample locations are shown on Figure 10. The results shown in Table 1 confirm the availability of fine sand for beach nourishment purpose at a depth of -20 ft NAVD88.

[*] The modeling was conducted using a 50 ft buffer rather than a 50 m buffer. However, the difference in radii is small compared to model resolution and scale of changes resolvable by the model, and therefore any changes to hydrodynamic results are negligible.

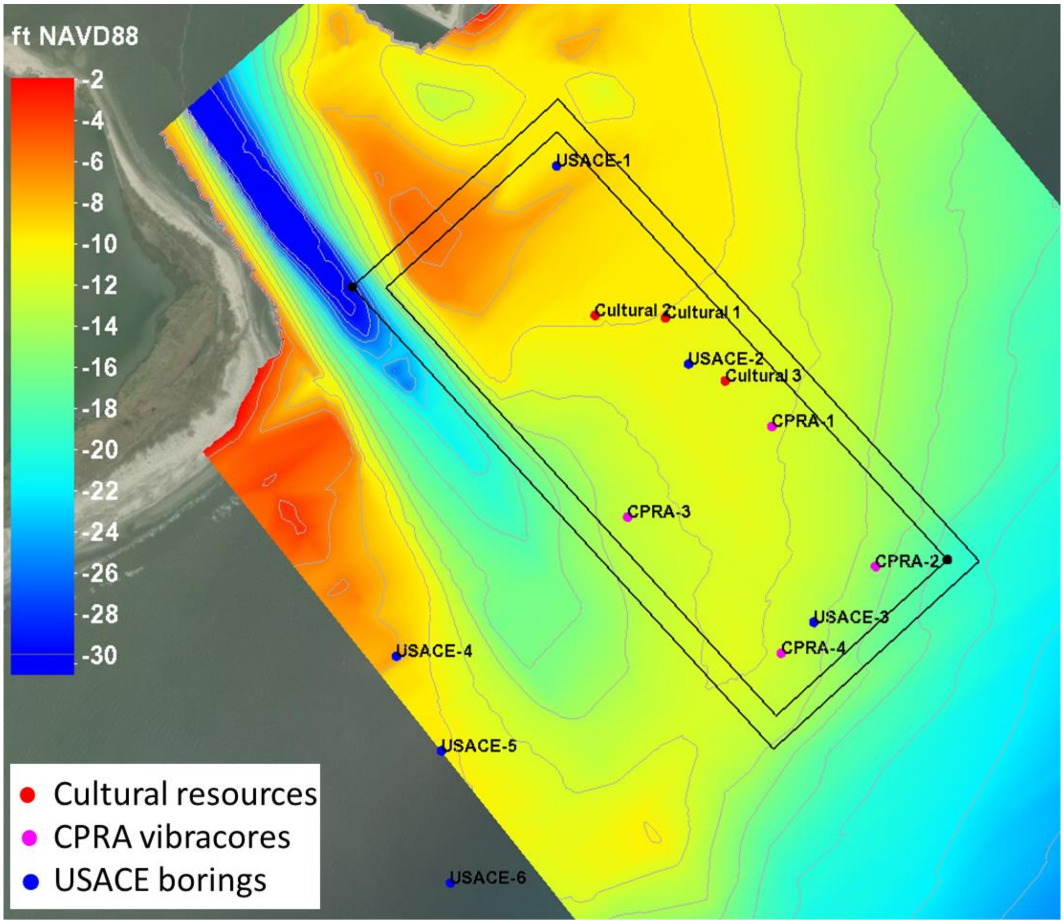

Figure 10. Bathymetric surface based on 2018 USACE hydrographic survey, USACE NEPA regulatory bounds shown in black rectangle, cultural resource anomality, USACE geotechnical borings, and CPRA vibracores locations.

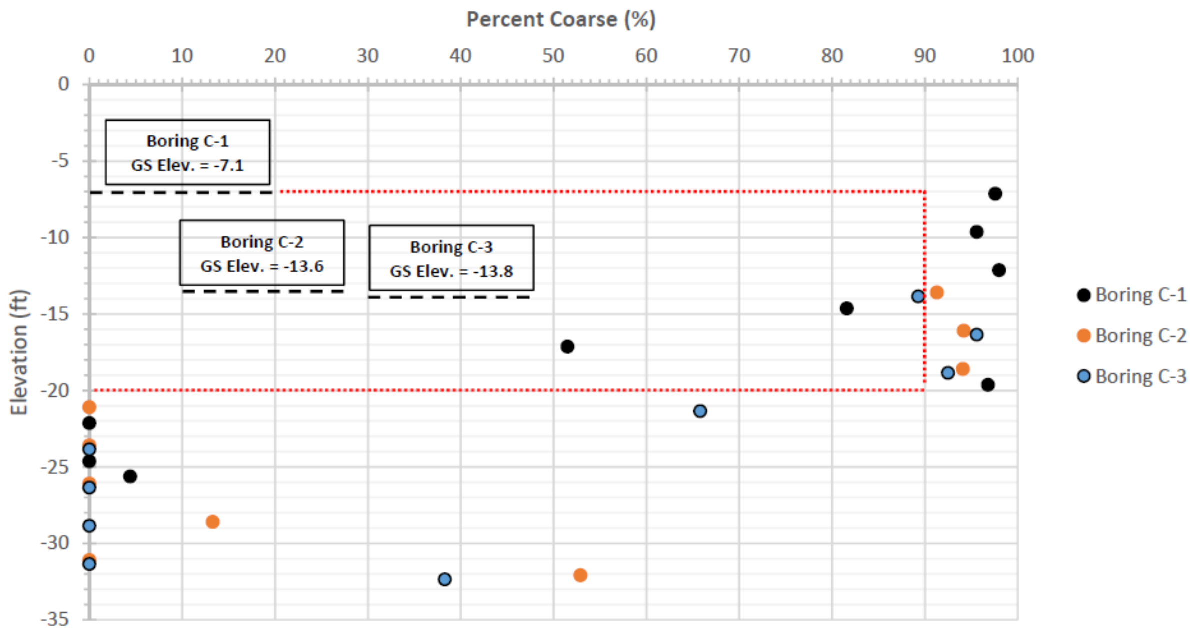

Figure 11. USACE geotechnical boring data collected at Caminada Pass (note: Boring C-1 is the same as USACE-1 shown in Figure 18, C-2 is USACE-2, and C-3 is USACE 3).

Table 1. CPRA vibracore material description

Vibracore ID	Max depth [ft]	Associated percent fine sand	Material description
CPRA-1	10	80.4%	Gray Poorly Graded Sand (SP)
CPRA-2	9	96.8%	Gray Poorly Graded Fine Sand (SP)
CPRA-3	8	89.7%	Gray Poorly Graded Fine Sand (SP)
CPRA-4	7	93.9%	Gray Poorly Graded Sand (SP)

2.2 Borrow Source Design

2.2.1 USACE Borrow Source Design

The Caminada Pass borrow source layout proposed by the USACE consists of a rectangular area bounded by the NEPA regulatory perimeter, see Figure 12. The USACE borrow source is divided into two sections called USACE-A and UASCE-B. The borrow source volumes when dredged to -20 ft NAVD88 are shown in Table 2.

Table 2. USACE borrow source volumes

Borrow source ID	Volume [CY]	Bottom elevation [ft NAVD88]
USACE-A	1,510,800	-20
USACE-B	2,015,800	-20
USACE-AB	3,526,600	-20

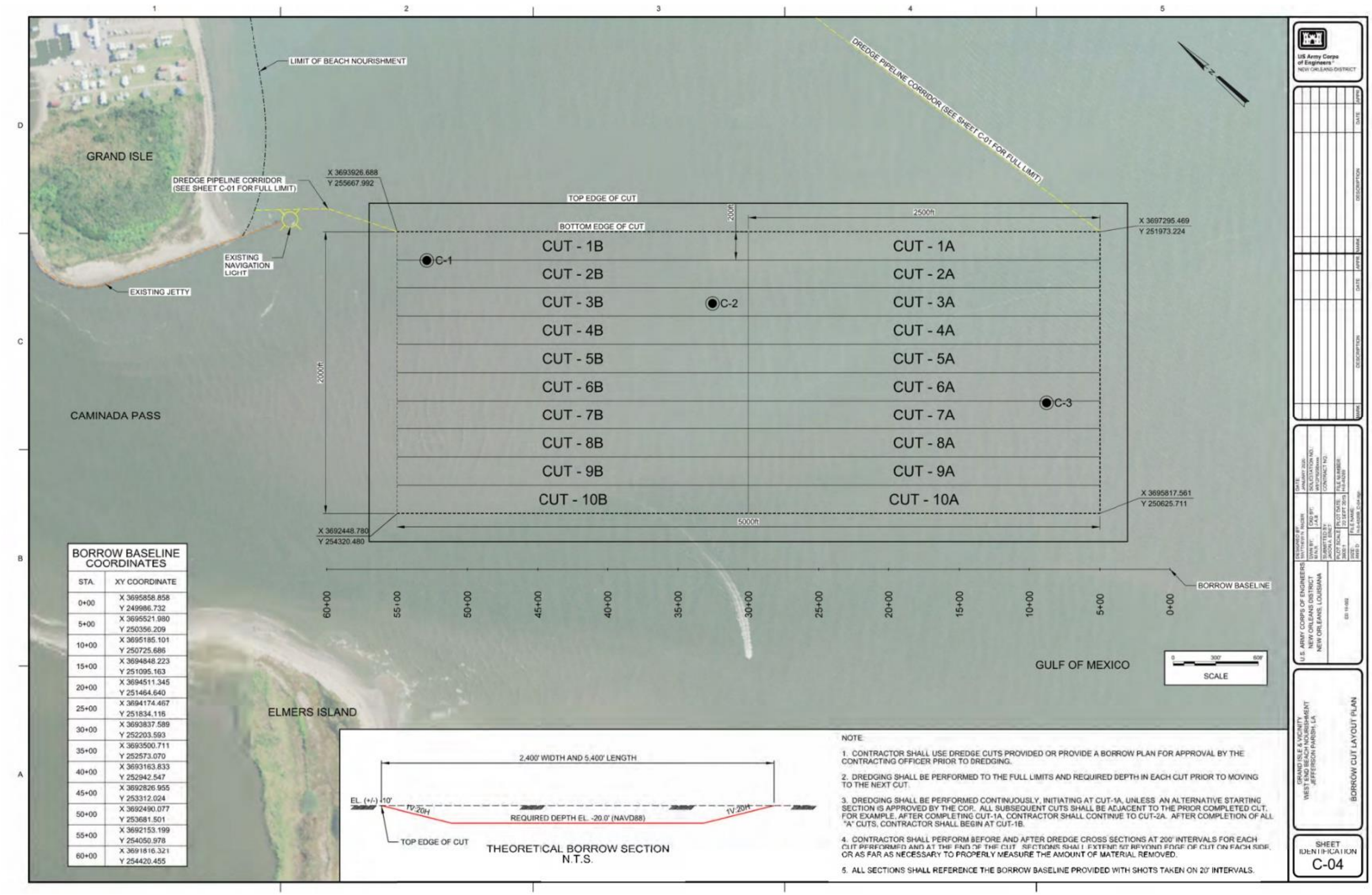

Figure 12. USACE Caminada Pass borrow source layout plan.

2.2.2 Borrow Source Alternatives

To minimize impacts to sediment bypassing and wave climate on Grand Isle shoreline and Elmer's Island, several borrow source configurations were developed and tested. The different configurations included flat bottom, stepped terrace, and aligning the pit parallel to Caminada Pass ebb shoal contours. The overall borrow source geometries and volumes are shown in Figure 13 and Table 3, respectively; the associated results are shown in Appendix A.

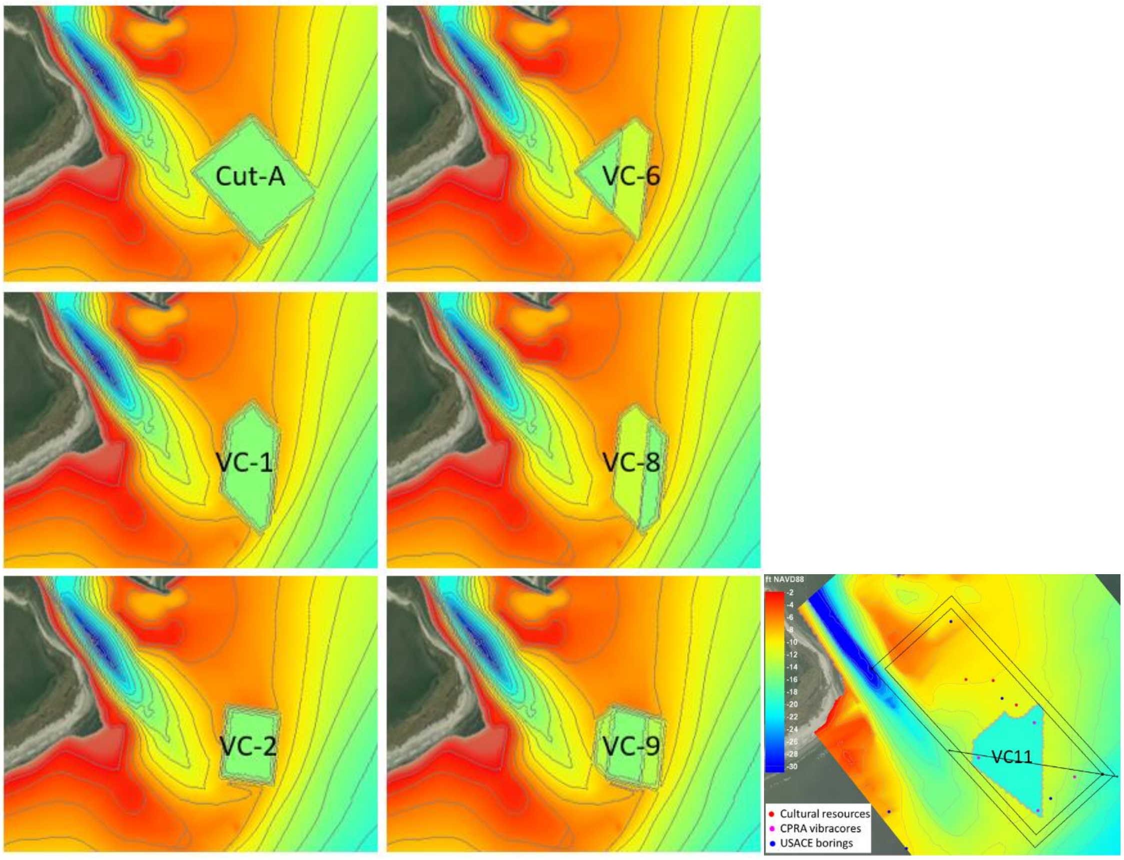

Figure 13. Borrow source geometries on 2018 bathymetric surface.

Table 3. Borrow source volumes based on 2018 bathymetric surface.

Pit ID	Volume [cy]	Bottom elevation [ft NAVD88]
USACE-A	1,510,800	-20
VC1	1,054,500	-20
VC2	742,600	-20
VC6	763,200	-20, -17
VC8	748,500	-17, -20
VC9	814,600	-18, -20, -18
VC11	764,500	-20

The results presented in this report focus on USACE-A and VC11 shown in Figure 15, with sections shown in Figure 14. CPRA intends to apply for a permit covering the entire area of USACE-A to a depth of -20 ft NAVD88; thus, the USACE-A modeling results are included in the report. However, borrow source VC11 provides sufficient and suitable beach fill material for the proposed Grand Isle Project. Borrow source VC11 has the following characteristics:

- Bounded by the NEPA regulatory perimeter
- Aligned with Caminada Pass ebb shoal contours
- Flat bottom facilitating dredging operations
- Encompasses three of the CPRA vibratory cores confirming the suitability of the beach fill material
- Includes a 164 ft (50 m) buffer from the outside edge of the cultural resource anomaly

After completing design optimization, which included borrow source depth, orientation, and volume calculations, in combination with numerical modeling, borrow source VC11 was found to be the best performing design in that it meets the required volume and has least impacts to bypassing and wave climate; the results are detailed in Section 3.

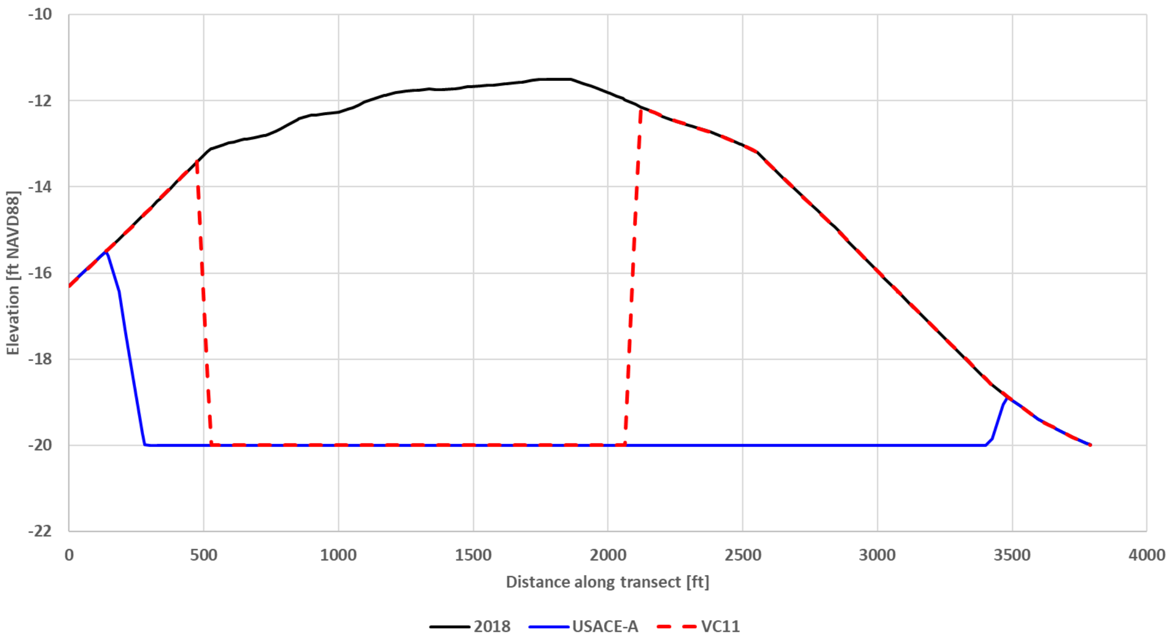

Figure 14. USACE-A and VC11 borrow source profiles along USACE-A diagonal (see Figure 15), based on 2018 bathymetric surface

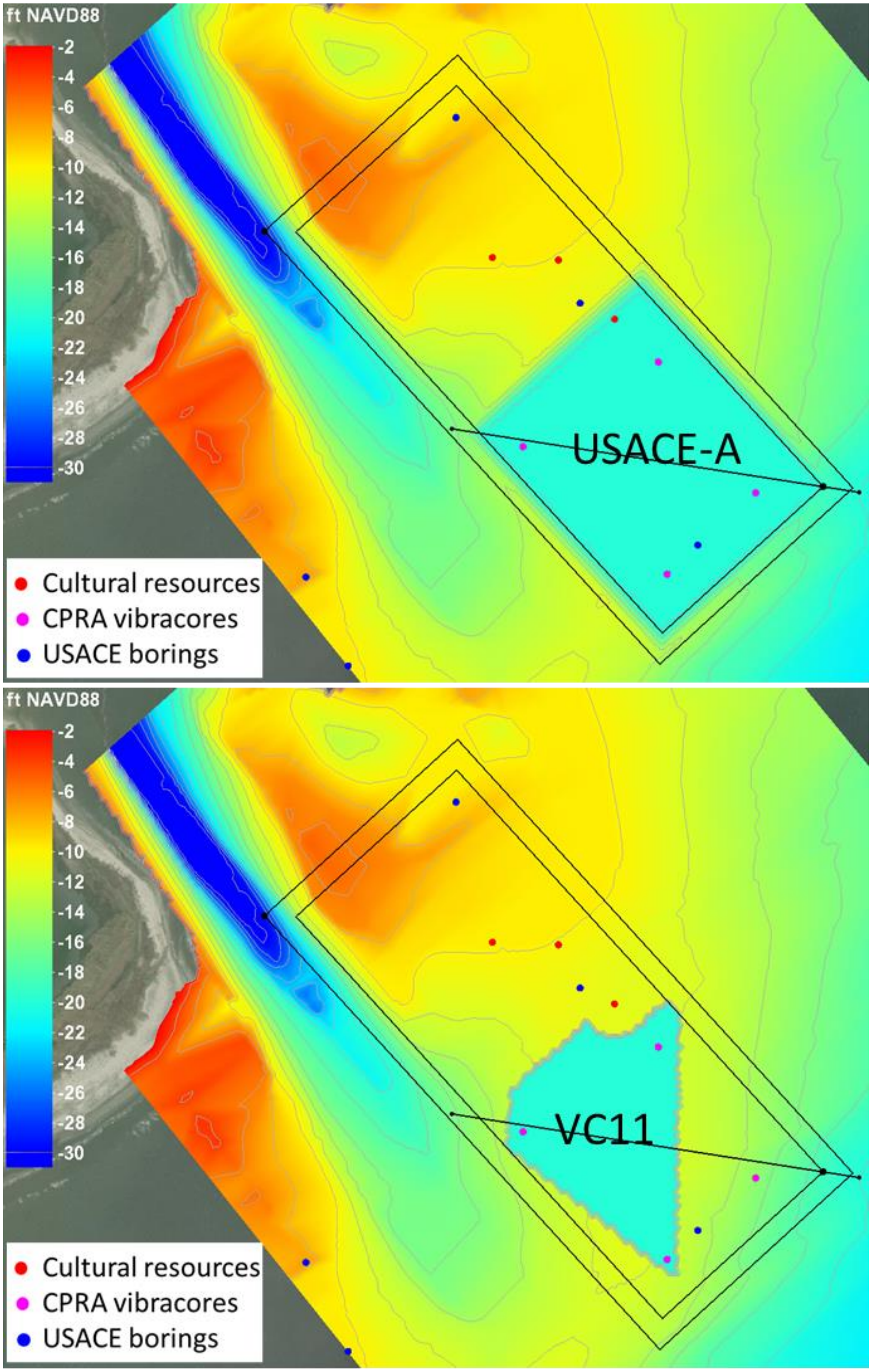

Figure 15. USACE-A and VC11 borrow sources on 2018 bathymetric surface.

3 Evaluation of Caminada Pass Borrow Source Impacts

Numerical modeling was conducted to evaluate the potential changes to sediment bypassing across Caminada pass and the local wave climate resulting from the proposed borrow sources on Caminada Pass ebb shoal. The numerical model Delft3D was utilized for the evaluation; the model simulated coupled tidal circulation, wave transformation, sediment transport, and bottom morphology. The dredge borrow sources are evaluated based on a relative comparison, i.e. comparing the without-project versus with-project conditions. The intent of the model is not to robustly quantify all sediment transport and morphological process in Caminada Pass shoal and Grand Isle but to understand the changes that proposed borrow sources have on the coastal dynamics.

3.1 Numerical Model

Model Settings

Numerical modeling was conducted using the process-based model suite Delft3D employed in Phase 2. Two nested models were used in the analysis: (1) a global model to capture the overall interaction of the hydrodynamics and waves of the Gulf and Barataria Bay and (2) a high-resolution nested model to capture the detailed Caminada Pass dynamics including circulation, waves, sediment transport, and morphology; all of which affect the project shoreline. The model bathymetric surface is based on the 2015 nearshore bathymetry and includes the 5 breakwaters constructed on the southwest end of Grand Isle.

Environmental Forcing

A time series of 12 reduced wave cases, water surface elevations, and morphological acceleration factor (MORFAC) shown in Figure 16 was used as environmental forcing conditions. The Delft3D model was run for a three-year period, using a representative set of wave cases to approximate the wave climate for that time period. The number of wave cases were chosen to produce sediment transport patterns that are similar to those experienced on the full time series of offshore waves. To account for the percent occurrence of each wave case and the duration of the study period, a variable Morphological Acceleration Factor (MORFAC) was used. The offshore wave climate during the modeling period was based hindcast waves between June 1, 2015 and June 1, 2018.

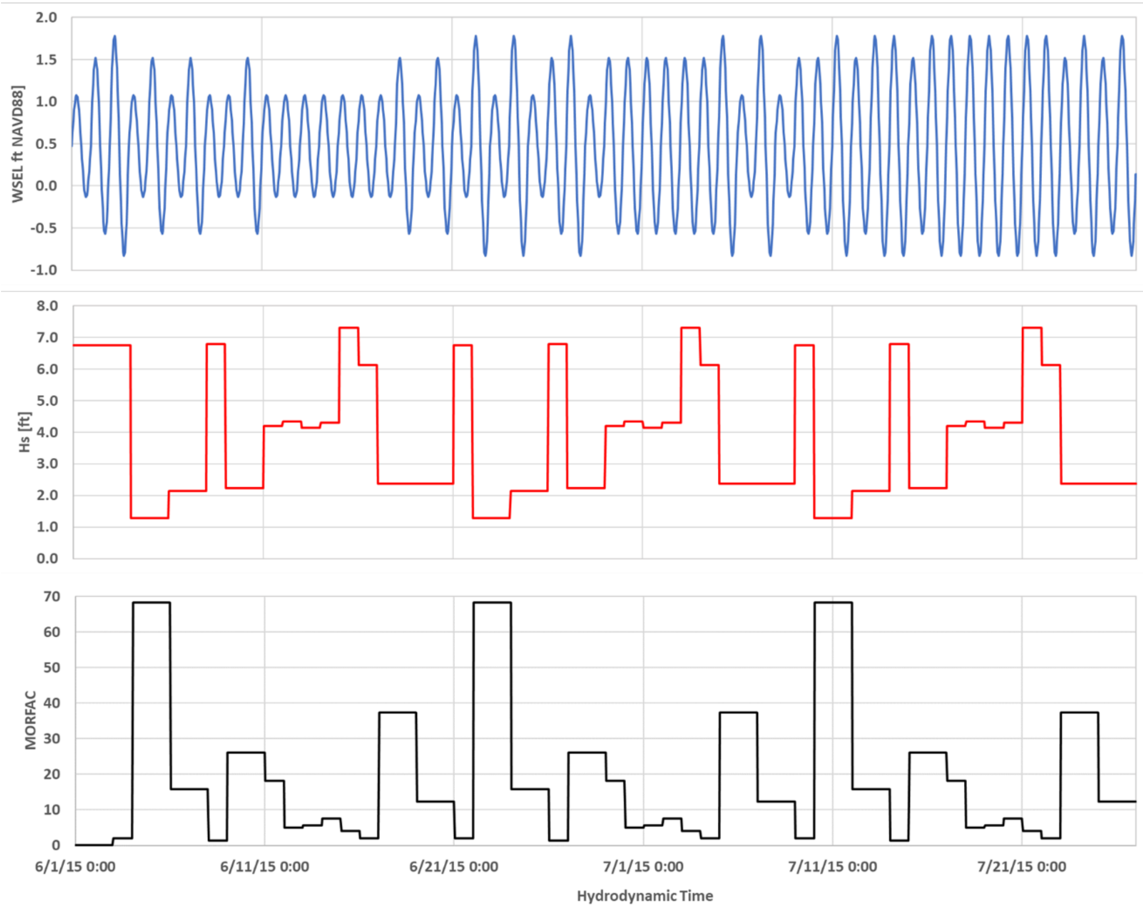

Figure 16. Time series of environmental forcing conditions: water surface elevations (top), wave height (middle), and MORFAC (bottom).

Sediment Class

Sediment transport rates are dependent on the sediment size. Three different spatially varying sediment classes were employed in the model to represent observed sediment gradations along the project vicinity where generally coarser material is located in the nearshore and finer material in the offshore: (1) fine sand with d_{50} equal to 200 µm, (2) very fine sand with d_{50} equal to 100 µm, (3) mud (sediment with d_{50} less than 80 µm).

For further details on the numerical model set up refer to Appendix B (Mott MacDonald, 2019).

3.2 Changes to Sediment Bypassing

The goal of the sediment transport analysis is to understand the sediment bypassing over the Caminada Pass ebb shoal. The changes to sediment bypassing due to the proposed borrow sources were evaluated using the numerical model described in section 3.1.

The impacts of the borrow pits on sediment bypassing were computed by calculating the flux of sand that is directed from the ebb shoal to the Grand Isle nearshore through the area shown in the black rectangular box (Figure 17); results are shown in Table 4. To further illustrate the changes in sediment bypassing over Caminada Pass ebb shoal, mean total transport difference plots are illustrated on Figure 18.

Table 4. Percent reduction in sediment bypassing over Caminada Pass ebb shoal for USACE-A and VC11 borrow sources with respect to existing conditions. For comparison, results from Pit 1 in the previous phase are included.

Simulation	% reduction in sediment bypassing from existing conditions
USACE-A	-10.0%
VC11	-2.2%
Pit 1	-18.0%

When comparing the USACE-A and VC11 borrow source results, the USACE results in a larger reduction in sediment bypassing compared to the existing condition. The smaller reduction in sediment transport resulting from VC11 borrow source can be attributed to a smaller borrow source volume and the geometry of the borrow source aligned parallel to the existing Caminada Pass ebb shoal contours. Both USACE-A and VC11 result in smaller impacts to bypassing compared to Pit 1 evaluated in the previous phase of the project.

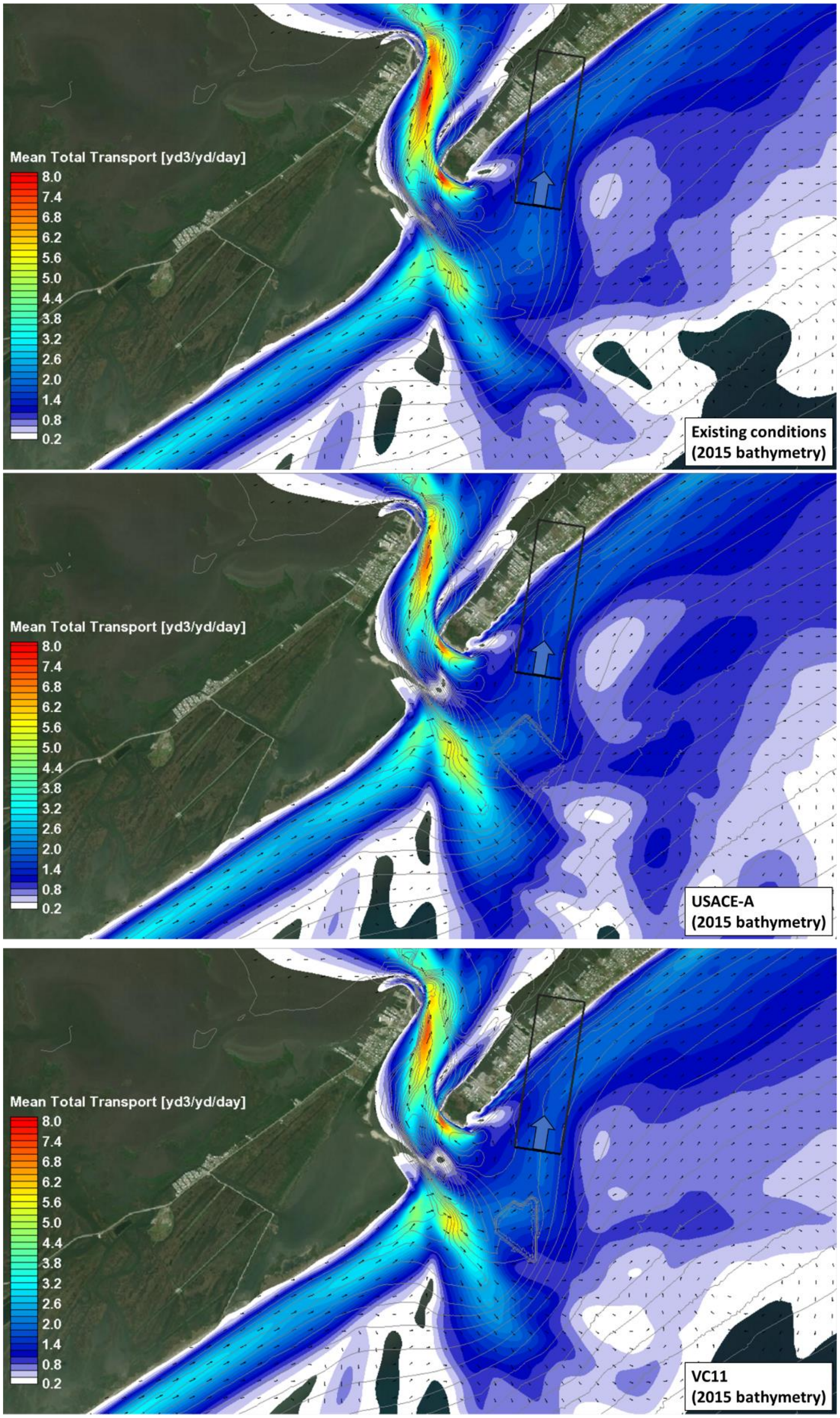

Figure 17. Mean total transport at Caminada Pass ebb shoal for existing (top), USACE-A borrow source (middle), and VC11 borrow source (bottom).

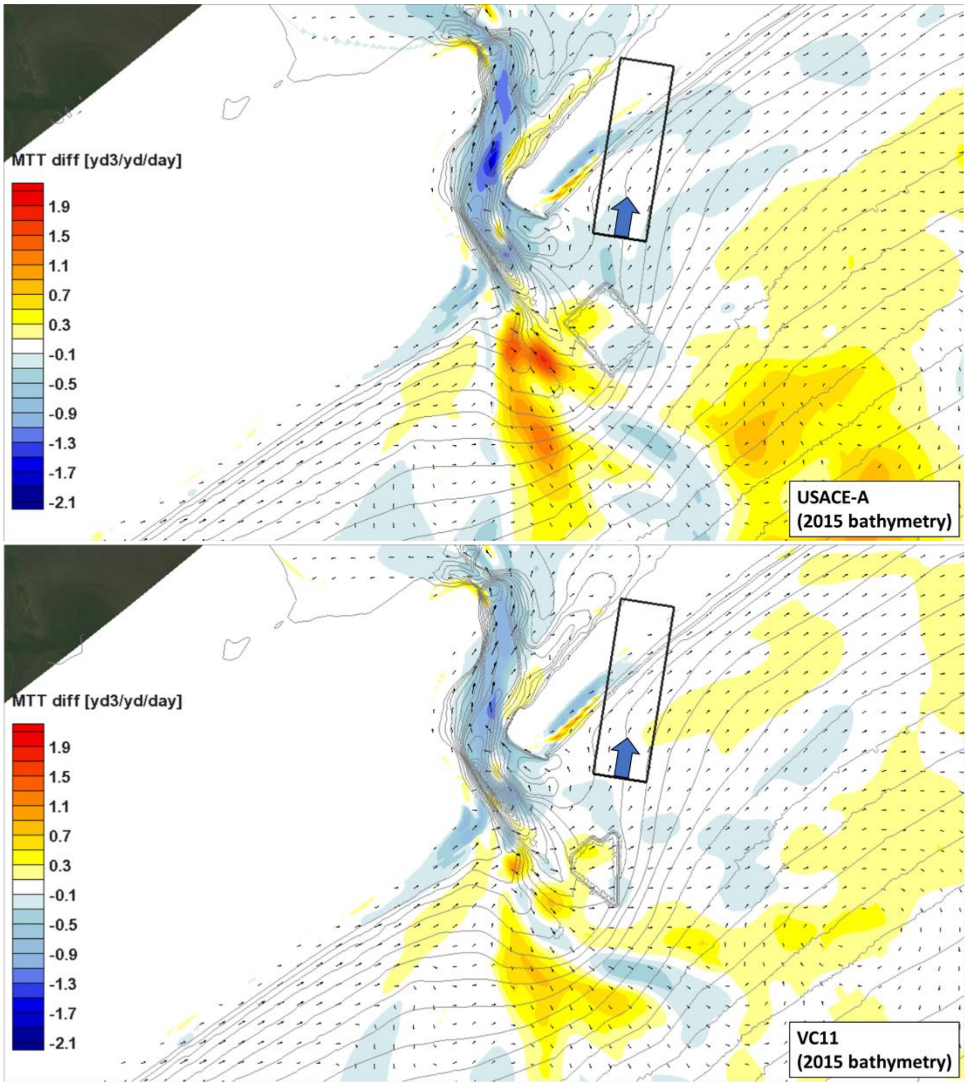

Figure 18. Mean total transport difference. USACE-A borrow source minus existing (top) and VC11 borrow source minus existing (bottom). Blue (red) represents a reduction (increase) in sediment bypassing

3.3 Impacts to Nearshore Wave Climate

Dredging a borrow source on the Caminada Ebb shoal may modify the wave climate along the project site. Previous analysis of the ebb shoal morphology showed that changes to the shoal can modify the wave climate which can result in localized hot-spot erosion. Therefore, we have evaluated the potential impacts of the proposed Caminada Pass borrow sources on wave climate of Grand Isle and Elmer's Island nearshore as an indicator of potential shoreline morphology induced by the borrow source.

The 12 wave cases described in section 3.1 were modeled for the existing conditions as well as the USACE-A and VC11 borrow source conditions. For conservative purposes, the wave transformation modeling coupled with tide circulation was conducted without accounting for bed level changes; in other words, the bathymetric surface remained constant during the wave simulations. The differences in significant wave height between the with-dredge-pit conditions and existing conditions are shown in Figure 19 and Figure 20 for USACE-A and VC11, respectively.

The borrow sources result in less than 1.5 ft change in wave height in the project vicinity, with smaller changes occurring in the nearshore. To further illustrate, wave height results were extracted at the -8 ft NAVD88 contour elevation; results are shown on Figure 21 and Figure 22 for USACE-A and VC11, respectively. In the USACE-A borrow source, the largest increase in wave height either on Grand Isle or Elmer's Island does not exceed +0.4 ft, having an average wave height increase less than 0.1 ft. The increase appears to be more constant on Elmer's island whereas Grand Isle experiences both an increase and decrease in average wave heights depending on alongshore location. To illustrate impacts from a larger storm event, we simulated a condition representative of the largest typical cold front that could be expected every year, which is shown with the red line on the bottom plot in Figure 21 and Figure 22; the borrow sources lead to only a marginal increase in wave height for this storm.

The VC11 borrow source wave height changes are even smaller than the USACE-A borrow source. The VC11 envelope of wave height increase is on the order of +0.3 ft, with negligible changes particularly on Elmer's Island nearshore. The representative cold front simulation resulted in a less than +0.1 ft increase in wave height. Also, the wave height gradient along the Grand Isle shoreline (responsible for the erosional hotspot) is smaller on VC11 than USACE-A.

Overall, the wave model and sediment bypassing results indicate better performance of the VC11 borrow source over USACE-A. While both borrow sources result in minimal changes in wave height with respect to existing conditions, VC11 resulted in more favorable impacts to the gradient in nearshore wave heights compared to USACE-A.

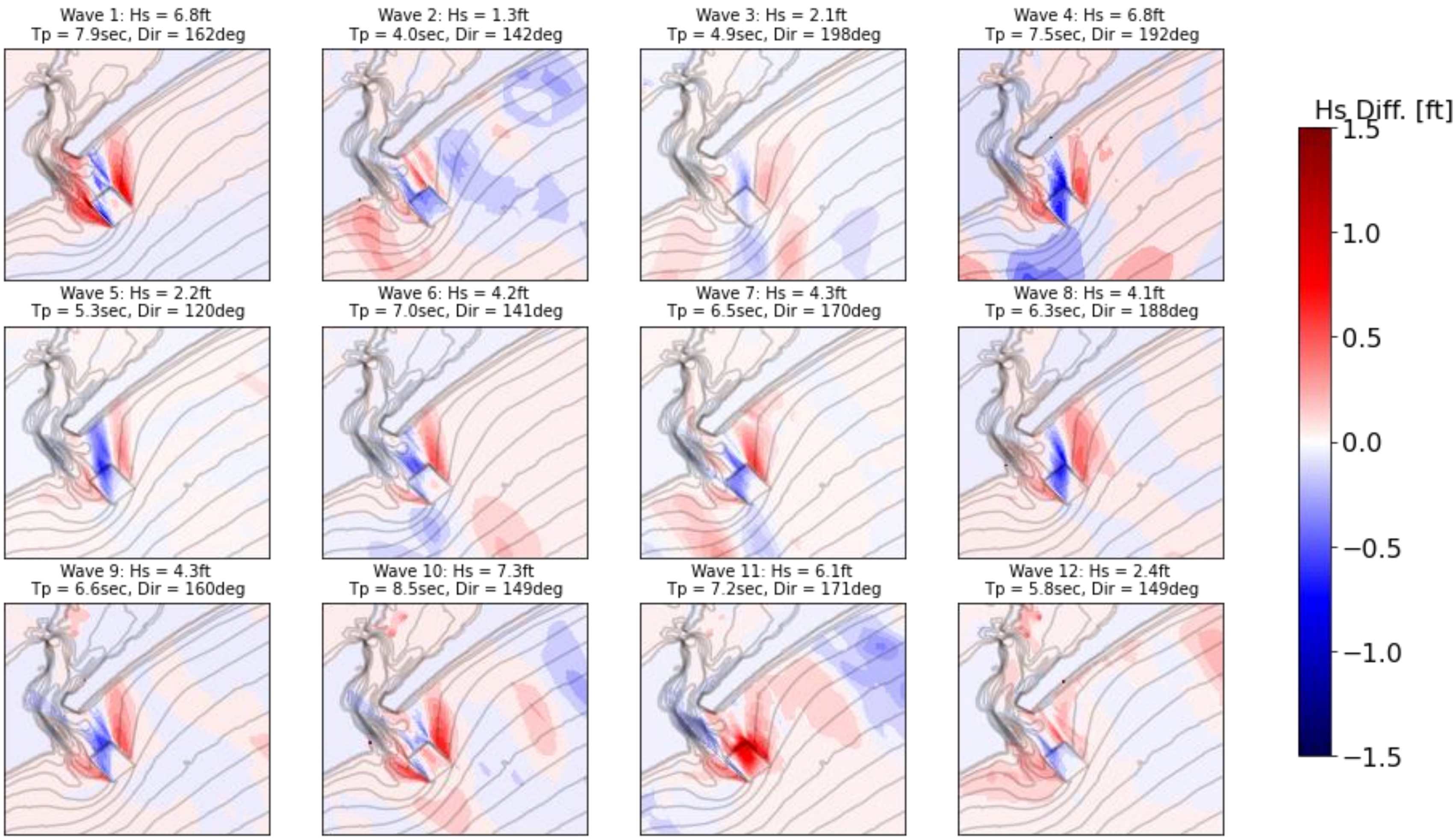

Figure 19. Difference in significant wave height between USACE-A borrow source and existing conditions for 12 wave cases, where red (blue) represents an increase (decrease) in wave height.

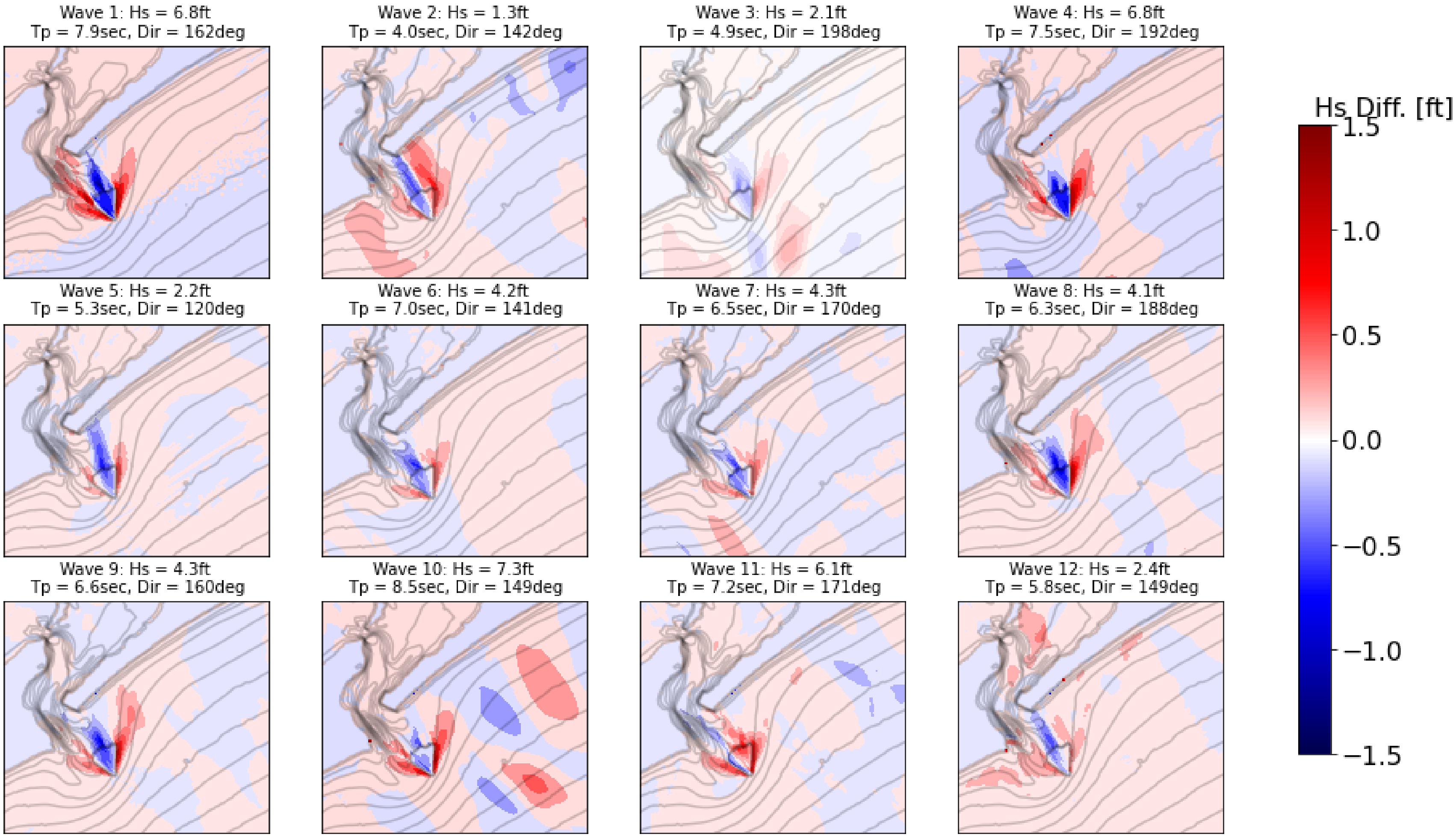

Figure 20. Difference in significant wave height between VC11 borrow source and existing conditions for 12 wave cases, where red (blue) represents an increase (decrease) in wave height.

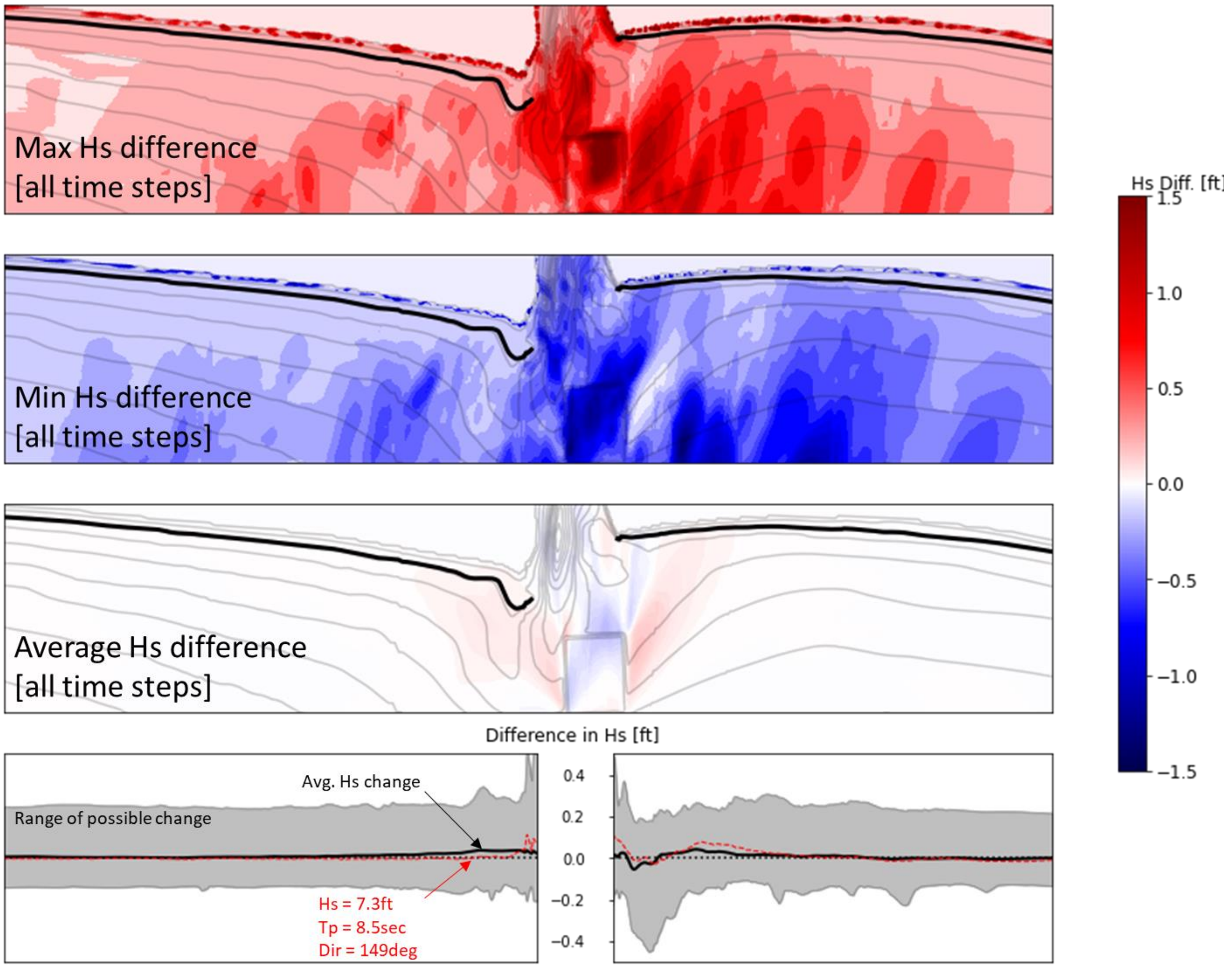

Figure 21. Top 3: maximum significant wave height difference, minimum significant height difference, and average significant wave height difference, for all time steps for USACE-A borrow source; red (blue) represents an increase (decrease) in significant wave height with respect to existing conditions. Bottom: range of significant wave height change in grey bounds, average significant height change in black, and representative cold front significant height change in red, for all time steps, at -8 ft NAVD88 extraction contour.

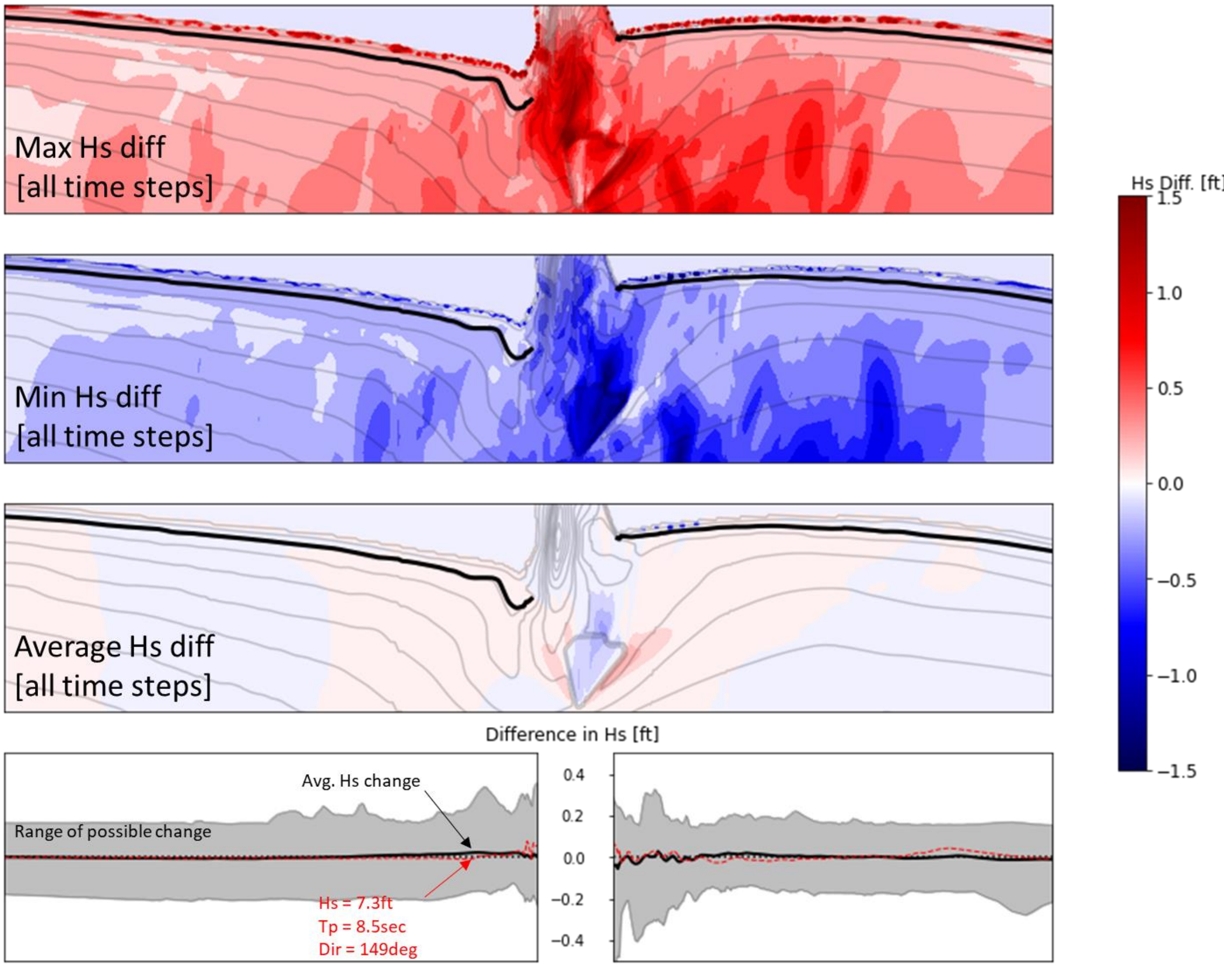

Figure 22. Top 3: maximum significant wave height difference, minimum significant height difference, and average significant wave height difference, for all time steps for VC11 borrow source; red (blue) represents an increase (decrease) in significant wave height with respect to existing conditions. Bottom: range of significant wave height change in grey bounds, average significant height change in black, and representative cold front significant height change in red, for all time steps, at -8 ft NAVD88 extraction contour.

4 Conclusions and Recommendations

The goals of this Grand Isle and Vicinity Breakwater Design – Caminada Pass Borrow Source Analysis are to:

1. Evaluate the potential changes to sediment bypassing and wave transformation induced by dredging a sediment borrow source on the eastern lobe of Caminada Pass ebb shoal
2. Develop a sediment borrow source geometry on Caminada Pass Ebb Shoal that minimizes impacts on sediment bypassing from Caminada Headlands to Grand Isle while providing sufficient sediment to complete a proposed beach nourishment.

Numerical modeling was conducted to evaluate the potential impacts of the various proposed borrow sources. The conclusions of this study are:

- Caminada Pass ebb shoal borrow sources aligned parallel to the shoal contours provide less impact to sediment bypassing than borrow sources that do not follow the ebb shoal contours
- Wave height changes show similar trends to the results seen in the sediment bypassing model with minimal changes in wave heights. However, when comparing the different borrow source geometries, the wave height gradient is reduced with shoal contour-aligned borrow sources
- When comparing USACE-A with VC11, VC11 shows minimal gradient in wave heights and improved performance in mean total transport and therefore is our recommended configuration
- Both USACE-A and VC11 borrow source configurations result in minimal changes to morphology on Grand Isle and Elmer's Island.

All dredge cuts evaluated in this study were designed using the latest 2018 bathymetric surface; thus, all dredge cuts need confirmation of top of cut elevations. We recommend verifying the adequacy of borrow source volumes using an updated hydrographic surface.

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A. Additional Borrow sources

A.1 Additional Borrow sources Geometries

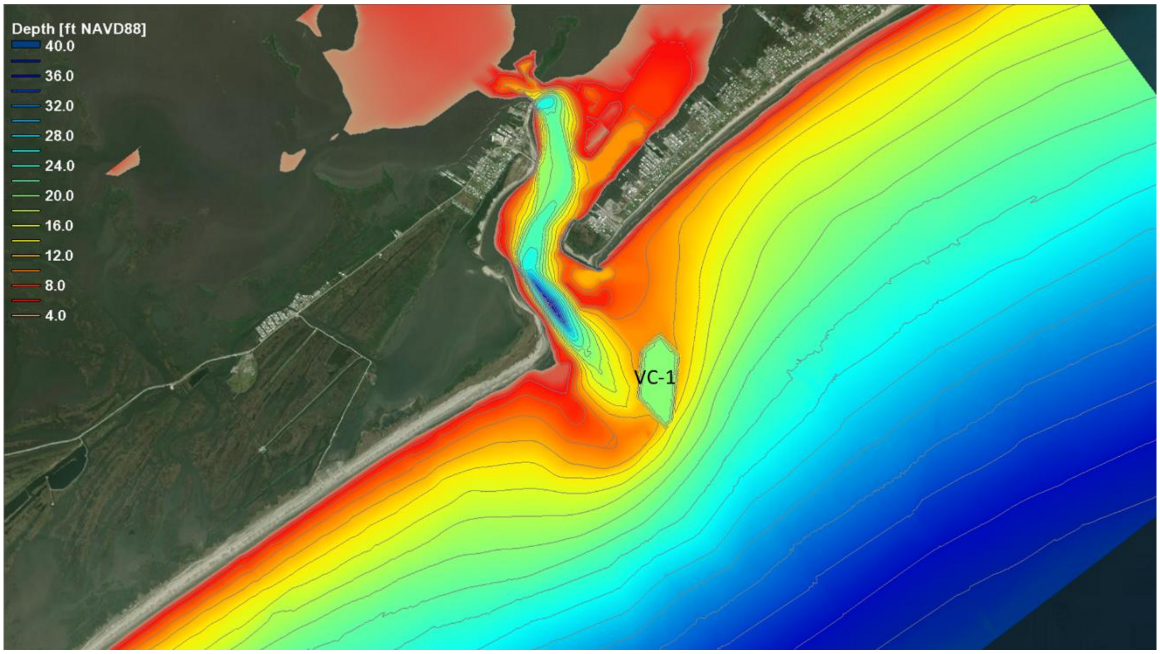

Figure 23. VC1 borrow source on 2018 bathymetric surface.

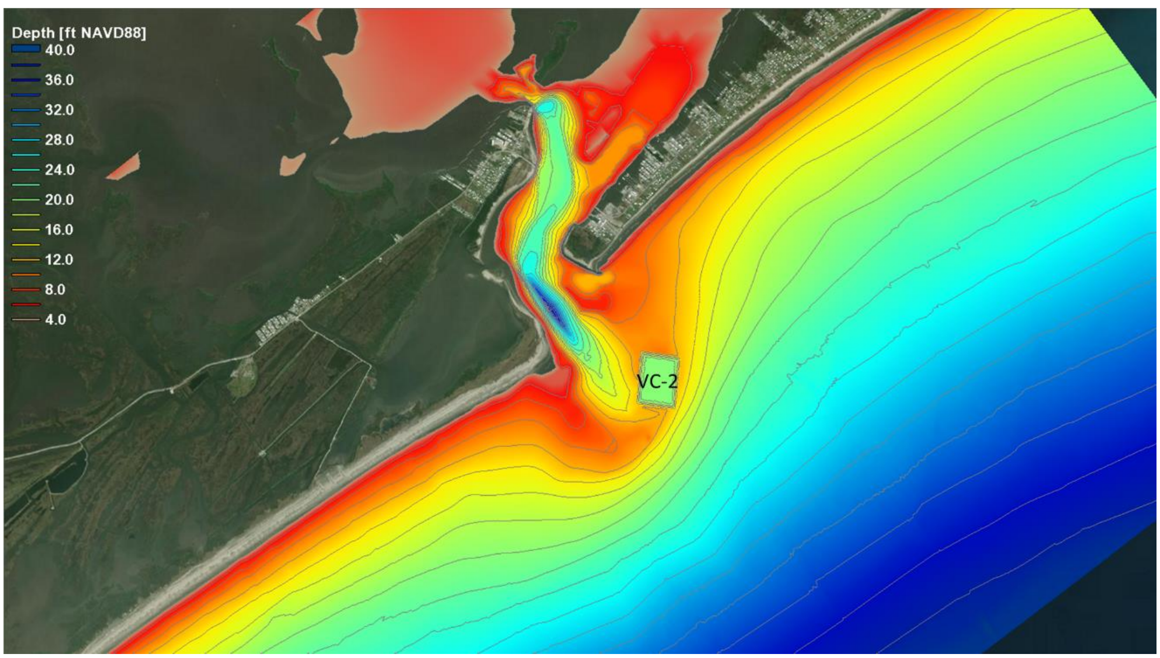

Figure 24. VC2 borrow source on 2018 bathymetric surface.

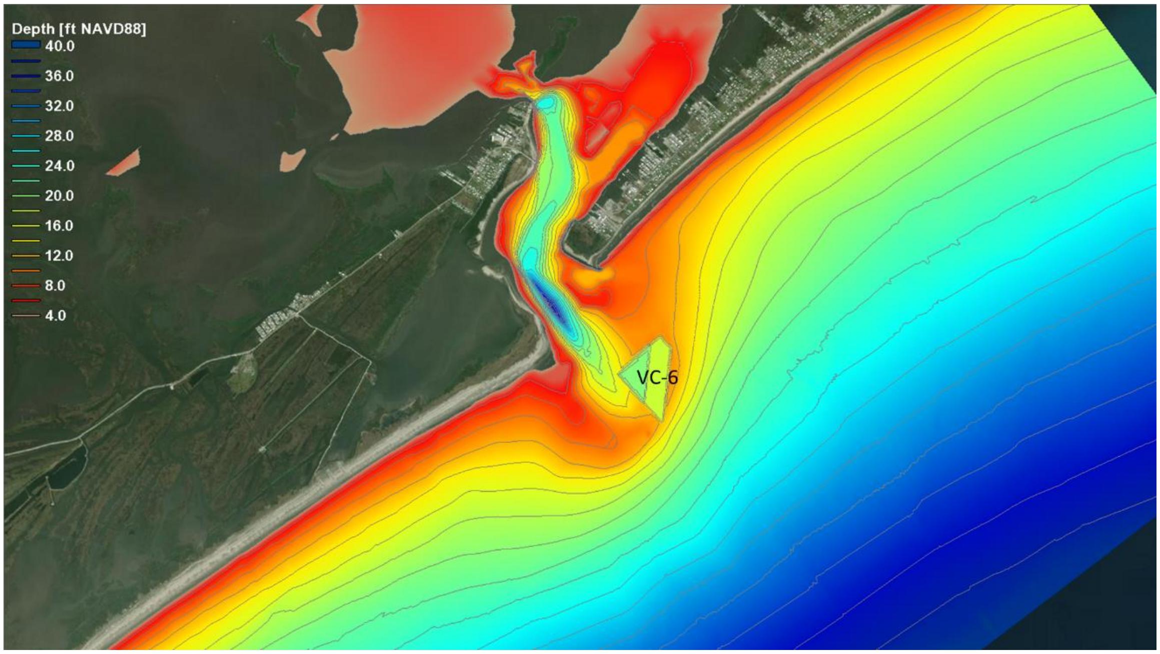

Figure 25. VC6 borrow source on 2018 bathymetric surface.

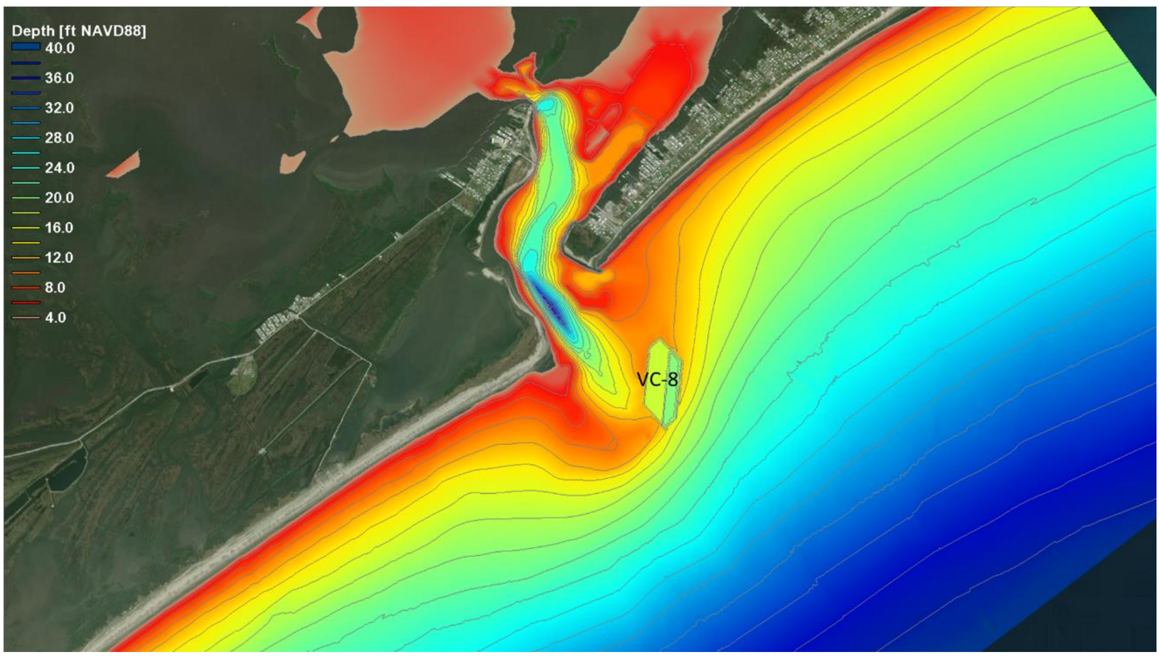

Figure 26. VC8 borrow source on 2018 bathymetric surface.

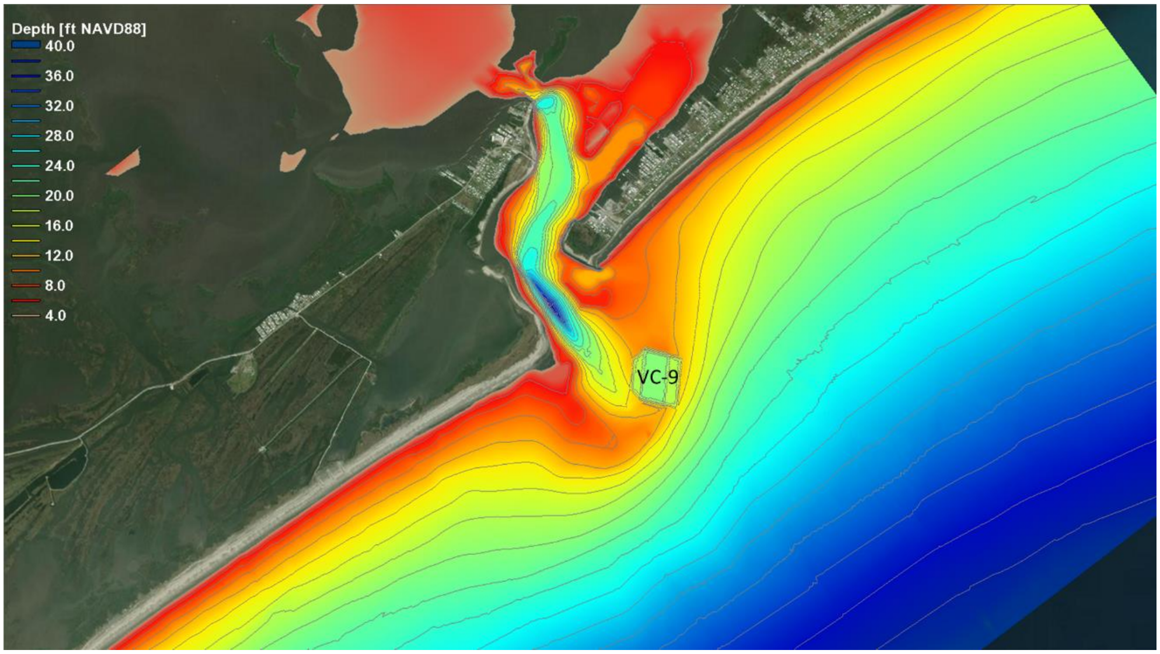

Figure 27. VC9 borrow source on 2018 bathymetric surface.

A.2 Additional Borrow sources Mean Total Transport

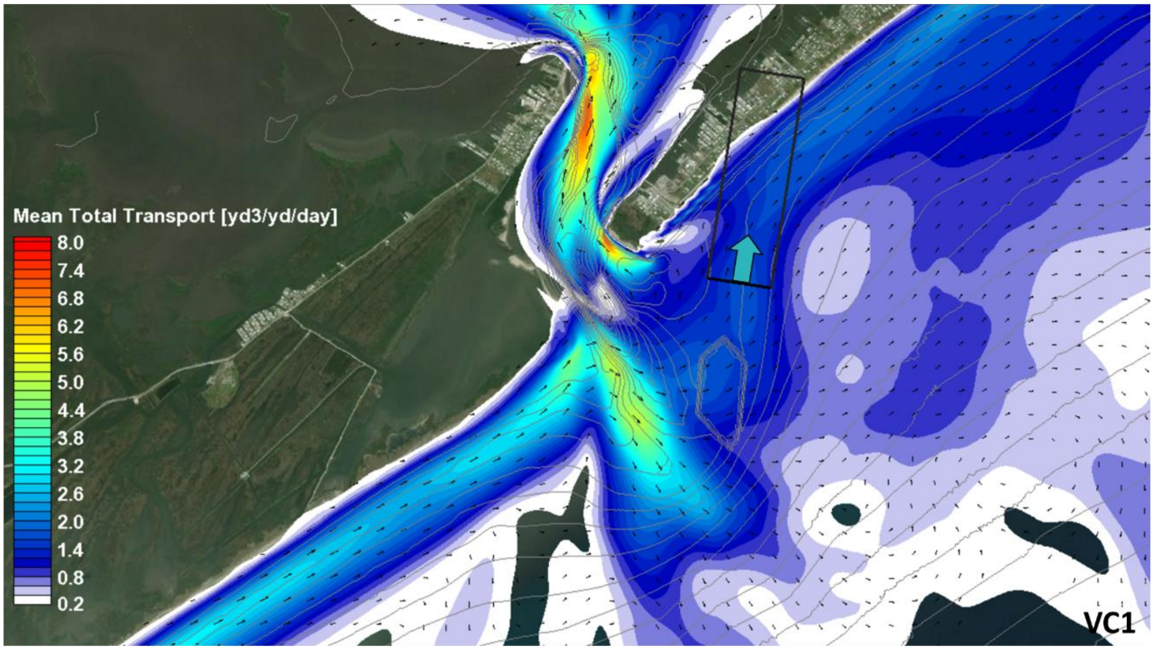

Figure 28. VC1 mean total transport at Caminada Pass

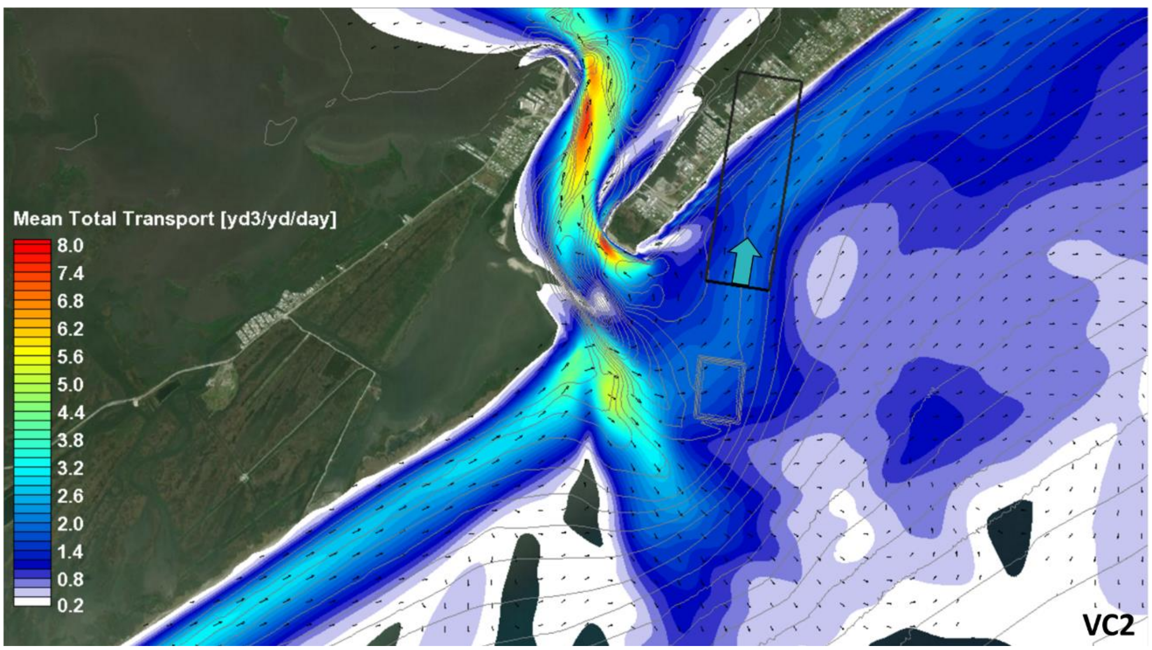

Figure 29. VC2 mean total transport at Caminada Pass

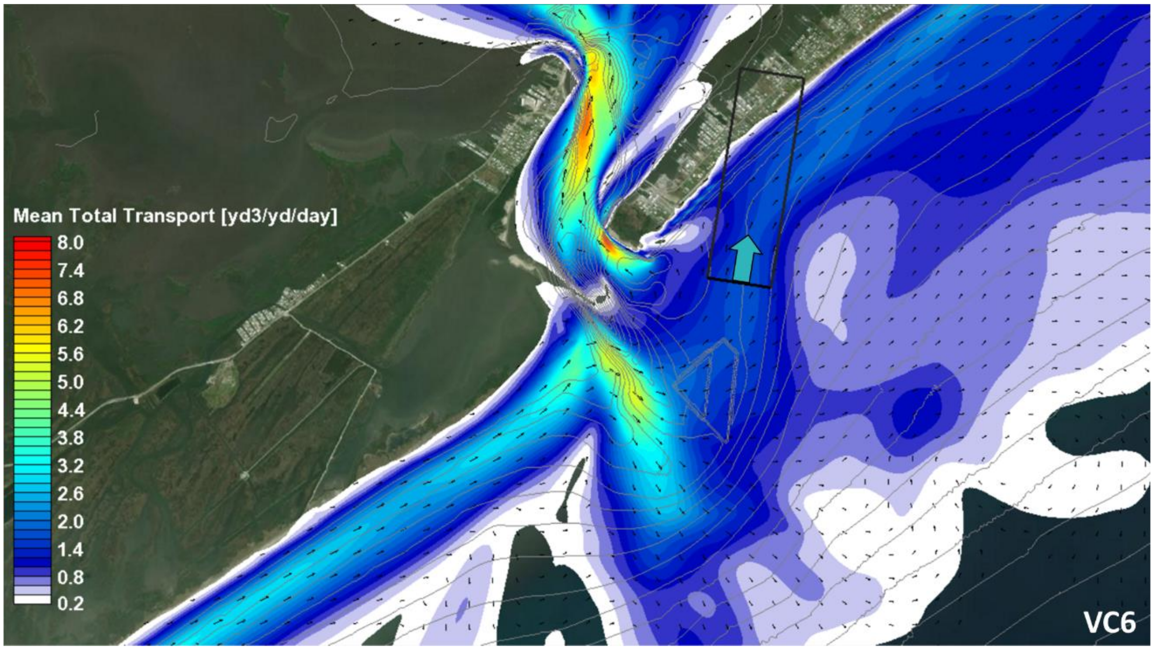

Figure 30. VC6 mean total transport at Caminada Pass

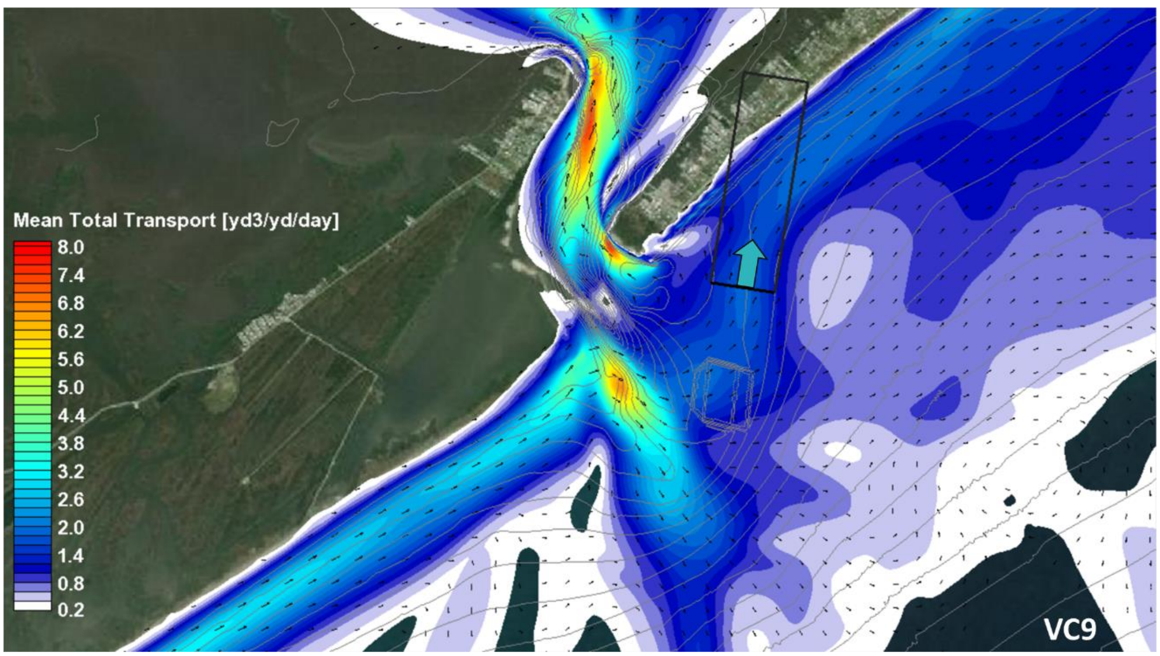

Figure 31. VC9 mean total transport at Caminada Pass

A.3 Additional Borrow sources Mean Total Transport Difference (with borrow source minus existing conditions)

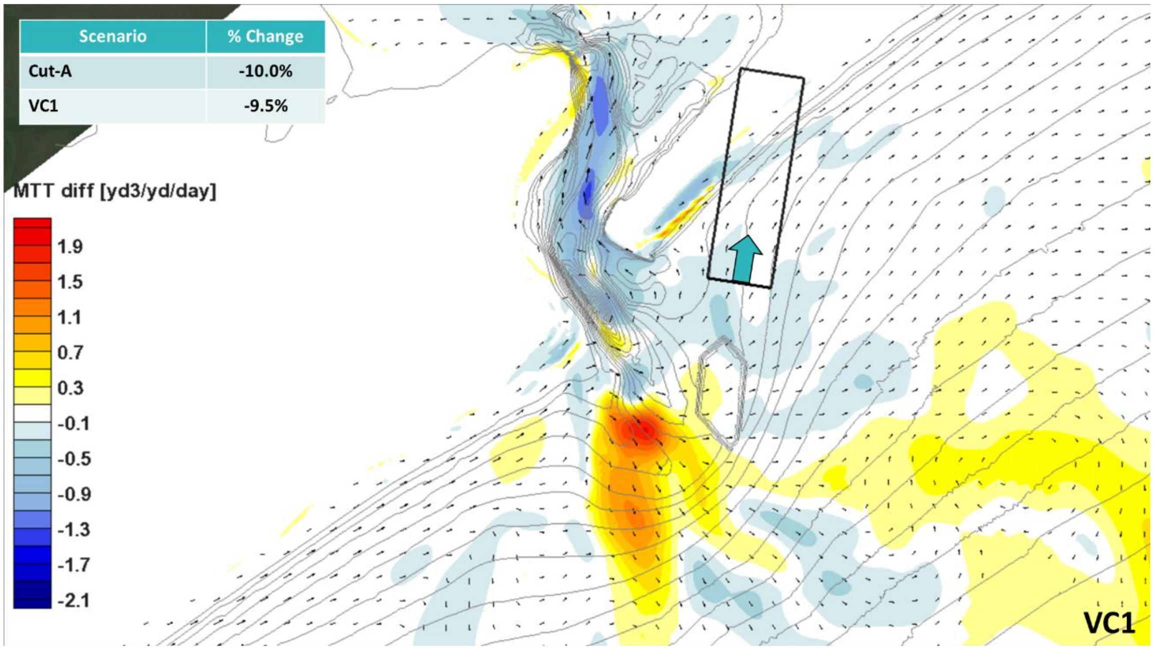

Figure 32. Mean total transport difference, VC1 borrow source minus existing.

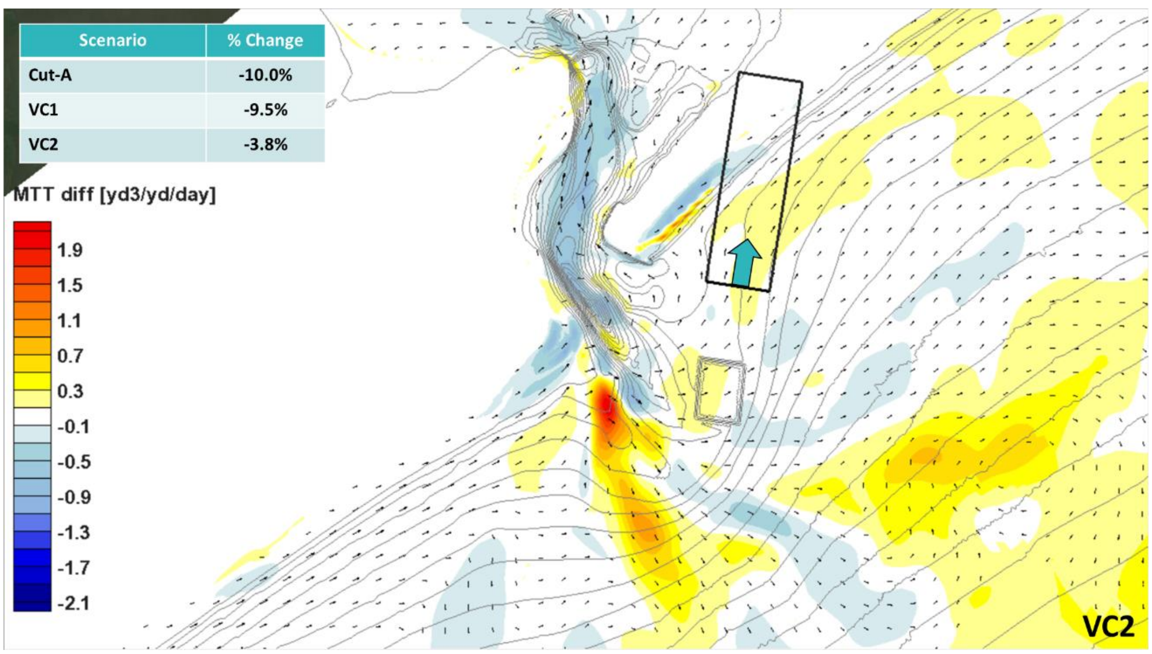

Figure 33. Mean total transport difference, VC2 borrow source minus existing.

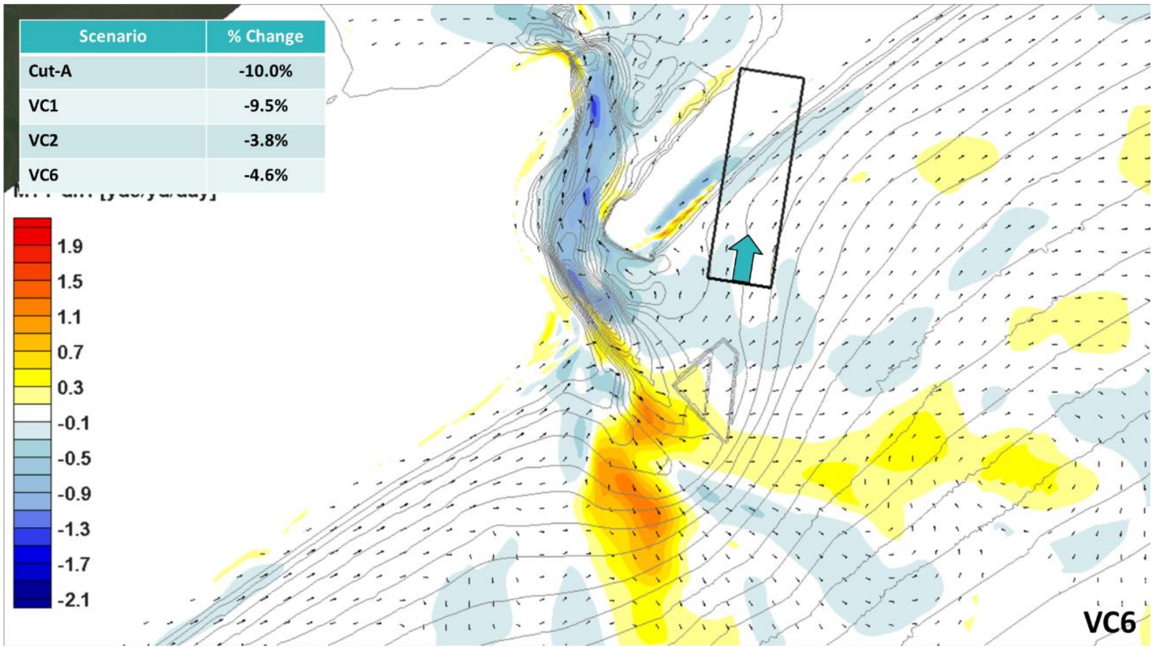

Figure 34. Mean total transport difference, VC6 borrow source minus existing.

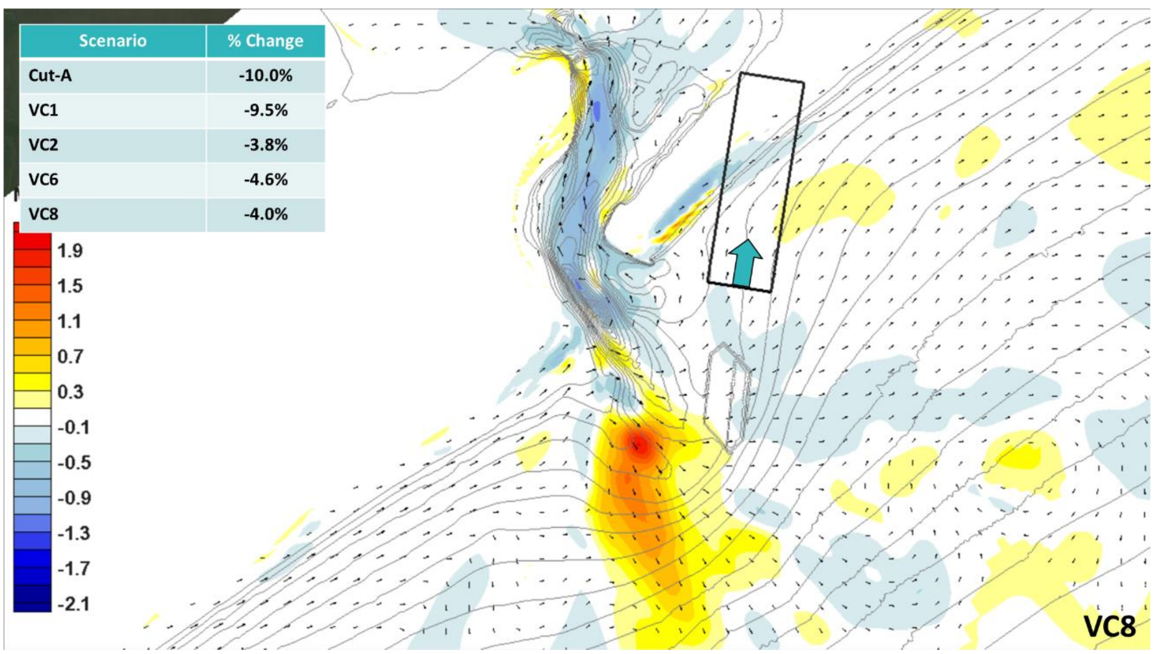

Figure 35. Mean total transport difference, VC8 borrow source minus existing.

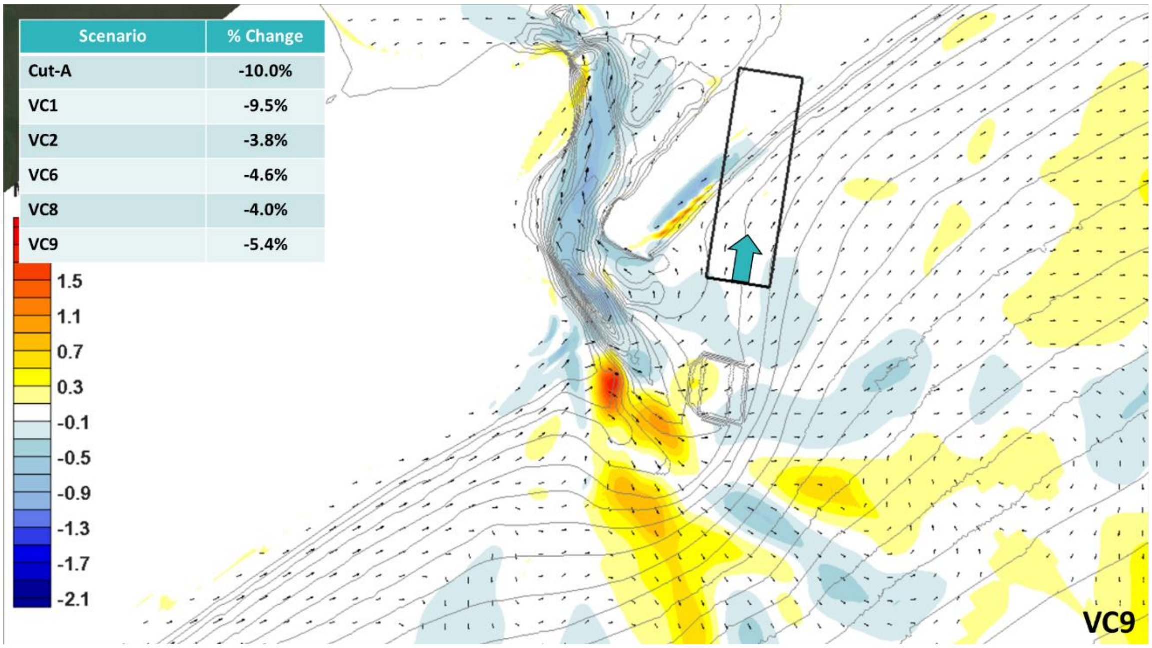

Figure 36. Mean total transport difference, VC9 borrow source minus existing.

B. Previous studies

Grand Isle Levee Dune and Beach Stabilization and Beach Nourishment Project

Coastal Processes Analysis and
Alternatives Development and Analysis Report

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Grand Isle Levee Dune and Beach Stabilization and Beach Nourishment Project

Coastal Processes Analysis and Alternatives Development and Analysis Report

May 9, 2017

Coastal Protection and Restoration Authority (CPRA)

Issue and revision record

Revision	Date	Originator	Checker	Approver	Description
0	3/15/2017	V.Curto	A.Agarwal	J.Carter	Addition of Alt Analysis to Coastal Engineering Analysis, first submittal
1	4/11/2017	V.Curto	A.Agarwal	J.Carter	Addressing of CPRA and USACE comments
2	5/9/2017	V.Curto	A.Agarwal	J.Carter	Revision to Alt 4b

Information class: Standard

Contents

Executive summary

This report has been prepared in accordance with CPRA IDIQ Contract No. 2503-15-14 for work performed under Phase II Task 3 (Coastal Engineering and Alternative Analysis) of the Scope of Work. The project objectives are to perform engineering to stabilize the levee dune area for the entire GI-01C project. This includes providing a technical assessment and re-evaluation of the Grand Isle and Vicinity, Beach Erosion and Hurricane Protection project, (including GI-01, GI-01A, GI-01B, and GI-01C), to identify any design deficiencies that may be contributing to the repeated damage of this section of the Grand Isle and Vicinity project. If possible, the design of the repair should remediate any deficiencies identified because of the technical assessment and re-evaluation.

The goal of the coastal engineering and alternatives analysis is to develop an understanding of the coastal processes and morphology along Grand Isle and determine how those conditions impact the Grand Isle Levee Dune project. As part of the coastal engineering analysis, a statistical analysis of water level, wind and wave was conducted to understand the coastal environment impacting the project shoreline. A bathymetric surface was developed to be used for various modeling analysis. Shoreline and bottom morphology change analysis was conducted to understand how the nearshore morphology has changed over time. Wave modeling transformed the waves form offshore to nearshore and was used to develop an understanding of the longshore transport along the project shoreline and to drive the shoreline morphology model. The longshore transport in conjunction with shoreline morphology formed the basis of a sediment budget along the shoreline. This understanding of the coastal processes was then used to assess offshore winds which indicate a varied offshore distribution with no predominant direction, however; stronger winds were observed from the south-southeast. Such winds result in net wave driven sediment transport toward to northeast. Wave modeling indicates that the Caminada Pass ebb shoal modifies the wave transformation near the west end of the Island so that the nearshore wave climate results in a divergent node in sediment transport despite the fact the overall net sediment transport is directed to the northeast. This divergent node results in an erosional hot spot which has led to severe erosion at that nodal point and localized accretion on the West Jetty.

Shoreline change rates analysis showed that prior to the construction of the rock revetment, the erosional hot spot lied around 0.3-0.4 miles east of West Jetty where the shoreline was eroding at almost 50 ft/yr. After the construction of rock revetment in 2013, the erosional hot spot has shifted downdrift of the revetment (0.3-0.6 miles east of West Jetty). It is expected that due to lack of sediment source due to the revetment holding sand landward of the structure, the shoreline immediately east of it will experience continued erosion.

The bottom morphology analysis illustrates the seaward migration of the Caminada Pass ebb shoal, modification of contours immediately offshore of the western end of the island, deepening of Caminada Pass channel, and updrift shift of the ebb shoal attachment point. As the Barataria Bay tidal prism increases, sediment deposition on the ebb shoal increases, which results in an increase in the Caminada Pass ebb shoal volume and an apparent seaward migration of the ebb shoal. As the ebb shoal changes, the attachment point on Grand Isle has shifted toward the West Jetty, the refraction associated with evolving bathymetric contours results in a concentration of wave energy near the revetment leading to divergent nodal transport and therefore, an erosional hot spot.

Longshore transport analysis shows that the predominant net direction of transport is along northeast for the Grand Isle shoreline. Near the Caminada Pass jetty, the transport is more bi-directional. Throughout most of the western end of the project shoreline (2 miles from the western jetty), transport rates constantly increase indicating an erosional trend along the reach. The transport pattern stabilizes for the middle section of Grand Isle (between 2-3.5 miles from western jetty) indicating the shoreline is relatively stable. The eastern shoreline shows decrease in the rate of longshore transport indicating shoreline accretion, likely due to the presence of offshore breakwaters in the area.

The coastal processes and resulting morphology of the western end of Grand Isle have eroded the beach at the Levee Dune; this erosion has impacted the Levee Dune as well. The erosional hotspot present along the western end of the Grand Isle shoreline has impacted successive Federal projects with erosion rates that have resulted in higher than the planned maintenance rate. The GI-01C project (revetment) was successful in protecting the Levee Dune in its immediate lee, but does nothing to alleviate erosion adjacent to the structure, which continues to impact the beach and Levee Dune. Successful alternatives will either provide sufficient sand to reduce the maintenance interval or work with the coastal processes to retain the sand and reduce the wave energy at the site, or some combination that can result in achieving a more stable shoreline.

A set of four main alternatives, each with sub-alternative variations, have been proposed for evaluation. The main alternatives include (1) replacing the GI-01C template, (2) larger scale beach nourishment, (3) nourishment and breakwaters, and (4) nourishment and headland breakwaters.

Alternative evaluation criteria such as performance criteria, cost, and recreational value are proposed. The performance criteria will evaluate the alternative's ability to withstand storm impact and the lifetime of the alternative relative to shoreline position, which are similar to the metrics developed by the USACE in triggering a maintenance event for the GI-01 project. Conceptual capital costs have been developed for each alternative and vary from $461K to $12M. The recreational value was considered and weighs the area of beach available over the project lifetime between maintenance intervals.

Cross-shore morphological modeling was conducted to analyze the cross-shore profile response of different alternatives for different storm events using the numerical model SBEACH. The cross-shore morphology modeling results show that Alt 1A (existing condition) will have its geotube core exposed within a year of building the dune to the GI-01C template. Beach berms placed in front of the dune (Alt 2A and Alt 2B) increase the maintenance cycle duration as the beach berm (rather than the dune) is eroded due to the storm impact. Constructing a hard structure in front of the dune (Alt 3A and Alt 4A) or berm alternative (Alt 3B and Alt 4B) reduces the wave energy impacting the shoreline leeward of the structure and therefore reduces the overall sand volume lost when compared to similar alternatives without the hard structure.

The alternatives' shoreline responses (planform morphology) were evaluated for various criteria including time to the GI-01C template (approximately the existing vegetation line), time to 100 feet landward of the GI-01A template (for applicable alternatives), and downdrift erosion. The evaluation criteria show that structures are necessary to slow the high rates of erosion along the project shoreline. The alternatives were evaluated by computing the time it takes the shoreline to retreat to the renourishment trigger (the vegetation line). The GI-01C template renourishment interval is approximately 1.5 years, beach nourishment increases the interval to 7 to 10 years, while structures increase the interval to 3 to 13.5 years. Structures cause downdrift erosion to the east of the project site when compared to the FWOP scenario. The costs to mitigate this downdrift erosion were included in maintenance costs.

Capital and maintenance costs were developed assuming a 50-year project lifespan for each alternative. Capital costs range from $460k to $12M, while total 50-year lifetime costs range from $57M to $186M.

The recreational value of each alternative was also assessed by calculating the beach acreage at years 1-10 of the project life. Alternatives with hard structures showed a slower rate of decrease in the beach acreage than those with beach fills only.

It is Mott MacDonald's opinion that the best performing alternative is Alt 3B which is breakwaters and GI-01A beach fill that has a moderate capital cost ($7.9M), long maintenance interval (13.5 years), and moderate total life-cycle costs at $57M. Breakwaters are proven to be effective on Grand Isle, while headland breakwaters have not been employed in Louisiana (they have, however, been shown effective on the Gulf and Atlantic coasts of Florida). The next best alternative is Alt 4B_v1 (headland breakwaters with dune fill and beach fill) which has a low capital cost ($6.4M), moderate maintenance interval (9.8 years) and a relatively low total life-cycle costs at $59M. Both alternatives provide reasonable access to recreational beach through their lifetime.

1 Coastal Processes Analysis

The objective of the coastal processes analysis is to develop an understanding of the coastal processes and morphology along Grand Isle. To do this, we conducted a statistical and extreme value analysis of coastal processes, morphology analysis, wave modeling, circulation modeling, and sediment budget. This understanding will be used as the basis for assessing and re-evaluating the Grand Isle Federal Levee project's alternatives development and analysis tasks.

1.1 Project Location and History

Grand Isle is located in Jefferson Parish, Louisiana as shown in Figure 1. The island is bounded by Barataria Pass on the north and Caminada Pass on the south. For a detailed history of the project site and a summary of projects executed along the project shoreline, please refer to Coast & Harbor Engineering (CHE, 2005) and CHE (2016).

For more than 60 years, the Grand Isle shoreline has been subjected to multiple projects and hurricane events as shown in Figure 2. Major works conducted since 1984 are based on the Federal Grand Isle Vicinity project. Based on the Grand Isle coastal engineering history from 1951 to 2015, dune replenishment or dune rehabilitation has occurred once every 5.8 years, on average.

Figure 1. Grand Isle project vicinity.

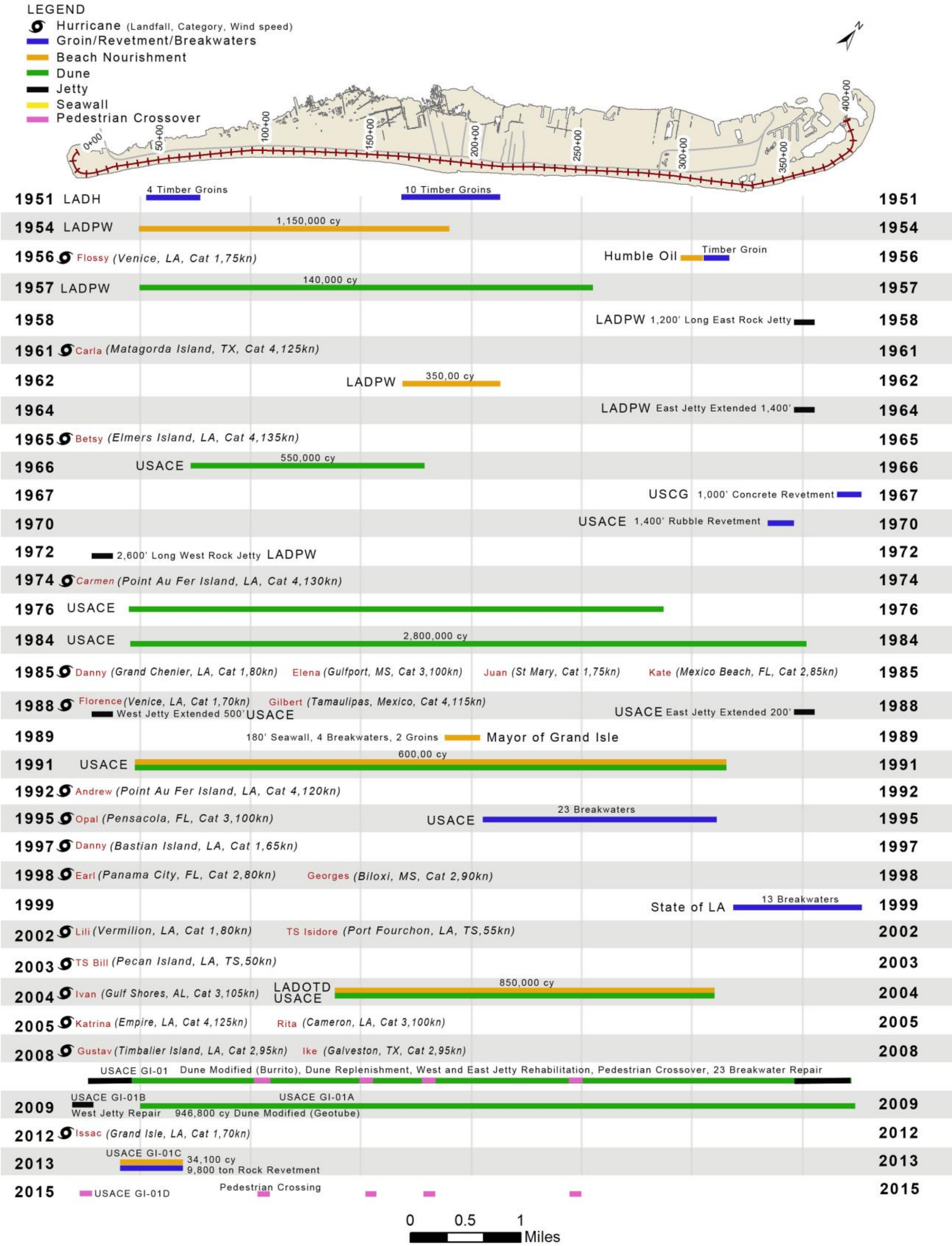


Figure 2. Engineering projects and hurricane history at Grand Isle, LA.

1.2 Statistical and Extreme Value Analysis

Statistical and extreme value analyses were conducted on the wave, wind, and water level data to develop an understanding of these coastal processes and how they impact the project shoreline during normal day-to-day conditions as well as during extreme events. Data were collected from available sources (WIS, NHC, and NOAA) near the project site and are shown in Figure 3.

Figure 3. Data sources and locations used for coastal processes analysis.

1.2.1 Tidal Elevation and Water Levels

Tidal elevations were obtained from the NOAA station ID 8761724, Grand Isle (NOAA, 2015) which is located within the project vicinity referenced to the latest tidal epoch; these elevations are shown in Table 1.

Table 1. Tidal elevations at location near the project site at NOAA station 8761724 Grand Isle based on the 2007-2011 epoch.

Water Surface Elevation	[ft NAVD88]
Mean Higher High Water (MHHW)	1.71
Mean high Water (MHW)	1.70
Mean Sea Level (MSL)	1.19
Mean Lower Hater (MLW)	0.66
Mean Lower Low Water (MLLW)	0.65

Statistical analysis was conducted for NOAA station 8761724 water level data only up to the 25-year return period due to the unreliability of the instrument to record higher water levels during storms. For return periods higher than 50 year, a previous study (Resio, 2007), estimated the surge levels along the Louisiana coastline by a complex joint-probability distribution analysis of empirical data for the five main hurricane parameters (maximum wind speed, storm path, radius of storm from center to maximum wind speed, central pressure of the storm, storm speed) and numerical simulation of hurricane events (152 events for a specific area of Louisiana coast) which included an estimation of uncertainties in the simulation. The uncertainties in simulation results were estimated by re-sampling of the stage-frequency relationships through a boot-strapped Monte-Carlo method. Based on this methodology, preliminary analysis from Resio (2007) provides the storm surge as a function return period for Caminada Pass, shown in Table 2.

Table 2. Extreme surge plus tide (storm tide) near the project site.

Return Period [yr]	Storm Tide [ft NAVD 88]
2	4.0
5	4.6
10	5.0
20	5.3
25	5.4
50	8.8
75	9.8
100	10.7
500	13.7

1.2.2 Wind

Statistical and extreme value analyses for Grand Isle winds were performed using two different data sources: WIS station 73130 and National Hurricane Center (NHC) database.

WIS

Statistical analyses for Grand Isle winds was performed using wind data from Wave Information Studies (WIS). WIS data are generated by the Coastal & Hydraulics Laboratory WIS model (USACE, 2010). Offshore wind and wave data was downloaded from the WIS Station 73130 (as shown in Figure 3), which provides hindcast wind and wave climatology data from 1980 to 2014. WIS data has been shown to reproduce measured conditions with good accuracy (USACE, 2010). To describe the wind characteristics around Grand Isle, a wind rose was developed using the historical WIS wind data. Wind roses illustrate the frequency of occurrence of wind events for 16 directional bins at 16 points of the compass for various wind speeds. The wind rose is shown on Figure 4.

The wind rose indicates a varied offshore distribution, with no predominant direction. The highest wind speeds are observed coming from the northeast and northwest directions; such wind speeds are associated with strong winter cold fronts that pass through the area. For winds coming from onshore directions, more energetic winds come from south-southeast to south direction compared to the east-southeast to east-northeast directions, indicating an overall potential of net longshore transport towards northeast along the Grand Isle shoreline.

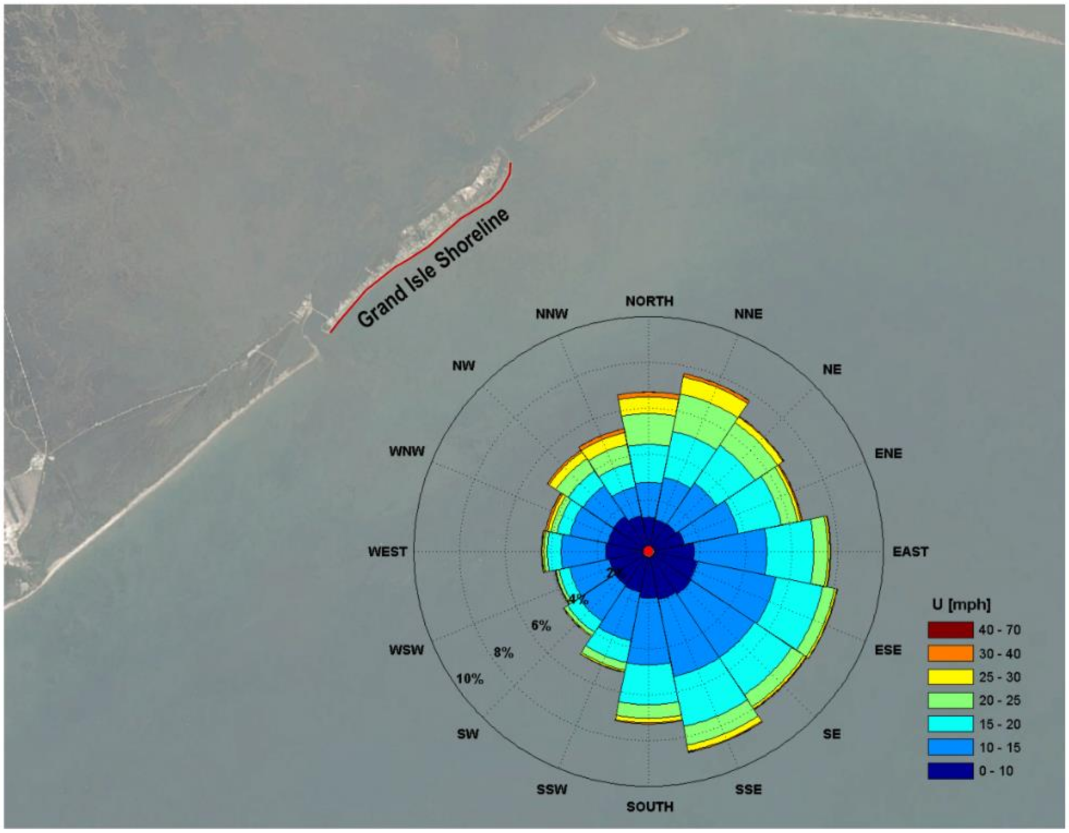


Figure 4. Grand Isle wind rose from WIS station 73130.

NHC

To provide a more comprehensive analysis of extreme wind speed, Mott MacDonald performed an extreme value analysis on all hurricanes influencing the project site using methodology consistent with the National Hurricane Center Risk Analysis Program (HURISK) (NOAA, 1987). Hurricane tracks, wind speed, and pressure data were obtained from the National Hurricane Center (NHC) database to perform this extreme wind analysis. The NHC storm database spans from 1842 to 2014. Maximum wind speeds were extracted for all storms passing within 75 nautical miles of the project site during the data record (total of 86 storms). An extreme value distribution was fit to these maximum wind speeds and the results are shown in Table 3. It should be noted that the NHC database reports 10-minute average wind speeds, so these wind speeds were converted to 2-minute average wind speeds using methodology provided in the Coastal Engineering Manual (USACE, 2002) for comparison to point gauge data. The 10-minute winds were used over the 2-miunte winds to achieve a fully developed sea state in the static SWAN model (discussed in Section 1.5.1).

Table 3. Extreme wind speeds near Grand Isle based on NHC data.

Return Period [yr]	Wind Speed [mph] 2-min averaging	Wind Speed [mph] 10-min averaging
5	93.7	84.1
10	118.7	97.7
15	131.8	106.6
20	140.7	118.4
25	147.3	126.4
50	166.9	132.3
75	177.8	150.0
100	185.3	159.8

1.2.3 Waves

Similar to WIS wind analysis, a wave rose from WIS Station 73130 was developed and is shown in Figure 5. The predominant offshore wave direction is southeast to south-southeast. This accounts for approximately 60% of all offshore waves. Similar to wind roses, the wave rose also shows a more energetic environment from south-southeast to south compared to east-southeast to east directions indicating a potential for net longshore transport towards the northeast along the Grand Isle shoreline. The time series WIS wave data was analyzed to produce the extreme value wave statistics presented in Table 4.

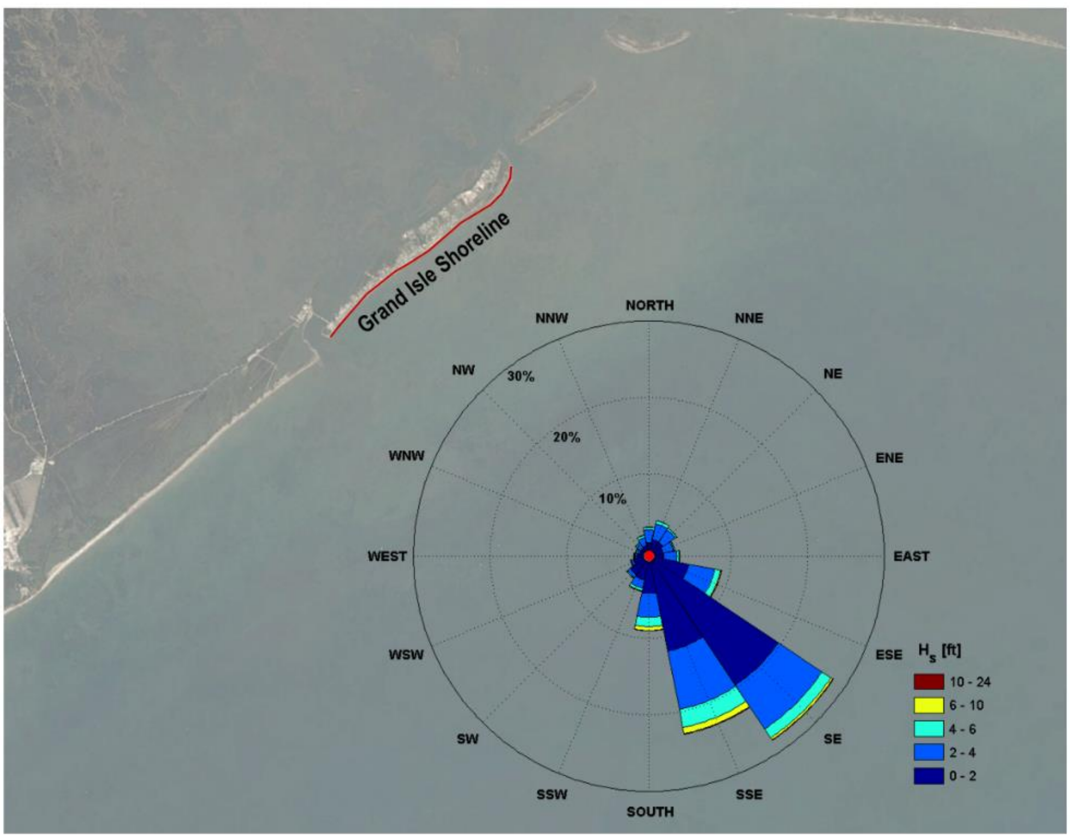


Figure 5. Grand Isle wave from WIS station 73130.

Table 4. Extreme wave heights and periods from WIS station 73130.

Return Period [yrs]	Hs [ft]	Tp [sec]
1	10.7	9.1
2	14.2	10.6
5	18.3	12.3
10	21.2	13.5
25	24.8	15.0
50	27.4	16.1
100	30.0	17.2

1.3 Sea Level Rise

Two different Sea Level Rise (SLR) projections, USACE and Intergovernmental Panel on Climate Change (IPCC), have been accounted in this study to assist assessing SLR in Grand Isle. The USACE projections were obtained from http://www.corpsclimate.us/ccaceslcurves.cfm (USACE, 2014) with data pertinent to Grand Isle, LA NOAA gauge 8761724. USACE includes three different projections:

- Low: historic rate of sea-level change extrapolated from NOAA tidal gauge record
- Intermediate: from the modified NRC Curve I with the local rate of vertical land motion (VLM) added
- High: from the modified NRC Curve III with the local rate of vertical land movement added

The IPCC projection was obtained from IPCC latest report dated 2013 (IPCC: Church, et al., 2013). Four different scenarios are provided by the IPCC. For this study, only the worst-case scenario, RCP8.5 medium, was selected. Because the IPCC projection (RCP8.5) only consists of a global component to SLR, the VLM was added to the predictions. The VLM rate was obtained from Estimating Vertical Land Motion from Long-Term Tide Gauge Records, NOAA Technical Report (Zervas, Gill, & Sweet, 2013) following the Grand Isle, LA data.

The SLR projections provided in this report are given with respect to 2017. Table 5 provides the 50-yr and 2100 (upper limits of SLR projections) SLR projections. The SLR projection curves are shown on Figure 6.

Table 5. Projected sea level rise values in the region of the project site relative to 2017.

Year	USACE Low [ft]	USACE Int [ft]	USACE High [ft]	IPCC (RCP8.5) + VLM [ft]
2017	0	0	0	0
2067	1.5	2.0	3.4	2.3
2100	2.5	3.5	6.6	4.3

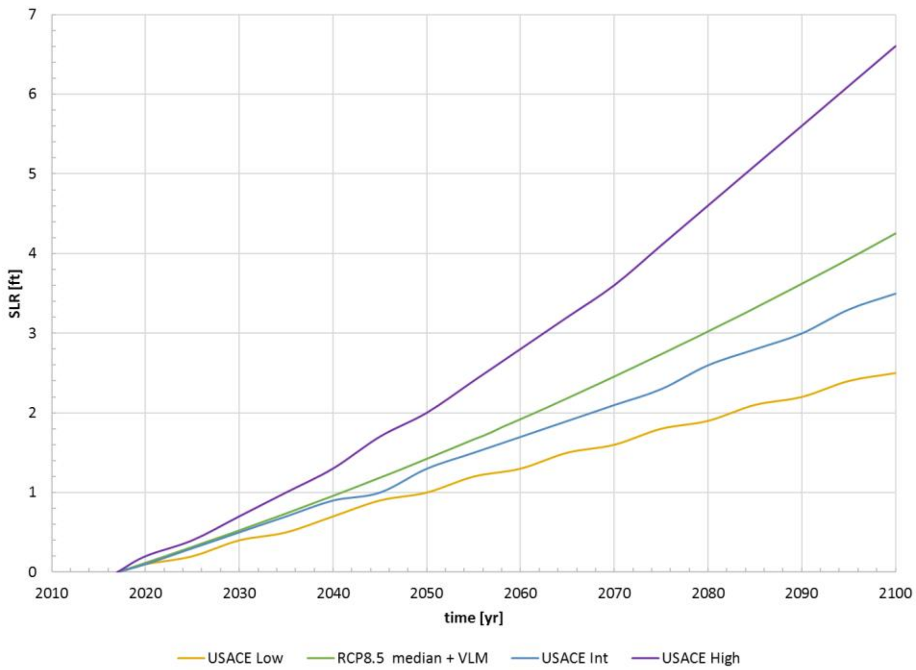


Figure 6. Projected sea level rise values in the region of the project site relative to 2017.

As shown in Figure 6, USACE high curve gives the highest SLR; this maximum seems to be overconservative when compared to the rest of the projections. The majority of the SLR projections fall within the range bounded by the USACE low and the RCP8.5 + VLM curves. This study recommends choosing the IPCC RCP8.5 + VLM as a conservative estimate.

Sea level rise (SLR) projections have been included in this report to inform the potential change in sea level. SLR was not accounted on the conceptual alternative design and analysis sections (reference pertinent sections). However, the change in SLR is not expected to affect the selection of the preferred alternative since without further analysis, the relative performance of each alternative with respect to each other is assumed to be independent of SLR. On the other hand, SLR is expected to have an effect on the performance, lifetime, and cost of each alternative, and it is recommended to be investigated in the final design.

1.4 Bathymetric Surface

A bathymetric surface model that covers a wide region was developed to obtain a consistent bathymetry set which is required for circulation and wave modeling. Two different bathymetry sets that were used include: (1) BICM-2 2015 Regional Bathy Survey (CHE, 2016) and (2) bathymetric surface created by Grand Isle Barrier Shoreline Stabilization Study Task 2 - Summary of existing Data and New Field Data Collection Plan (CHE, 2005). The latter was almost exclusively used for bathymetry offshore of the 2015 data, and was comprised of Coastal Relief Model (CRM) data. The 2015 BICM data was smoothed prior to merging with 2005 data.

The preliminary 2016 survey was employed in the morphology analysis (Section 1.7) but at present has not been included in the wave and circulation modeling (Sections 1.5 and 1.6). Once the 2016 is completed, quality checked and approved, the coastal engineering analysis will be updated to further develop the understanding of coastal processes; specially how they have changed over time.

1.5 Wave Modeling

Wave modeling was conducted to assess the wave conditions at the project site. The nearshore wave modeling is used to develop an understanding of (1) the typical nearshore wave climate, which drives longshore transport and shoreline morphology models, and (2) extreme wave climate which is employed in design.

Wave modeling was conducted to transform waves from offshore to the project shoreline. Wave modeling was conducted using the SWAN model. SWAN (Delft University of Technology, 2012) is a 2-D, spectral (phase-averaged) wave transformation model that can be used to generate wind-waves and transform wave conditions to the nearshore project area. The wave modeling grid is 17 miles in length (shore parallel) and 11 miles in width (shore normal). It uses variable spacing where larger grid cells are used in the offshore, deeper water and on areas that do not influence waves at the site, and resolution is increased in the nearshore and at the project site. The bathymetric surface as well as the grid extents and spacing are shown in Figure 7.

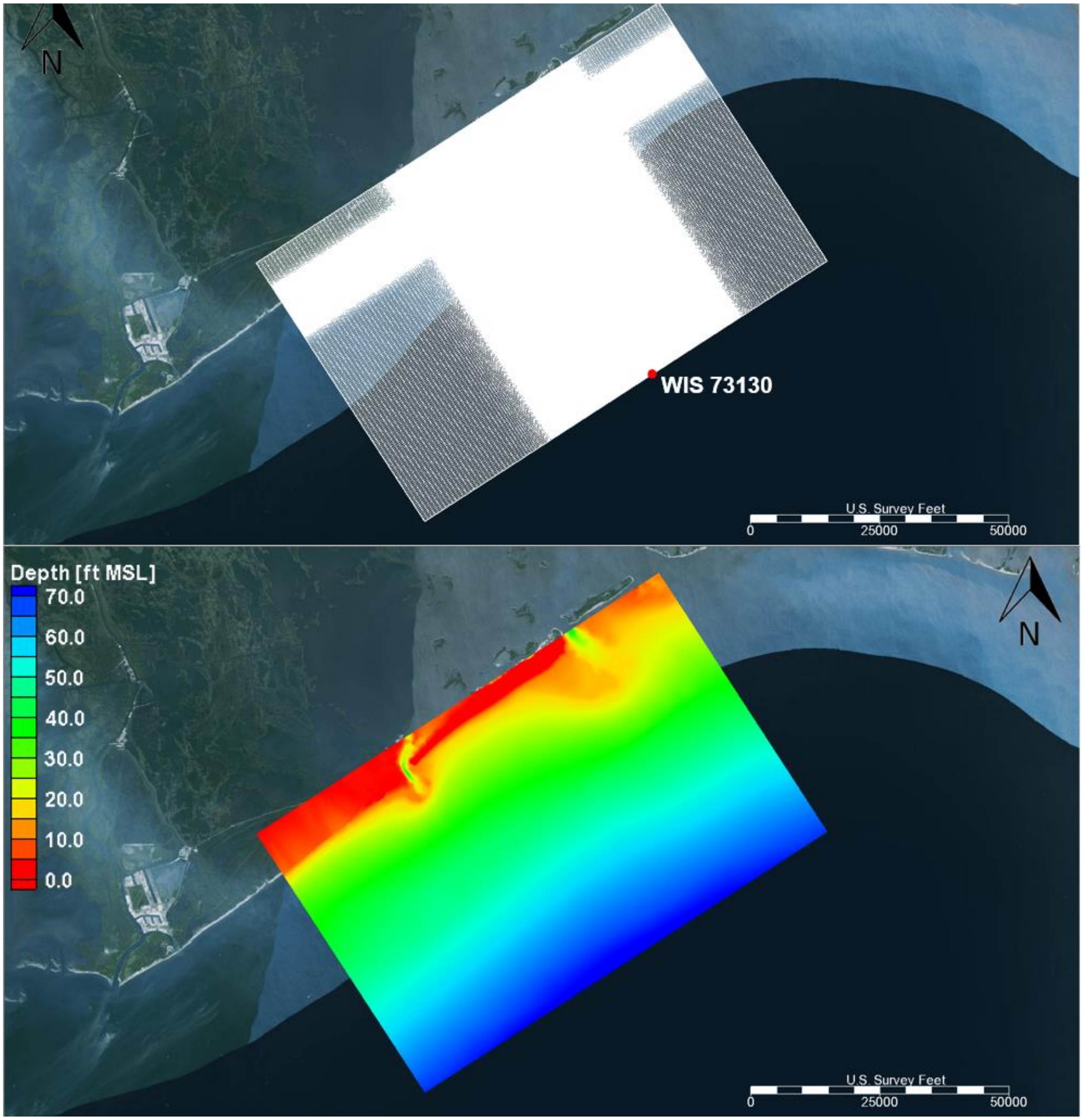


Figure 7. Grid extents with grid cell spacing shown with white lines (top) and bathymetric surface developed for wave modeling (bottom).

1.5.1 Typical Wave Conditions

To understand the typical nearshore wave climate at Grand Isle, a series of wave model runs were conducted using WIS wave and wind data. The WIS data ranging from 1980 to 2014 was filtered based on the wind and wave directions that can propagate waves towards Grand Isle. A statistical downscaling method was used to sort the filtered wave and wind inputs from the WIS gauge into bins designed to capture the relevant combination of conditions representative of the input wave climate. This includes the joint distribution of wind speed, wind direction, wave height, wave period, and wave angle. Distribution tables showing the relationship between Hs & Tp and Hs & Direction at the offshore WIS location, are shown below in Table 6 and Table 7, respectively along with the binning scheme. Table 8 shows the binning scheme and the frequency of wind speeds occurring within each bin, as well as the offshore relationship between wind speed and direction.

Table 6. Hs vs. Tp relationship, shown as percent occurrence within each bin. The range of each bin is shown in the table header.

Tp [s]	2.43 - 4	4.1 - 5	5.1 - 6	6.1 - 12	12.1 - 19	Sum
Hs [ft						
0 - 2	21.84	13.32	14.08	7.25	0.03	56.52
2.1 - 3	8.09	4.15	3.50	5.40	0.02	21.17
3.1 - 4	0.84	4.65	2.33	4.01	0.02	11.84
4.1 - 8	0.01	1.90	2.16	5.62	0.10	9.79
8.1 - 12			0.01	0.51	0.05	0.57
12.1 - 24				0.07	0.05	0.12
Sum	30.78	24.02	22.08	22.85	0.27	**100.0**

Table 7. Hs vs. Direction relationship, shown as percent occurrence within each bin. The range of each bin is shown in the table header.

Dir	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Sum
Hs [ft]																	
0-2	1.13	1.53	1.56	1.19	1.26	4.73	19.95	12.32	4.25	2.48	2.20	1.34	0.81	0.54	0.51	0.71	56.52
2.1-3	0.99	1.33	1.31	0.93	1.10	2.16	4.38	4.71	1.83	0.66	0.28	0.18	0.15	0.17	0.36	0.63	21.17
3.1-4	0.57	0.74	0.57	0.43	0.58	1.15	2.20	2.79	1.22	0.37	0.13	0.09	0.11	0.15	0.30	0.43	11.84
4.1-8	0.36	0.40	0.22	0.14	0.32	0.81	1.53	2.89	1.52	0.42	0.15	0.12	0.16	0.19	0.25	0.30	9.79
8.1-12	0.00		0.00	0.00	0.02	0.04	0.10	0.21	0.15	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.57
12.1-24			0.00	0.00		0.00	0.04	0.05	0.01	0.01	0.00						0.12
Sum	3.06	4.00	3.67	2.70	3.28	8.90	28.20	22.97	9.00	3.96	2.76	1.73	1.23	1.05	1.42	2.07	100.0

Table 8. Wind Speed vs. Direction relationship, shown as percent occurrence within each bin. The range of each bin is shown in the table header.

Dir	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Sum
U (mph)																	
0-8	0.68	0.57	0.56	0.63	0.76	0.80	0.87	0.83	0.82	0.72	0.68	0.70	0.76	0.74	0.81	0.76	11.67
8.1-16	2.40	2.83	3.17	3.55	4.69	5.13	4.94	5.01	4.31	3.25	2.91	2.71	3.00	2.70	2.58	2.04	55.23
16.1-24	2.48	3.14	2.67	2.15	2.07	2.04	2.04	2.45	1.86	1.02	0.57	0.45	0.54	0.62	1.14	1.57	26.81
24.1-40	1.15	1.11	0.45	0.25	0.17	0.17	0.18	0.39	0.29	0.14	0.07	0.06	0.12	0.28	0.55	0.84	6.21
40.1-80	0.00	0.01	0.01	0.00	0.01	0.01	0.01	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.08
Sum	6.71	7.66	6.86	6.58	7.69	8.15	8.05	8.69	7.27	5.13	4.23	3.92	4.43	4.34	5.08	5.22	100.0

Each bin shown in the tables above represents a unique combination of the two parameters investigated. A series of 3,388 unique cases representing a combination of wave height, wave period, wave direction, wind speed, and wind direction bins were created by using the bins shown above. The 3,388 cases represent 86% of all the cases within the WIS time history where either the waves or winds were traveling onshore (14% of the cases have both winds and waves travelling offshore and therefore are not relevant for wave transformation to the project site). The SWAN model run results (wave height, wave period and wave direction) at nearshore locations corresponding to each of the 3,388 input cases were used as a transfer function to recreate the full WIS station time history (by matching each WIS time series incidence to the matching scenario from the 3,388 input cases), which spans from 1980 to 2014, at a series of nearshore extraction points. This nearshore wave time series was then used to calibrate the shoreline morphology model described in Section 1.7.3.

Representative wave model results are shown in Figure 8. This figure shows the wave height as color contours and wave direction as arrows. A representative case was selected from a statistical analysis performed on WIS station 73130. Based on this analysis, the month of December typically produces the most energetic offshore wind and wave environment. The SWAN model was forced with a wave height of 4.3 feet, peak period of 6.43 seconds, and a wind speed of 23.2 mph. These values represent one standard deviation above the mean wave and wind condition during the month of December. Both the winds and waves were directed at angles of 100 degrees from true north and 190 degrees from true north. These directions represent approximately 45 degrees east and west of shore normal. The wind and wave forcing directions were selected to represent the theoretical maximum longshore transport condition, which occurs when waves are directed 45 degrees from shore normal.

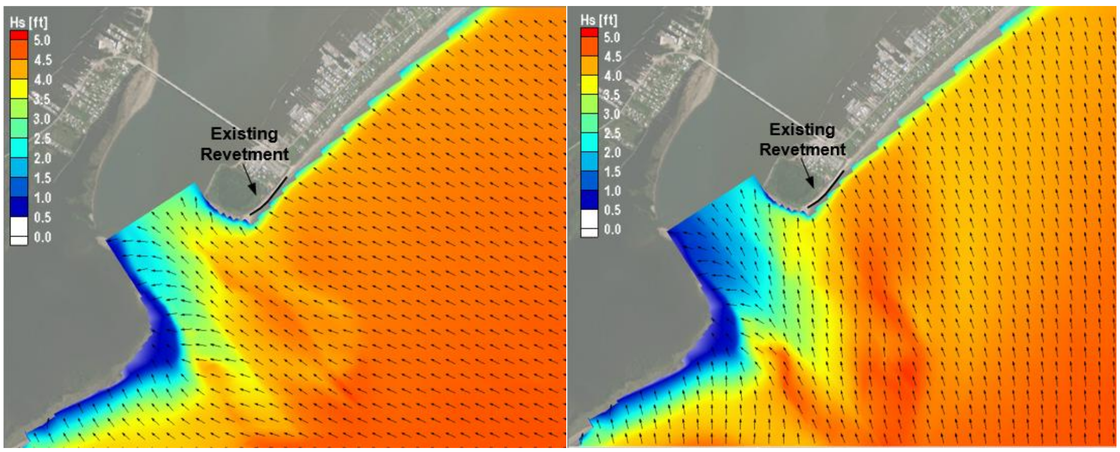


Figure 8. Representative wave cases showing transport with wind and wave forcing at 100 degrees from true north (left), and 190 degrees from true north (right). Note the more energetic wave conditions and change in wave direction near the perimeter of the Caminada Pass ebb shoal.

The results shown in Figure 8 clearly show the increased wave shoaling and refraction near the Caminada Pass ebb shoal. To further investigate the effect of the Caminada Pass ebb shoal on nearshore wave directions, nearshore wave roses were developed from the dynamic downscaling process described earlier in this section. Wave roses were developed seaward of the existing revetment, within the Caminada Pass ebb shoal, and further east from the Ebb shoal. Figure 9 shows these nearshore wave roses.

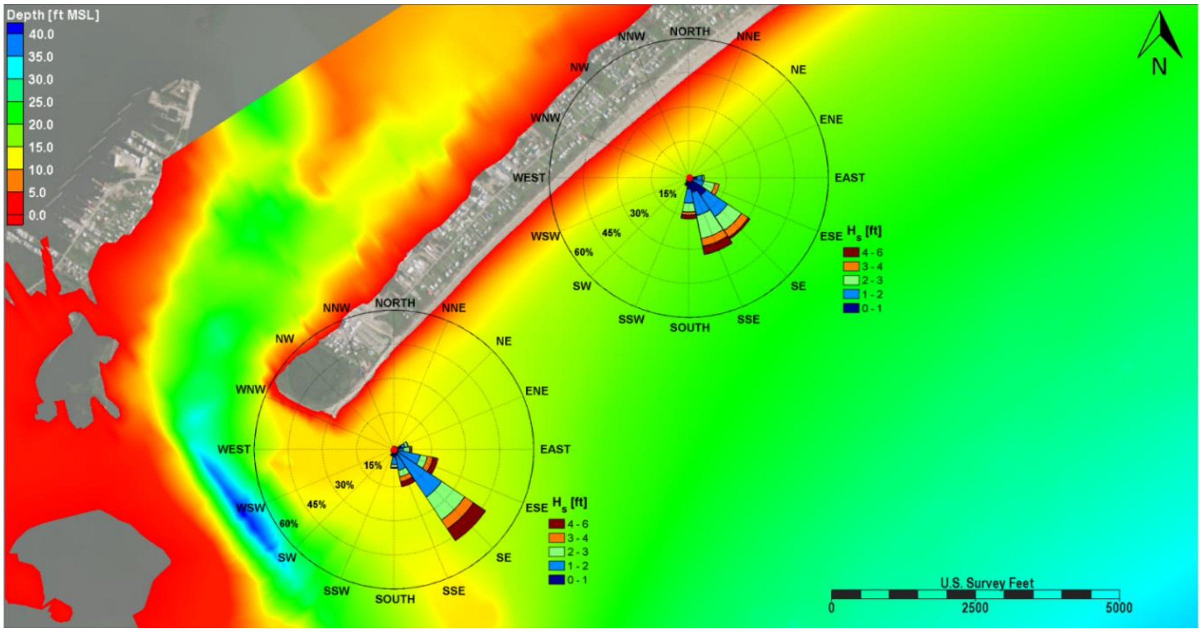


Figure 9. Bathymetry plot and nearshore wave roses at two extraction points. The red dot in the center of the wave rose represents the extraction point.

The western wave rose along the Caminada Pass ebb shoal shows a higher concentration of east-southeast to east-northeast waves. Waves from this direction would direct longshore transport southwest towards Caminada Pass. At the eastern extraction point, the wave rose shows a much higher distribution of south-southeast waves, which would direct transport northeast along the shoreline. The nearshore wave roses indicate the potential for a nodal point, or bi-directional longshore transport near Caminada Pass, which could cause increased erosion relative to the rest of the Grand Isle shoreline. Longshore transport patterns are further discussed in Section 1.8.1.

1.5.2 Extreme Wave Conditions

Extreme wave conditions were modeled for 25-year and 50-year return periods. The SWAN model described in Section 1.5 was used to transform offshore waves to the project site. A summary of the extremal waves for the offshore WIS station is shown in Table 9. As previously described in Section 1.2, point gauge data does not provide suitable estimates of extreme winds, and therefore the 10-minute extremal winds determined from the NHC analysis were used and are shown in Table 9. Results are shown on Figure 10 and Figure 11.

Table 9. Offshore significant wave heights and peak periods from extremal analysis of WIS station 73130, and wind speed from NHC extremal analysis for return periods.

Tr [yrs]	Hs [ft]	Tp [s]	U [mph]
5	18.3	12.3	84.1
10	21.2	13.5	97.7
15	22.8	14.2	106.6
20	23.9	14.6	118.4
25	24.8	15.0	126.4
50	27.4	16.1	132.3
75	28.9	16.7	150.0
100	30.0	17.2	159.8

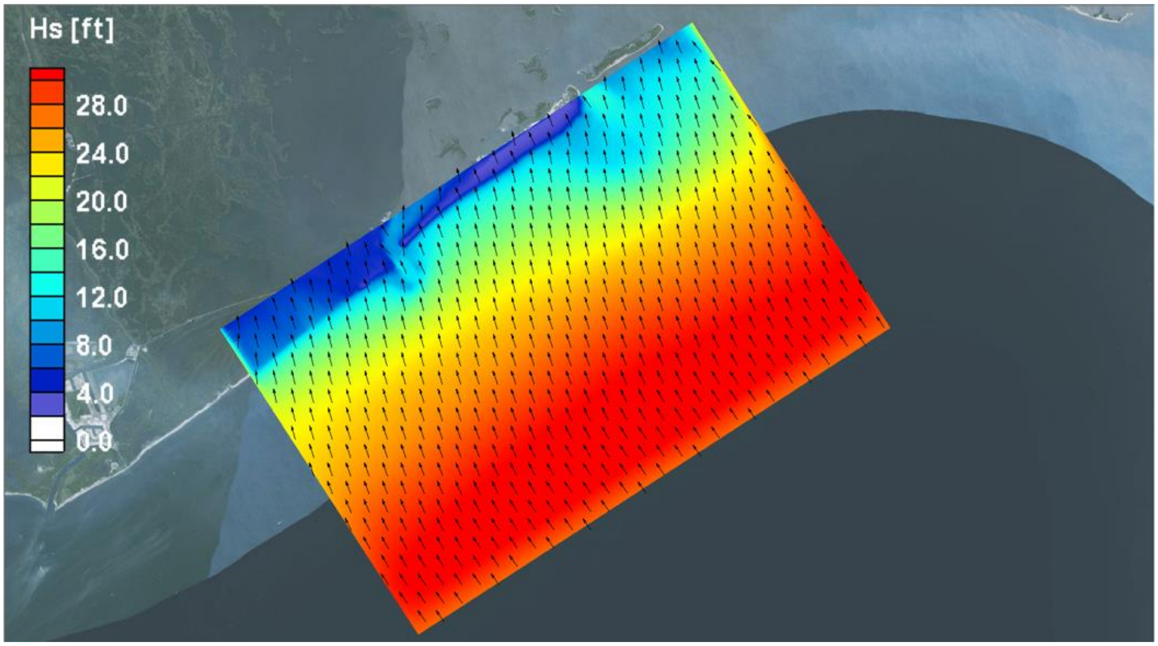


Figure 10. Wave model results for 25-year return period.

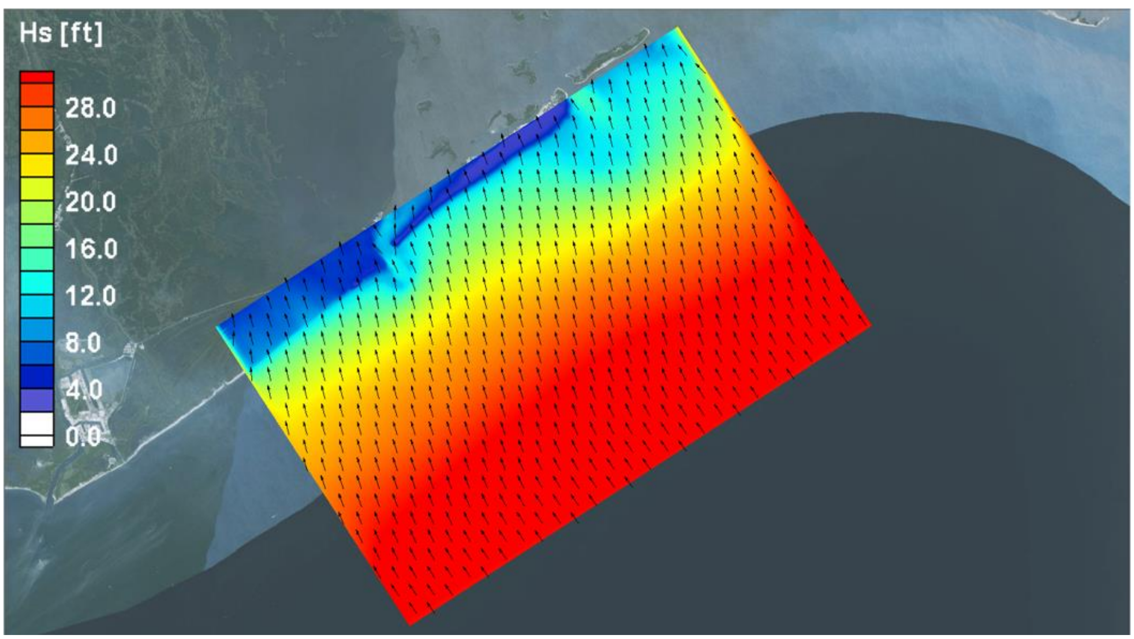


Figure 11. Wave model results for 50-year return period.

The 50-year waves were extracted at a depth of 13.0 feet NAVD88 (including 8.8 feet of storm surge), approximately 1300 feet seaward of the Grand Isle shoreline. The 25-year waves were extracted at a depth of 12.3 feet NAVD88 (including 8.1 feet of storm surge), approximately 1300 feet seaward of the Grand Isle shoreline. The 25-year and 50-year nearshore significant wave height were determined to be 10.7 feet and 11.3 feet respectively, at the extraction location.

1.6 Circulation Modeling

Circulation (hydrodynamic) modeling was performed for the project vicinity to evaluate the effects of circulation and inlet processes to the local shoreline morphology. Circulation modeling was conducted using the 2-Dimensional free surface circulation model ADCIRC (Luettich, 1991). The model domain development and calibration was discussed in the CHE (2005) Grand Isle Barrier Shoreline Stabilization Study Preliminary Engineering Report prepared for the Louisiana Department of Natural Resources (2007); details on the model calibration will not be discussed herein. The bathymetry surface discussed in Section 1.3 was used to update the previously developed circulation modeling grid.

The tidal fluctuations in the model were forced at the boundary by prescribing variable tidal harmonic constituents along the offshore boundary. Wind speeds and directions also influence circulation; the wind speeds and direction as well as barometric pressure were obtained from the GDIL1 NOAA gauge located on the Northeast side of Grand Isle, and used as input across the entire modeling domain.

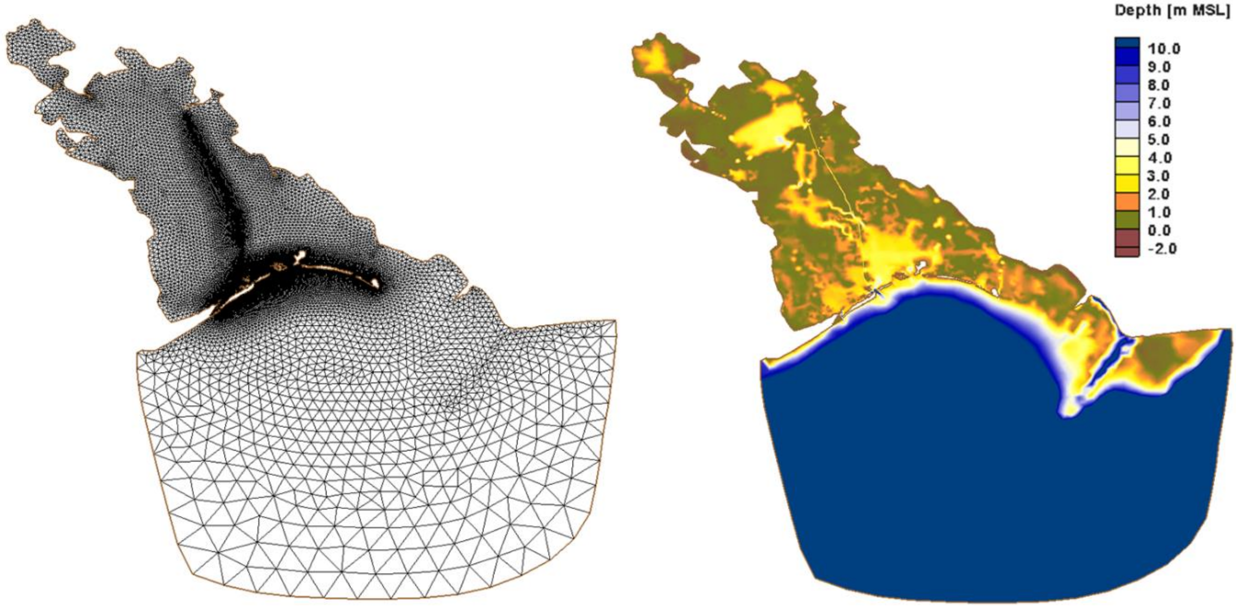


Figure 12. ADCIRC Modeling domain model mesh (left) and model bathymetry (right).

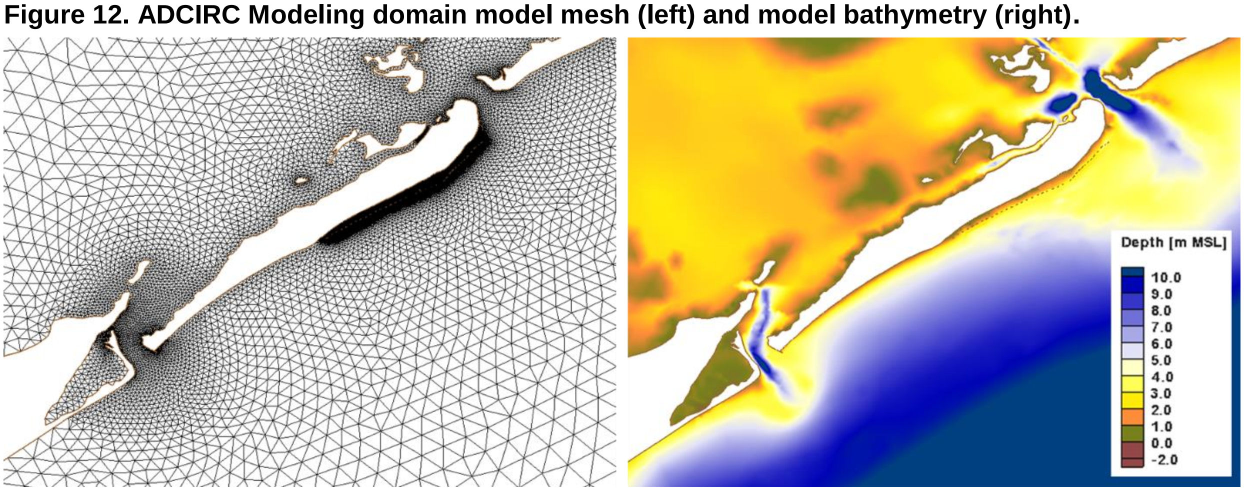


Figure 13. ADCIRC modeling domain of project vicinity model mesh (top) and model bathymetry (bottom).

Currents in Caminada Pass are shown in Figure 14. On both flood and ebb, flows reach about 0.7 to 0.8 m/sec (2.3 to 2.6 ft/sec) in the main throat of the pass. On the flood cycle, flow velocities are fairly uniform throughout main throat of the pass until the flow diverges at Cheniere Caminada. High velocities are experienced in the breach in Cheniere Caminada of more than 1.5 m/sec (5 ft/sec) on both flood and ebb.

The average currents at the southwest end of Grand Isle by the end of the revetment (0.3 mi from the West Jetty) are shown on Figure 15. The tidal currents vary from 0 to 0.1 m/s. The presence of non-negligible currents moving towards Caminada Pass along the western end of Grand Isle shoreline indicates that the tidal currents will further increase the localized net longshore transport moving towards west. This will further increase the already occurring shoreline erosion due to the presence of divergent point of transport.

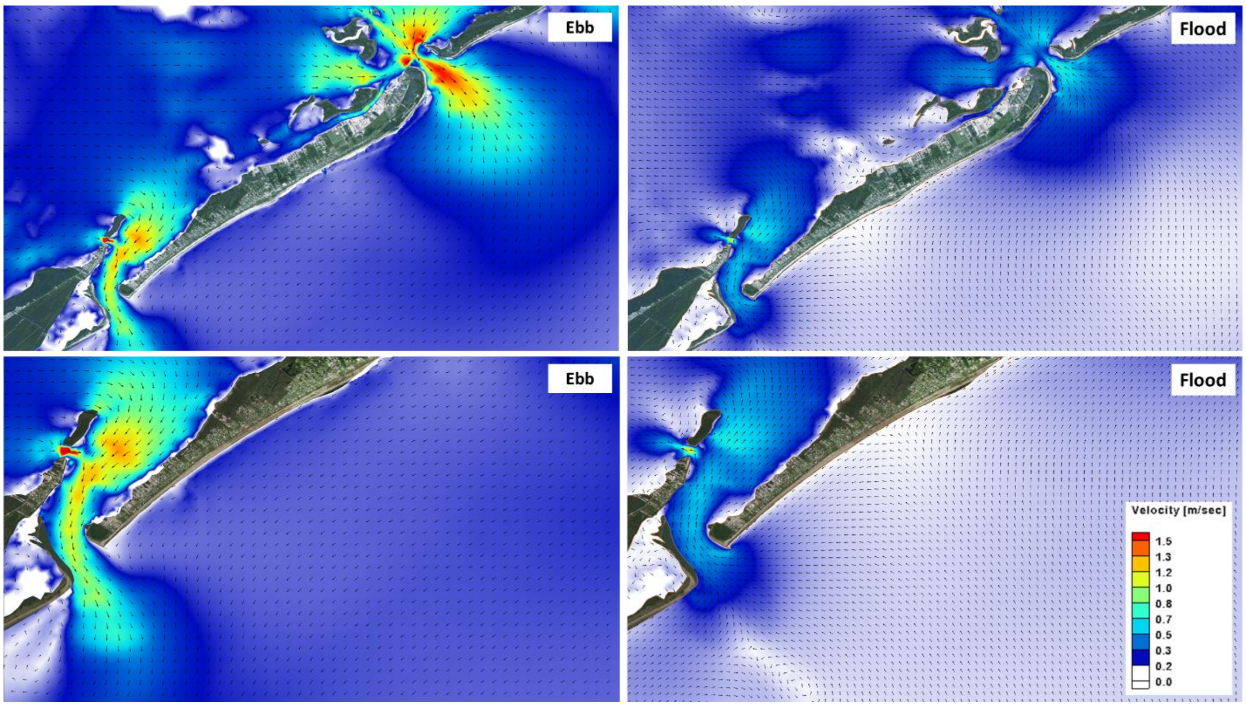


Figure 14. Circulation model results at Grand Isle (top) and Caminada Pass (bottom). Scale on bottom right applies to all plots.

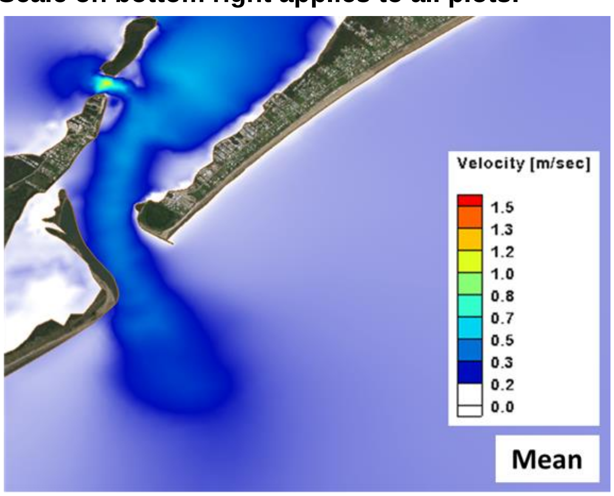


Figure 15. Average currents at southwest end of Grand Isle.

1.7 Morphology Analysis

The goal of the morphology analysis is to develop an understanding of the morpho-dynamic conditions along Caminada Pass and Grand Isle with an emphasis on the western end of the island. Morphology analysis was achieved via shoreline change analysis (Section 1.7.1), bottom morphology analysis (Section 1.7.2), and morphology modeling (Section 1.7.3).

1.7.1 Shoreline Change Analysis

Shoreline change analysis was conducted to develop an understanding of how the project shoreline has changed over the years. The shoreline position was derived by delineating the visible wet-dry line from aerials collected during the previous phase of this project (CHE, 2016). The shoreline positions (referenced to a baseline landward of all the shorelines) were determined at orthogonal transects spaced 30 m along the baseline extending from Caminada Pass to Barataria Pass.

This process involved the use of GIS software to first derive digital shorelines by delineating the visible wet/dry line in each of the georeferenced aerials. A total of 21 shorelines covering the entire Gulf of Mexico Grand Isle shoreline were delineated from aerial images from 1945 to 2015. Figure 16 shows the extents of the delineation and two shoreline delineation samples. The following steps involved casting transects across the shorelines from a baseline, measuring the shoreline positions, and ultimately quantifying the shoreline change rates.

Figure 16. Left: the 2007 aerial photograph and delineated shoreline; right: the 2015 aerial photograph and delineated shoreline.

Figure 17 and Figure 18 displays the position of each delineated shoreline relative to the average shoreline position from 1945 to 2015. The shorelines were plotted using the average shoreline from 1945-2015 as the detrending base shoreline. The 1984 shoreline clearly shows the effect of the beach nourishment project done in 1984 as the shoreline advanced almost 200 feet seaward compared to the 1979 shoreline. In the years after the1984 beach nourishment, salient like features developed on either side of the dredge possibly due to the wave refection over the dredge template. These salients diffused gradually over years. Construction of the breakwater field along the eastern end of the shoreline seemed to have stabilized the shoreline from erosion from about 3.5 miles east of the West Jetty. Since 2010, the west 2 miles of the shoreline have been eroding as indicated in Figure 18. The exception is the shoreline immediately adjoining the jetty where slight accretion is observed. This indicates a potential nodal point of bi-directional transport present along the western end of Grand Isle shoreline.

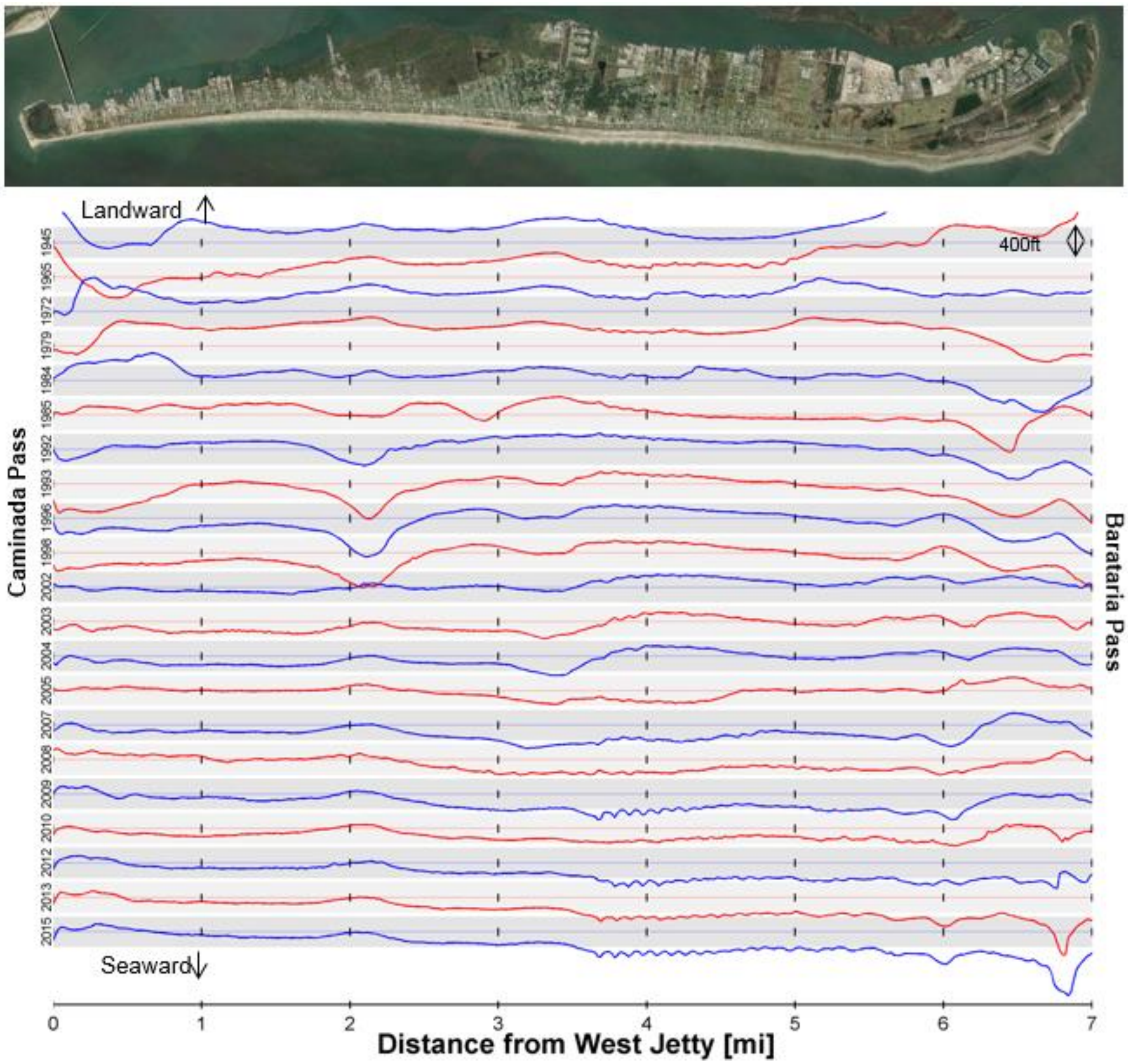


Figure 17. Grand Isle shoreline positions for each year relative to the average shoreline position from 1945 to 2015. The grey box represents a distance of +/- 400 ft from the average position. For a given year, sections of the shoreline below (above) the horizontal line represents accretion (erosion) with respect to average shoreline.

Due to the execution of multiple beach nourishment projects since 1979 along the project shoreline, it is not feasible to predict the background long-term shoreline change rates. Therefore, shoreline change rates were determined using the shoreline for the past few years; before (2010-2012) and after (2013-2015) construction of the rock revetment along the western end of Grand Isle shoreline. These shoreline changes are shown in Figure 19 for 3.5 miles of western end of island shoreline. The hot spot for shoreline erosion lied around 0.3-0.4 miles east of West Jetty where the shoreline was eroding at almost 50 ft/yr prior to the construction of the rock revetment. After the construction of rock revetment in 2013, the erosional hot spot has shifted downdrift of the revetment (0.3-0.6 miles east of West Jetty) as indicated in Figure 19 and the site photo shown in Figure 20. The shift in the erosional hotspot is thought to be attributed to the sand source starvation (from the original hot spot of erosion) due to the construction of the rock revetment.

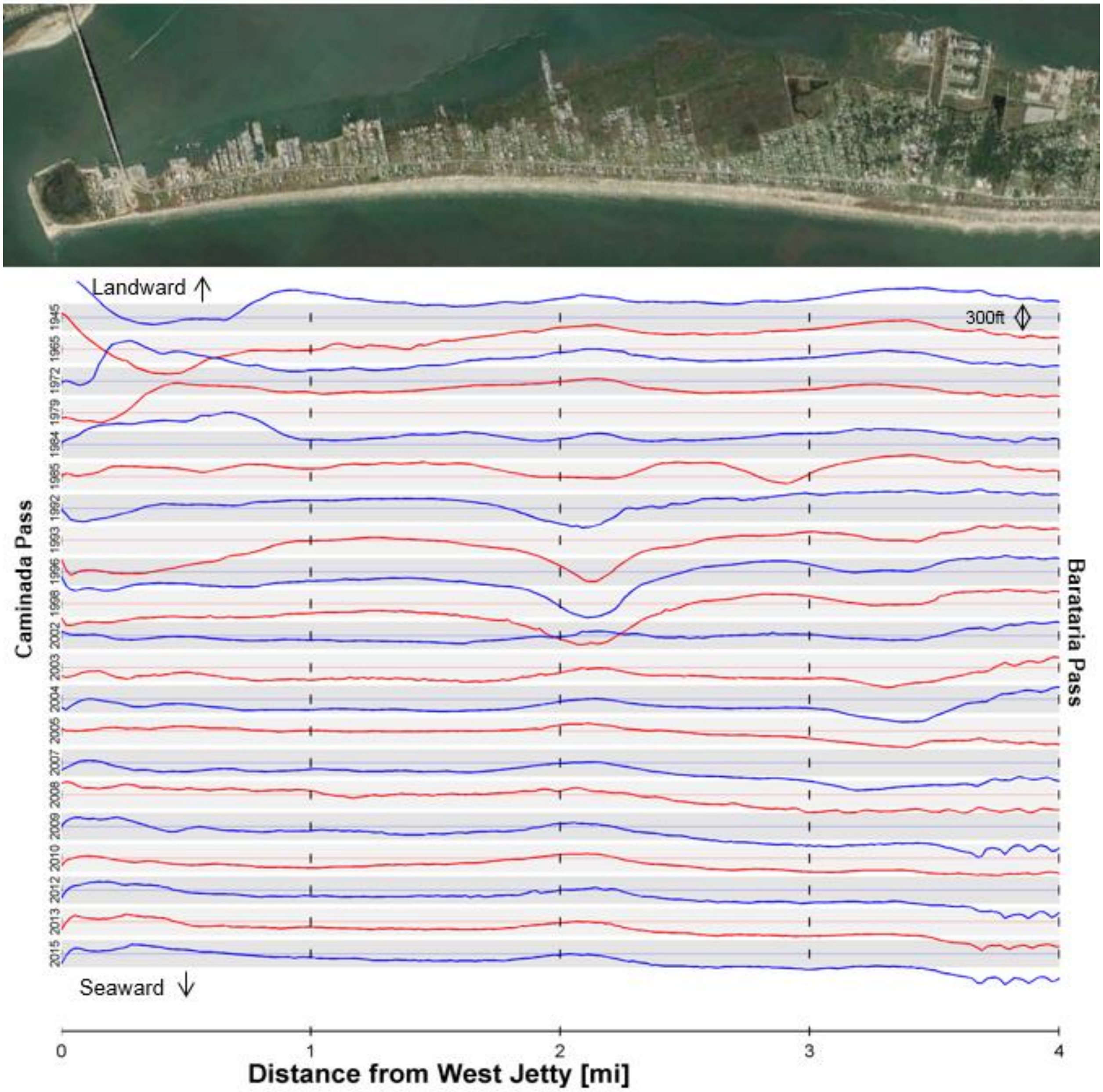


Figure 18. Southwest end of Grand Isle shoreline positions for each year relative to the average shoreline position from 1945 to 2015. The grey box represents of +/- 300 ft from the average position. For a given year, sections of the shoreline below (above) the horizontal line represents accretion (erosion) with respect to average shoreline.

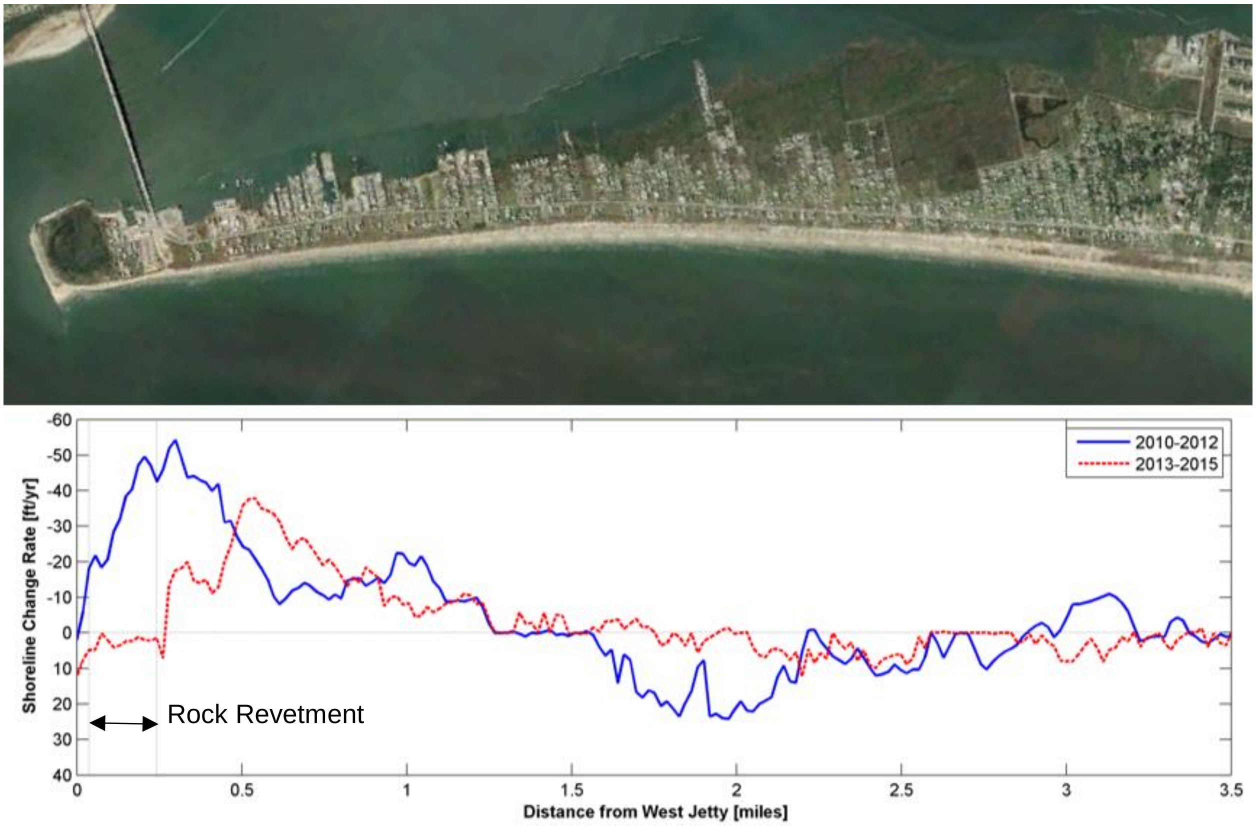


Figure 19. Shoreline change rates at southwest end of Grand Isle from 2010 to 2012 (blue) and 2013 to 2015 (red), vertical axis is reversed for visual purposes, negative values indicate erosion, and positive values indicate accretion.

Figure 20. Northern end of rock revetment, existing scarp is indicative of severe erosion. Picture taken on July 2016 looking towards the southwest.

1.7.2 Bottom Morphology

Changes of bottom morphology were assessed by evaluating the differences between bottom contours throughout different years. The discrete bathymetric surfaces shown in Figure 21 were created from the following available data. (For detailed information on data sources refer to Bathymetry and Topography section in CHE, 2005.)

- 2016: Preliminary Hydroterra survey
- 2015: BICM-2 2015 Regional Bathymetric Survey by John Chance Land Surveys, Inc.
- 2006: BICM Volume 3: Bathymetry and Historical Seafloor Change 1869-2007
- 1980: BICM Volume 3: Bathymetry and Historical Seafloor Change 1869-2007
- 1930: BICM Volume 3: Bathymetry and Historical Seafloor Change 1869-2007

The channel in the Caminada Pass appears to be deepening each year and shifting slightly to the west, or perhaps straightening, from 2006 to 2015. The accretion observed around the end of the West Jetty on the east side of Caminada Pass could be caused by the detached spit attaching to this point or change in transport patterns caused by the ebb shoal itself. In addition, the contours on the eastern side of the ebb shoal have deepened and slightly changed alignment which modifies the wave directions through refraction which can modify the nearshore morphology.

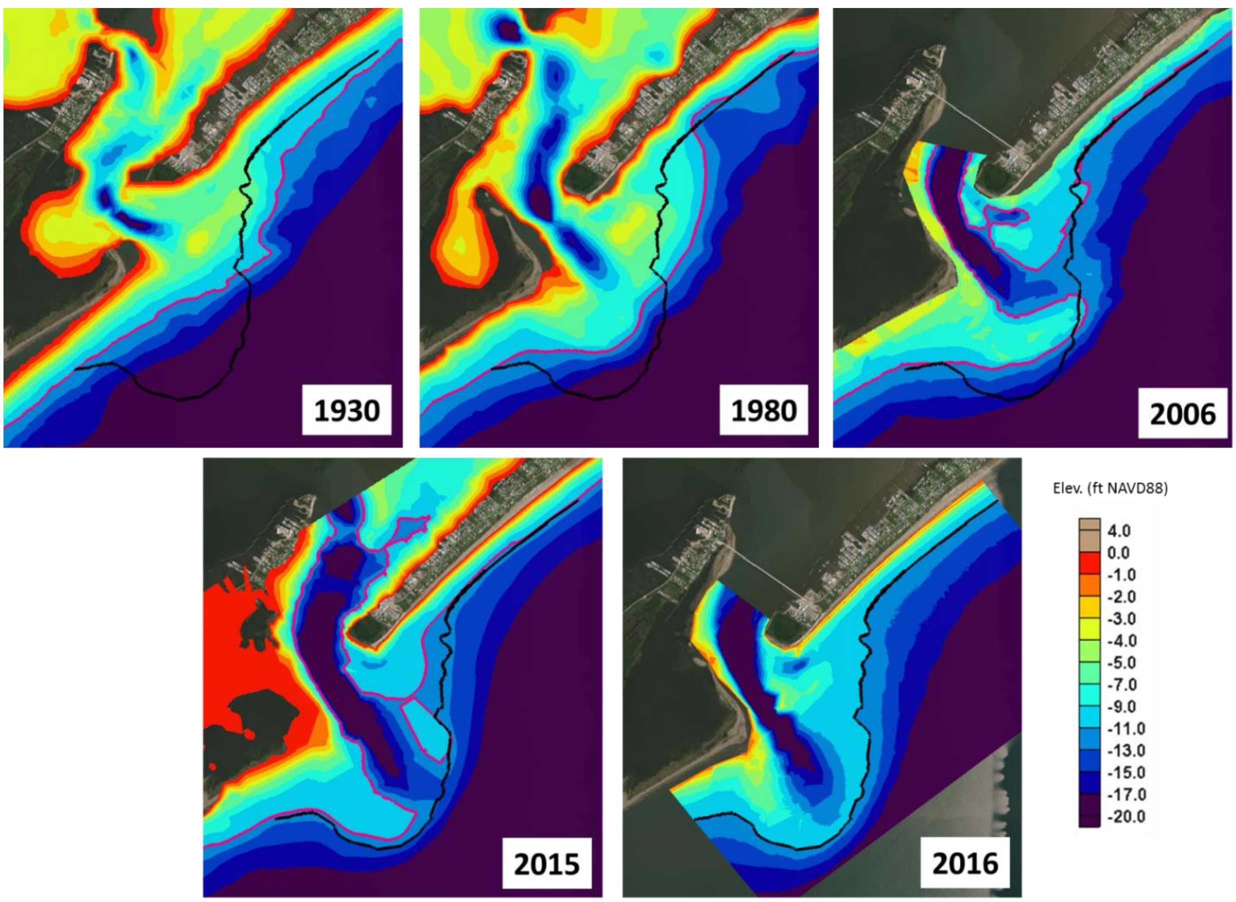


Figure 21. Western end of Grand Isle bathymetric surfaces for 1930, 1980, 2006, 2015, and 2016. Pink line represents 11-ft contour for the given year; black line represents the 2016 11-ft contour. Scale applies to all plots.

1.7.3 Morphology Modeling

A morphology model was developed to simulate the Grand Isle shoreline changes using the GenCade shoreline morphology model (Frey et. al., 2012), a 1-D shoreline morphology numerical model based on the synthesis of the GENESIS model (Hanson and Kraus, 1989) and the Cascade model (Larson et. al., 2003). GenCade calculates wave-induced longshore sand transport rates and the resulting shoreline change.

The GenCade model setup requires defining the initial shoreline, wave forcing conditions, boundary conditions, time step and duration for model run and beach characteristics (effective grain size, average berm height, depth of closure, longshore sand transport calibration coefficients) as the primary parameters for model execution. The model setup also provides the capability to define other natural and structural features such as inlets, breakwaters, groins, seawalls, and beach nourishment events.

The GenCade modeling grid used for calibration and validation spanned approximately 37,000 feet of the shoreline, extending from the jetty along the western edge of Grand Isle abutting Caminada Pass and extending northeast to the jetty abutting Barataria Pass. A gated boundary condition was applied at both ends of the grid to simulate the presence of the jetties on either side of Grand Isle. The jetty in the model was set to the approximate length of the existing jetties based on 2015 aerial photography.

Sediment input towards the Grand Isle shoreline from Caminada Pass is estimated to be approximately 90,000 cubic yards per year, as shown in Section 1.7.2 . Morphological analysis discussed in Section 1.7.2 shows that the Caminada Pass ebb shoal is growing. It is then hypothesized that a portion of these 90,000 cubic yards is being deposited in the Caminada Pass ebb shoal. A morphological analysis was conducted to determine the amount of sand being trapped in the Caminada Pass ebb shoal. Bathymetric surveys from 2006 and 2016 were analyzed. It was determined that the ebb shoal is growing at approximately 40,000 cy/year. It was assumed that this volume contributing to ebb shoal growth is not bypassing to the Grand Isle shoreline. Therefore, morphological modeling assumed 50,000 cubic yards per year (90,000 cubic yards minus 40,000 cubic yards being trapped by the Grand Isle ebb shoal) was being bypassed to the Grand Isle shoreline. The extents of bypassing were determined by sensitivity testing during calibration testing.

Sensitivity testing was conducted by varying the porosity of the groin placed at the Caminada Pass gated boundary and measuring the associated shoreline response. The porosity of each groin was varied from 0.0 (sand tight) to 0.9 (simulating a structure that is 90% porous). Sensitivity testing showed that almost no change occurred in shoreline response between the sand tight gated boundary and the porous boundary conditions. This is due to the relatively short length of the Caminada Pass jetty and the relatively minor volume of westerly transport due to the local bi-directional transport in this area of the project site, as discussed later in this Section. Therefore, a porosity of 0.0 was selected for the gated boundary condition.

A second test was performed by artificially extending the jetty by 100 m. The extension showed an additional 50 ft of accretion when compared to the FWOP shoreline, however this accretion only extended 100 meters east of the jetty, and does nothing to alleviate the erosion experienced at the eastern edge of the existing revetment. This shows the model is responsive to the presence of the jetty and the jetty has an influence on transport, but the influence is minor. Therefore, a jetty extension is not recommended as a potential alternative.

The wave time series developed from the wave modeling (Section 1.5) were used as the wave forcing to drive the longshore transport within the GenCade model. The extraction points selected were done close enough to shore that the effect of the large ebb shoals flanking the project site were captured by the wave model transformation. Based on the review of available geotechnical data and surveyed beach profiles, including previous work by Mott MacDonald the effective grain size, active beach profile height and depth of closure were set as 0.15 mm, 8.5 ft NAVD88, and -12.0 ft NAVD88, respectively. The longshore sand transport calibration coefficients were finalized during the calibration run and are discussed in Section 1.7.3.1. The model setup was calibrated and validated (see Sections 1.7.3.1 and 1.7.3.2) to be used as a tool for Alternative Analysis.

GenCade, as a one-line model, has limitations. It assumes a uniform cross-sectional beach profile (essentially uniform shore parallel contours) for the modeling domain. The model also assumes a constant grain size, berm height and depth of closure for the entire model domain and therefore, detail level variations along the modeling grid cannot be included in the model setup. GenCade does not consider the material lost from the beach due to cross-shore transport which is the primary cause of material loss from the beach profile during storm events. Therefore, model results should be interpreted with these fundamental limitations kept in mind.

1.7.3.1 Model Calibration

The model calibration establishes two sediment transport coefficients, K1 and K2, which are discussed in the following paragraphs.

The model was calibrated using the measured 2010 shoreline as the initial shoreline and running the model for three years to predict the 2012 shoreline. The K1 coefficient, which governs transport rates, was calibrated to 0.175 to match the morphological changes during this period. The modeled 2010 shoreline was compared with the measured 2012 shoreline, the results of which are shown in Figure 22. When determining the accuracy of the modeled shoreline, it is important to consider the inherent error involved in delineating shoreline position from historical aerials. Error is inherent in the delineation of shorelines, where even field identification can be highly subjective. LiDAR-derived shorelines are horizontally accurate to approximately 3.0 m (Stockdon et al., 2002). Error due to seasonal variability has been determined to range from approximately 10 m (Moore, 2000) to 50 m (Ruggiero et al., 2003) and is based on the beach slope, wave energy, and wave run-up on the beach. The positional error due to GPS location has been determined to be approximately 2 m (Ruggiero et al., 2003). Table 10 shows values used to calculate the total measurement error using the estimates root mean square method for various data sources (Crowell et al., 1991; Moore, 2000).

Table 10. Estimates of Measurement Error of Delineating Shoreline Position in Meters.

Measurement Errors [m]	LiDAR	GPS	Aerial [pre-1995]	USGS DOQ	NAIP
Source Error (Es)	3	2	14	7	7
Digitization Error (Ed)	--	--	5	5	5
Shoreline Variability Error (Esv)	10	10	10	10	10
Total Shoreline Position Error (Ep)=√(Es)2+(Ed)2+(Esv)2)	10.4	10.2	17.9	13.2	13.2

The modeled minus measured shoreline position error for 2012 were also computed and are shown in Figure 22. Figure 22 shows that most the Grand Isle shoreline position varied within the bounds of shoreline positional error (+/- 43.3 ft or +/- 13.2 m/yr). The modeled shoreline position adjacent to the jetty and at the Eastern end of the project area over predicted erosion compared to measured data. This can be attributed to boundary condition effects, which become more apparent at the edges of the modeling domain.

The K2 calibration tests the shoreline response due to diffraction around structures. Shoreline morphology along the eastern half of grand isle was simulated to calibrate the K2 coefficient. The breakwaters along this section of shoreline were built into the modeling grid. The K2 coefficient was calibrated to be 0.03. Calibration of the K2 coefficient is included in the results shown in Figure 22. While the model captures some shoreline response of the breakwaters, it does not simulate the response completely. GenCade also allows for breakwater transmission to be included in model results. The breakwater transmission coefficient, determined to be 0.127, was calculated using the methodology of Buccino and Calabrese (2007) for an idealized breakwater section. It is expected that each structure will have a unique transmission characteristic depending on the current state of breakwater damage. Detailed additional calibration to refine these results along the eastern stretch of the Island was not undertaken as the project focus was on the western portion of the island, and due to the transport patterns, the eastern end is expected to have negligible influence on the western shoreline. Figure 23 shows the de-trended modeled shoreline (2012) compared to the measured 2012 shoreline and the initial 2010 shoreline. All shorelines shown in Figure 23 were de-trended using the same methodology as described earlier in Section 1.7.1.

Model run times for the calibration setup, which spanned 2 years of real world wave conditions from 2010 to 2012, took approximately 14 hours to complete. To allow for more computationally efficient alternative analysis, a shortened grid was also tested. The grid was cut off at the westernmost existing breakwater on the Grand Isle shoreline. This resulted in a shortened grid length of approximately 3.6 miles, extending from the Caminada Pass Jetty to the westernmost existing breakwater. Calibration results for the shortened grid are shown below in Figure 24. Calibration results for the shortened grid showed similar accuracy to the longer grid that spanned the full island. Therefore, the shortened grid was deemed sufficient for alternatives analysis.

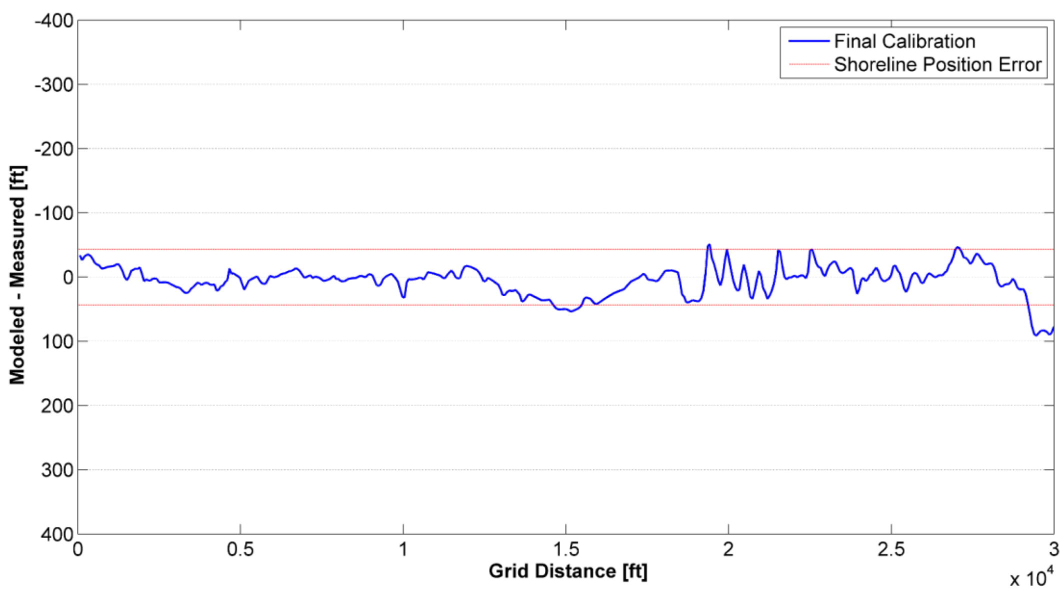


Figure 22. Modeled - Measured shoreline positions for the calibration run. The red lines represent the positional error.

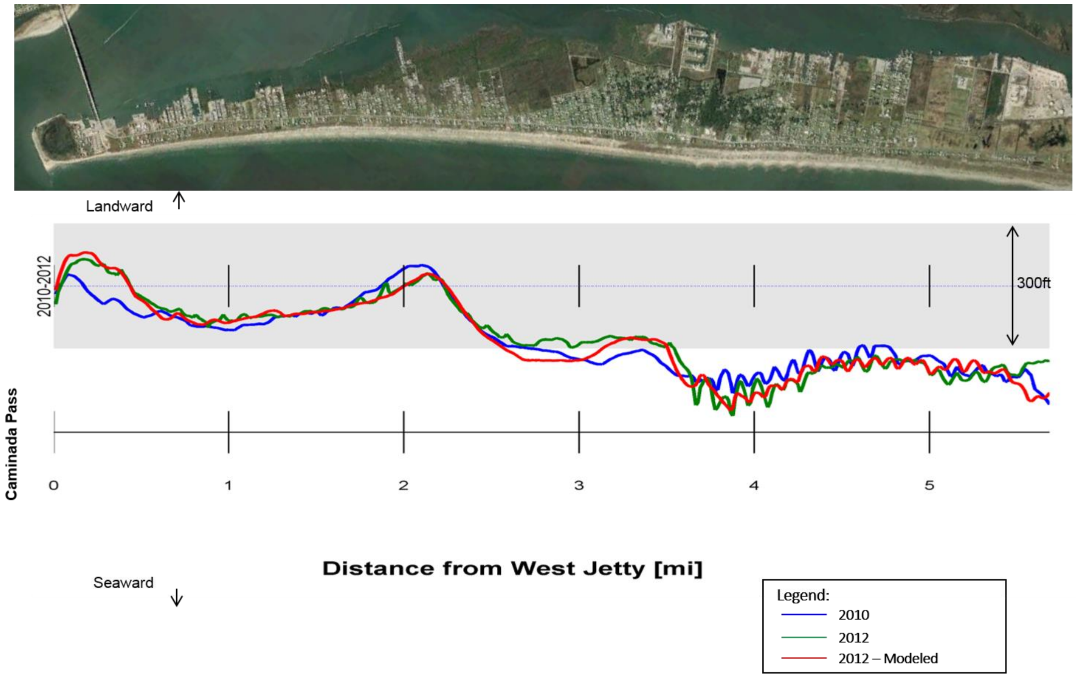


Figure 23. Grand Isle shoreline positions for 2010 (blue), 2012 (black), and modeled 2012 shoreline (red) relative to the average shoreline position from 1945 to 2015. The grey box represents of +/- 300 ft from the average position. For a given year, sections of the shoreline below (above) the horizontal line represents accretion (erosion) with respect to average shoreline.

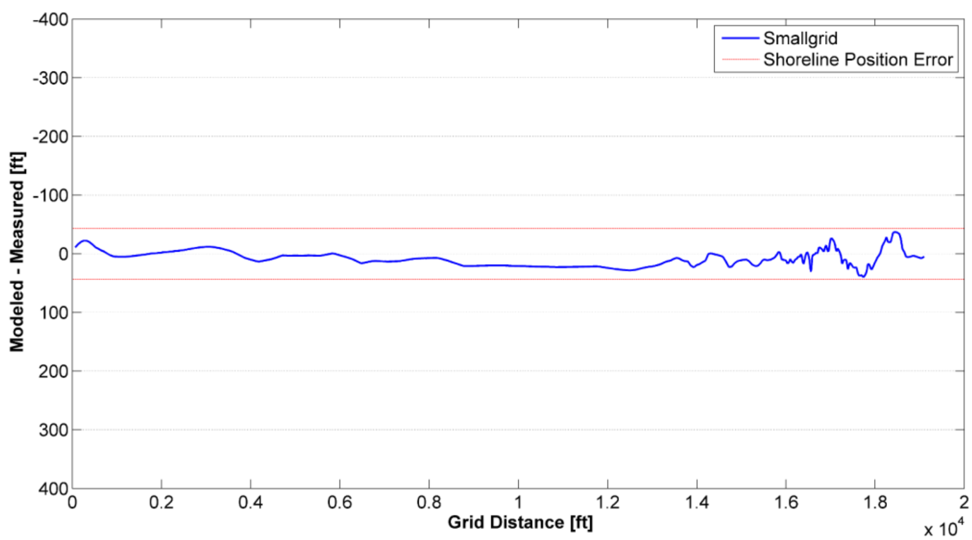


Figure 24. Modeled - measured results for calibration run, shortened grid.

1.7.3.2 Model Validation

The model validation was conducted by using an initial 2013 shoreline and running the model for two years to predict the 2015 shoreline position, the resulting longshore transport rates, and the average erosion rates. The wave time series from 2013-2015 generated from the wave modeling as discussed in Section 1.5 was used to force the wave boundary. It should be noted that the 2015 shoreline position was captured in May of 2015. The WIS time series that was transformed to the project site ends on December 31, 2014. To model the shoreline morphology from January 2015 to May 2015, the 2014 time series for this timeframe was used. Figure 25 shows the modeled minus measured shorelines for the validation run. The modeled shoreline was matched with the measured 2015 shoreline. The modeled shoreline varies within +/- 40 ft for most of the project shoreline which is within the positional accuracy of +/- 43.3 ft (+/-13.2 m) discussed in Section 1.7.3.1. The results for the measured minus modeled shorelines and the grand isle shoreline positions for the validation run can be seen in Figure 25 and Figure 26, respectively.

Similar to the calibration setup, the shortened grid to be used for alternative analysis was analyzed to determine if any loss of accuracy occurred for the validation runtime. The validation run for the shortened grid showed similarly positive results to the calibration run. The measured minus modeled shoreline error for the shortened grid can be seen below in Figure 27.

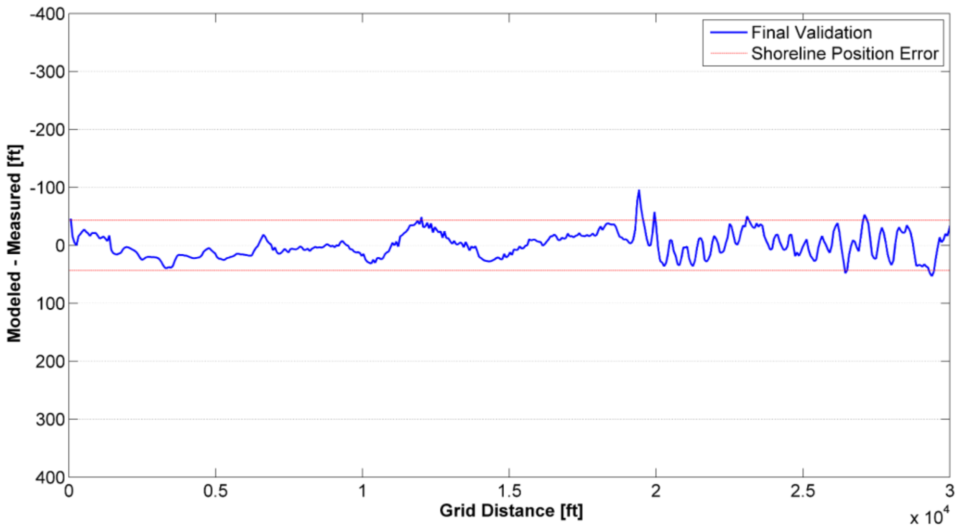


Figure 25. Modeled minus measured shoreline for validation run. Red lines represent the positional error.

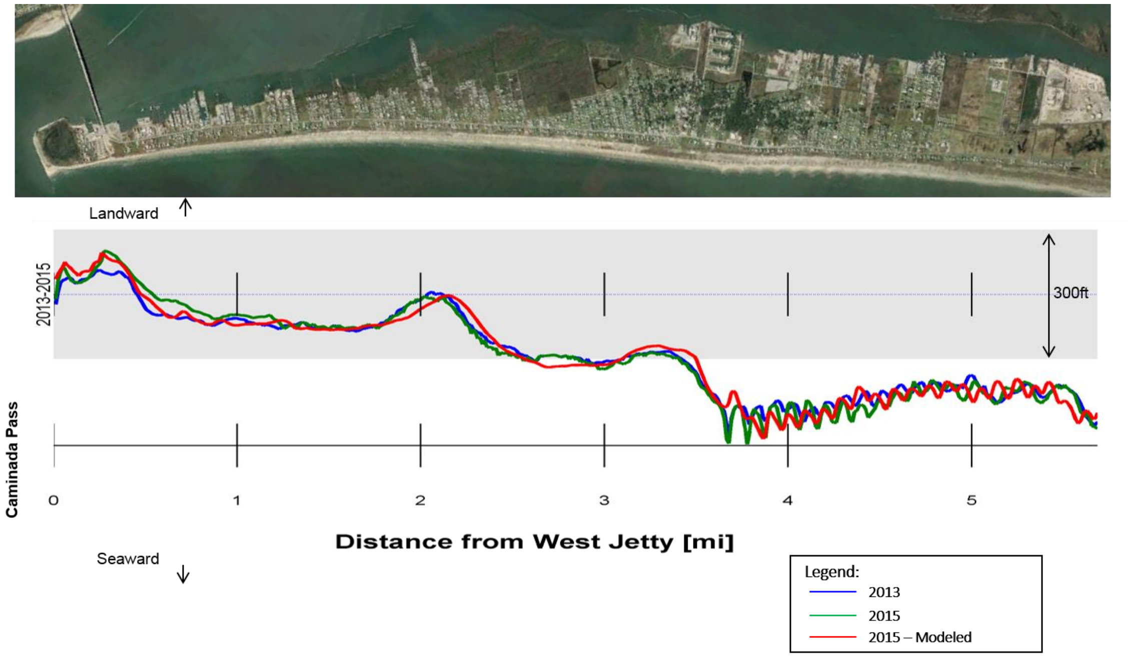


Figure 26. Grand Isle shoreline positions for 2013 (blue), 2015 (black), and modeled 2015 shoreline (red) relative to the average shoreline position from 1945 to 2015. The grey box represents of +/- 300 ft from the average position. For a given year, sections of the shoreline below (above) the horizontal line represents accretion (erosion) with respect to average shoreline.

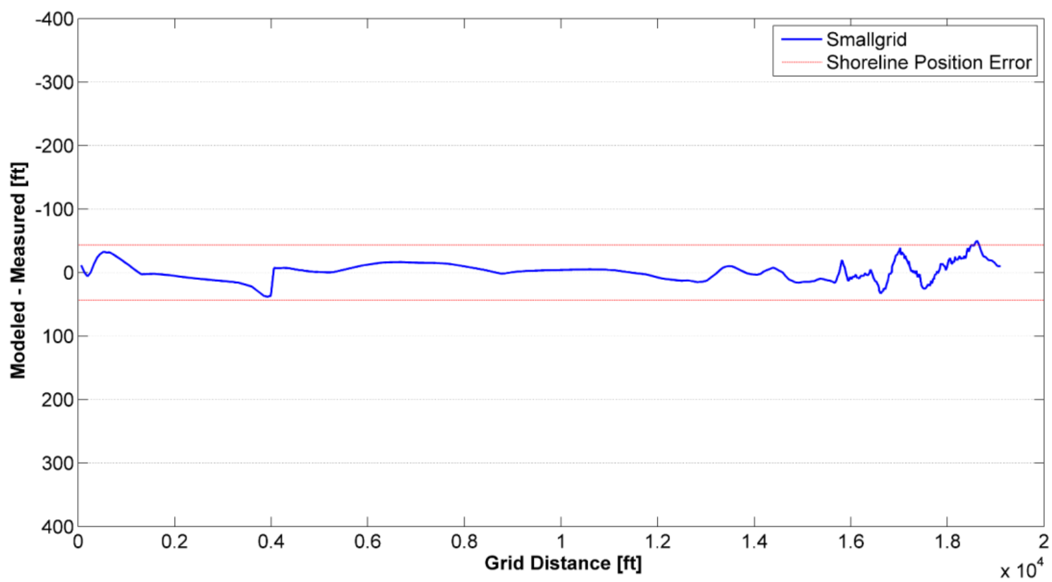


Figure 27. Modeled - measured shorelines for validation run on shortened grid.

1.8 Sediment Budget

The goal of the sediment budget is to develop an understanding of shoreline morphology at Grand Isle. Sediment budget was accomplished by combining results from sediment transport (Section 1.8.1) and results from previous studies such as and Batten et. al. (2004).

1.8.1 Sediment Transport

Longshore transport (LST) rates at the project site were calculated from calibrated and validated GenCade (Frey et al. 2012) model results for the Grand Isle. Complete discussion on the methodology and results from the shoreline morphology modeling using GenCade can be found in Section 1.7.3.

Figure 28 shows the average yearly longshore transport rates along Grand Isle. The predominant direction of the net longshore transport is northeast. Near the western jetty, the transport is more bi-directional. Throughout most the western end of project shoreline (denoted by the dotted lines), the shoreline is erosional as shown by the positive slope of the net transport. The middle section of Grand Isle (between 2-3.5 miles from western jetty is relatively stable shown in Figure 28 with the zero slope of the net transport. The eastern shoreline shows mostly accretion, likely due the offshore breakwaters in the area.

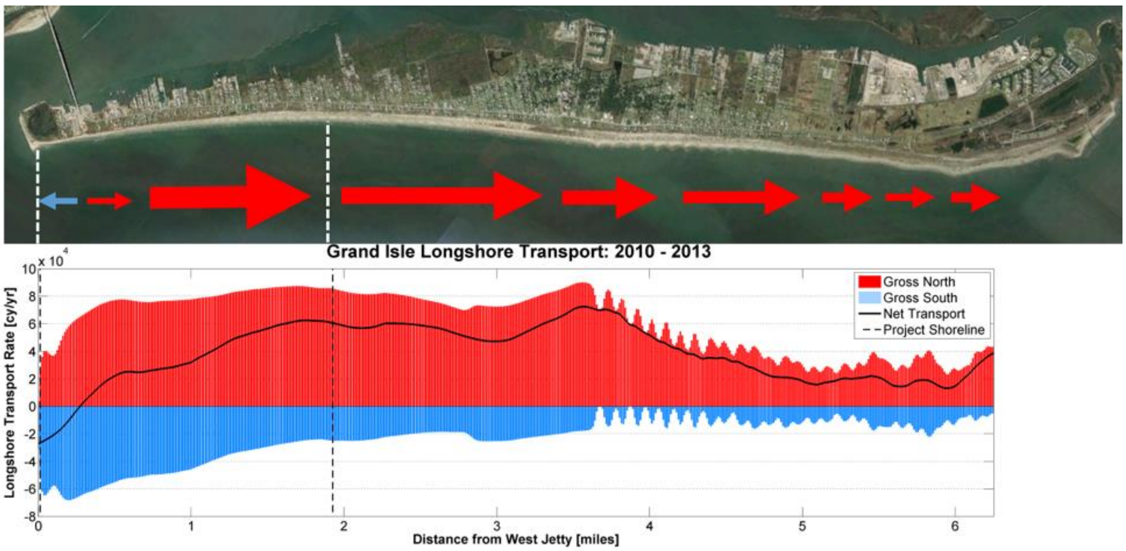


Figure 28. Computed LST rates from 2010 to 2013. Gross transport directed toward the southwest is shown with blue bars, gross transport directed toward the northeast is shown in red bars, and the thick black line shows the net longshore transport rate.

1.8.2 Sediment Budget Analysis

A sediment budget was compiled using the results from a previous study (CHE, 2012) and from the longshore transport rates from the GenCade model developed in this study. The study done by CHE (2012) developed a conceptual level sediment budget along the Elmer's island and calculated approximately 90,800 CY bypassing Caminada Pass to the Grand Isle shoreline. This matches closely to the value of 83,000 CY reported by a USACE study (Batten et al., 2004) which only examined the volume changes on Grand Isle. As described in Section 1.7.3, approximately 40,000 cubic yards of sediment is being deposited in the Grand Isle side of the Caminada pass ebb shoal. Therefore, the input volume of 90,800 CY previously computed by CHE (2012), was reduced to 50,000 CY for bypassing to the Grand Isle sediment budget on the southwest end of the shoreline. The sediment budget along the shoreline was developed using longshore transport rates, described in the Section 1.8.1 and is shown in Figure 29. Consistent with the trends shown in previous analysis, sediment budget shows the highest erosion occurs along the 2 miles of shoreline immediately next to western jetty. The rest of the Grand Isle shoreline shows stable/no erosion (2-3.5 miles from West Jetty) or shoreline accretion (shoreline behind the breakwater field).

Figure 29. Sediment budget for Grand Isle for existing conditions (units in cy/yr).

1.8.3 Morphology Impacts on Sediment Transport

This section describes the changes in sediment transport patterns associated with the changes in bottom morphology (Section 1.7.2). Following the dominant wind and wave directions ESE, SE, SSE (Section 1.2), wave driven sediment transport was calculated using the bathymetric data sets from 2006, 2015, and 2016; the results are shown on Figure 30, Figure 31, Figure 32, respectively.

Results indicate how changes in the Caminada ebb shoal have affected the nearshore wave clime which has led to changes in sediment transport patterns. A focusing of wave energy in the southwest end of Grand Isle developed between 2006 and 2015 and has persisted through 2016. This new nearshore wave climate has resulted in a diversion node in sediment transport located at the end of the rock revetment. The chronology of sediment transport patterns in Grand Isle verify the presence of the erosion hot spot presented on Section 1.7.1.

Note that the cases shown in Figure 30, Figure 31, and Figure 32 represent idealized cases for each wave and wind direction. They are not a reflection of overall transport patterns, and are instead meant to show idealized transport patterns for a given wave and wind direction. Overall transport patterns are summarized in Figure 28. This figure shows overall net transport along the Grand Isle shoreline to the east. A nodal point is located just east of the Caminada Jetty. West of this nodal point, a small amount of sediment is traveling west towards the jetty. Note that the transport is very bi-directional in this area, explaining the lack of significant buildup along the Caminada jetty. The growth of the ebb shoal has caused refraction of waves, directing sediment transport west. This explains relatively small bypassing of sediment from Caminada Pass to the Grand Isle shoreline.

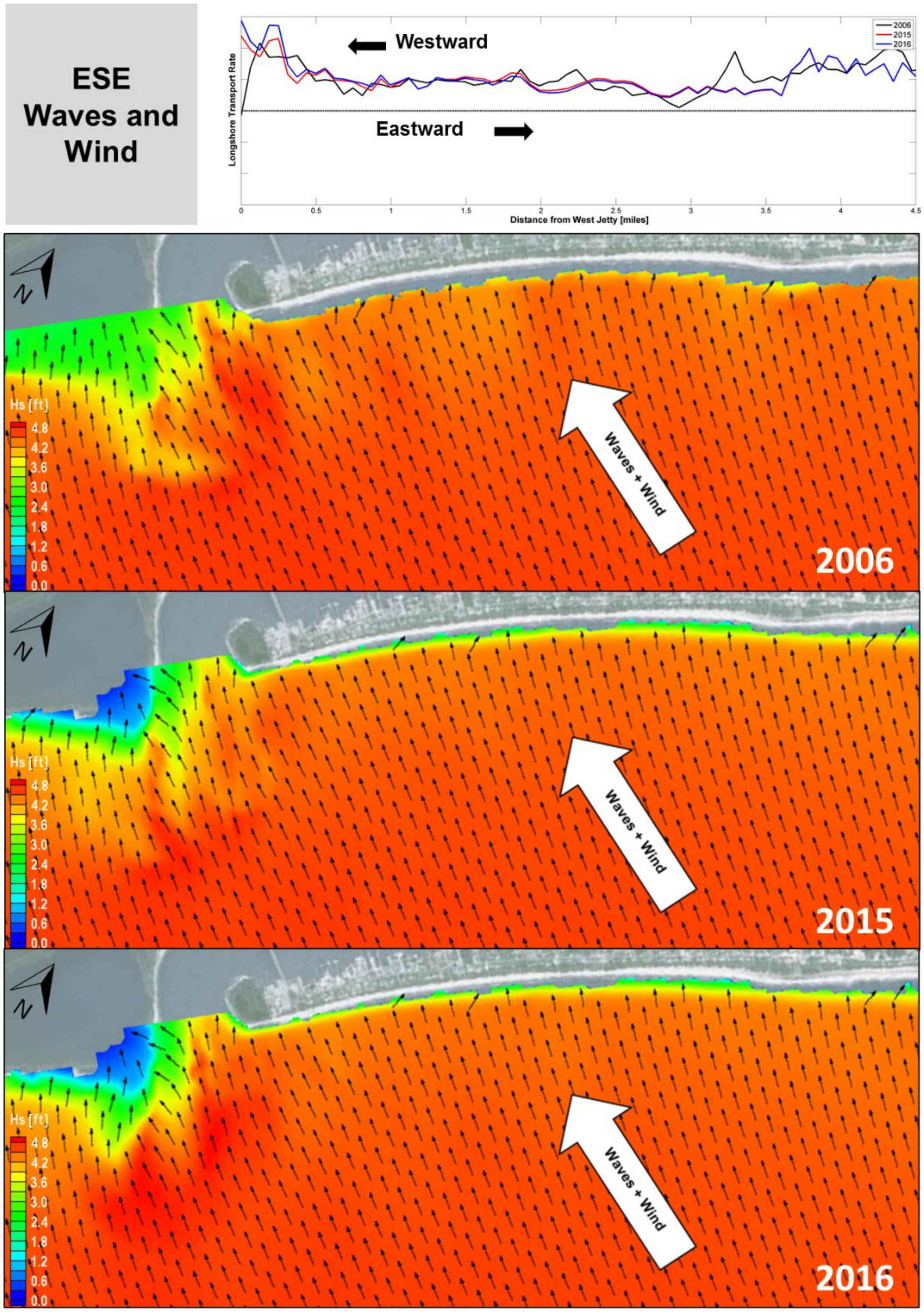


Figure 30. Nearshore wave significant wave height and longshore sediment transport for ESE wind and wave condition using 2006, 2015, and 2016 bathymetry.

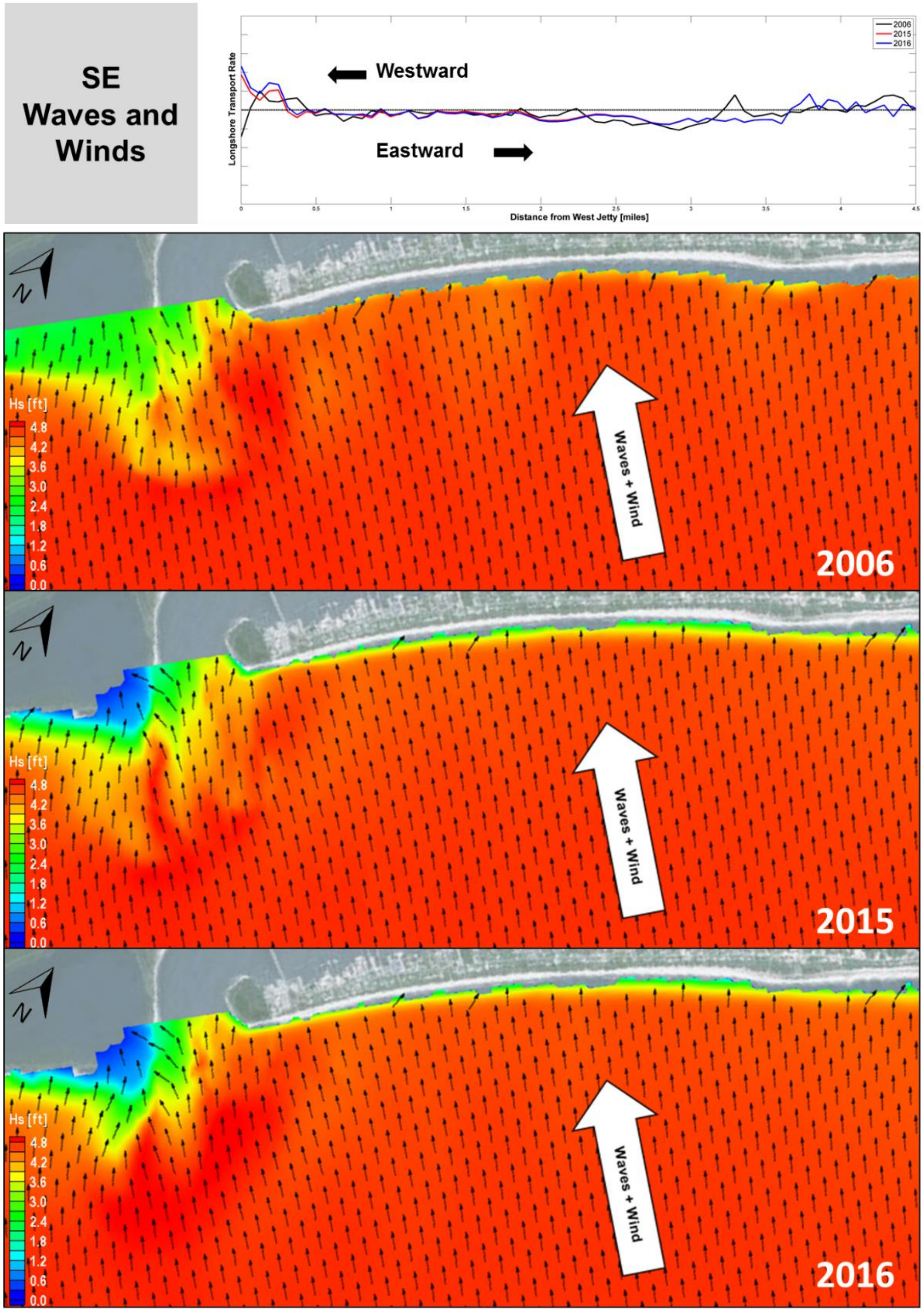


Figure 31. Nearshore wave significant wave height and longshore sediment transport for SE wind and wave condition using 2006, 2015, and 2016 bathymetry.

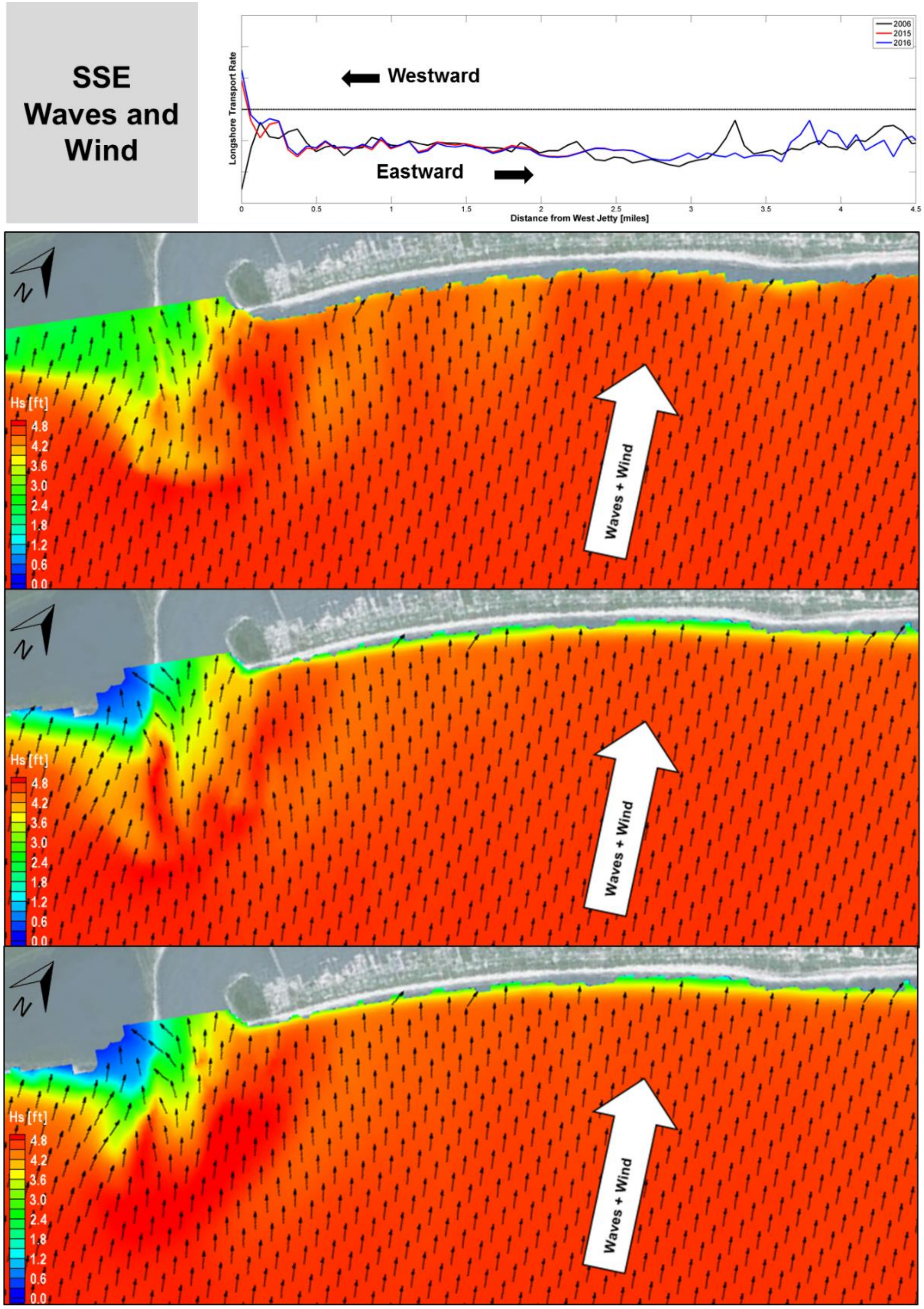


Figure 32. Nearshore wave significant wave height and longshore sediment transport for SSE wind and wave condition using 2006, 2015, and 2016 bathymetry.

2 Technical Assessment of the Grand Isle Federal Levee Project

The goal of this Section is to complete a technical assessment and re-evaluation of the Grand Isle Federal Levee Project developed by the USACE. The first project (GI-01) was performed in April 2008 and concluded in November 2008. Since then, 4 additional projects were constructed concluding with the Grand Isle Federal Dune Crossover Repair Project (GI-01D) which completed in March 2016. A brief description of each project is provided below (CHE, 2016):

- GI-01: following the 2008 storm season, rehabilitation alternative to damaged burrito consisted of replacing sections of burrito with dredge-filled geotube and beach re-nourishment. Repairs of 23 breakwaters was also included.
- GI-01A: following the 2009 storm season, repairs of damaged burrito consisted of replacing with geotube.
- GI-01B: following the 2009 storm season, rehabilitation of the West Jetty by rock placement.
- GI-01C: following the 2012 storm season, repairs consisted of replacing the damaged geotube and placing stone armoring on seaward side of levee dune. The GI-01C differed from the GI-01 and GI-01A project designs with the construction of the stone armoring; no design documentation is available that supports this change in approach
- GI-01D: construction of one timber crossover in conjunction with four additional ACB turnarounds along the Grand Isle shoreline

Under 2014 OMRRM, the obligations of the Non-Federal Sponsors are as follows (USACE, 2014):

- Non-Federal Sponsors Monitoring Program
  1. Quarterly visual inspections
  2. Biannual collection of quantitative survey data
     a. Cross-sections of dune/beach/near shore profiles,
     b. Centerline profile elevations on the dune crown,
  3. Every three years, profile and cross-section surveys of the East Jetty, West Jetty, and Detached Segmented Breakwaters.
  4. Evaluation and action: by comparing successive surveys and inspection reports, measurements and observations can be utilized to detect developing problems. The following benchmarks are to be used to determine when remedial action is necessary:
     a. When portions longer than 100 linear feet of polyurea-coated geotextile tube, scour apron, anchor tube or geo-burrito have become exposed.
     b. When survey comparisons show a shoreline retreat of 100 feet or more at any elevation.
  5. Semi-annual inspection of the dune vegetation.
     a. Restore to a vegetated state when non-vegetated areas are larger than 100 sq mi

- Dune and beach re-nourishment
  - Based on whole-island net sediment transport rate of 90,000cd/yr, periodic nourishment may be anticipated on a 4-year cycle barring impacts from major storms (Batten et al, 2004)
  - The beach shall be periodically nourished when survey comparisons show a shoreline retreat of 100 feet or more at any elevation.
  - If storm activity is above normal, nourishment of the beach may be necessary at intervals more frequent than the estimated four (4) year cycle to preserve the integrity of the beach and dune cross section.

The western end of the Grand Isle shoreline has sustained successive damage over years and all repairs intended to restore the damaged system to the original project conditions able to withstand a storm surge with a 50-year return period (USACE, 2008), have not been able to stabilize the shoreline. The available literature does not provide clarification on the 50-year return period design conditions. Since documents are not available, it has been assumed that conditions from the USACE (1979) and Batten (2004) reports were considered as the design conditions.

The shoreline change analysis (Section 1.7.1) presented in this report clearly indicates the presence of an erosional hotspot along the western end of Grand Isle shoreline around the rock revetment, shown on Figure 33. The bottom morphology analysis (Section 1.7.2) indicates the evolution of the Caminada ebb shoal over the years resulting in the nearshore refraction and focusing of waves on the southwest end of Grand Isle. The latter has led to changes in sediment transport patterns (section 1.8.1) and an associated divergent node located on the southwest end of the island. The presence of this diversion node is causing this section of the shoreline to become the hotspot of erosion and experience the highest shoreline erosion rates along the entire Grand Isle shoreline.

Figure 33. Location of critically eroded dune on southwest end of Grand Isle

As shown on Figure 34 and based on the shoreline change analysis results (Section 1.7.1), the frequency of nourishment required to prevent shoreline retreat of 100 ft as stipulated on the

2014 OMRRRM is less than a 4-year cycle between 0 to 0.5 mi from the West Jetty or 10+00 to 35+00. The results indicate a 2-year nourishment cycle at the revetment shoreline would be required to maintain 2014 OMRRRM shoreline requirements.

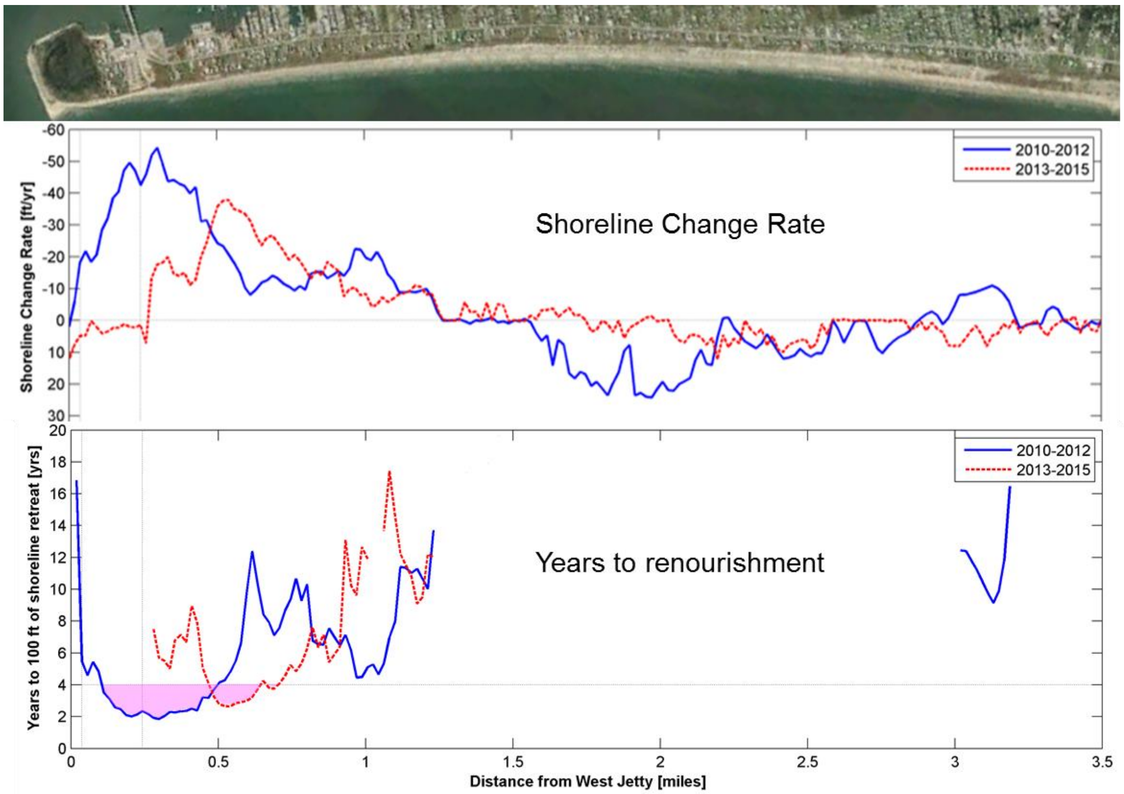


Figure 34. Time in years for 100 ft of shoreline to retreat based shoreline change analysis results

3 Conceptual Alternatives Development

Potential long-term solution alternatives at a conceptual level have been developed to maximize stabilization of the Grand Isle shoreline and mitigate deficiencies based upon the understanding of the physical processes along the Grand Isle shoreline developed in this study. It is understood that any structure that retains sand within the project shoreline may cause increased erosional downdrift impacts, therefore these impacts were evaluated while during alternative analysis. The goals of the alternatives are as follows:

- Protect levee dune
- Decrease maintenance interval (stabilize shoreline)
- Minimize capital costs
- Retain recreational beach
- Minimize downdrift impacts

Four different conceptual alternatives with sub-alternatives have been proposed.

1. GI-01C + Mitigation Dune + Beach Nourishment
   a. Sand Only
   b. Revetment Core
2. Beach Nourishment
   a. GI-01A 2009 Template
   b. GI-01A 2009 + Mitigation Fill
3. Segmented Offshore Breakwaters
   a. GI-01C 2013 Template Replaced
   b. GI-01C 2013 + Mitigation Dune + Beach
4. Headland Breakwaters
   a. GI-01C 2013 Template Replaced
   b. GI-01C 2013 + Mitigation Dune + Beach

The conceptual alternatives will be refined during the alternatives analysis. The templates used in the alternatives are based around the GI-01 project templates, which are shown in Figure 35 and Figure 36 for GI-01A and GI-01C, respectively.

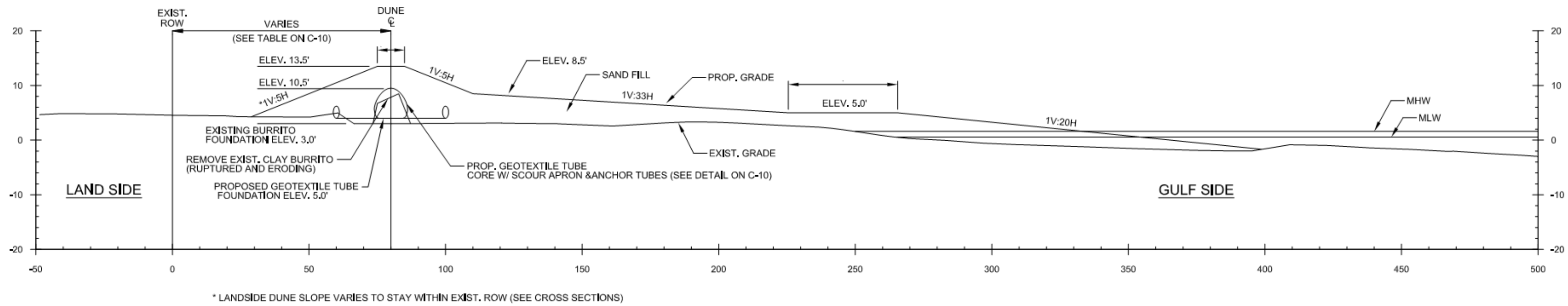


Figure 35. GI-01A Template. Taken from Grand Isle and Vicinity Hurricane Protection Project Station 0+00 to 386+00 along Grand Isle Beach. Rehabilitation of Hurricane Gustav and Hurricane Ike Damage drawings, dated February 2009.

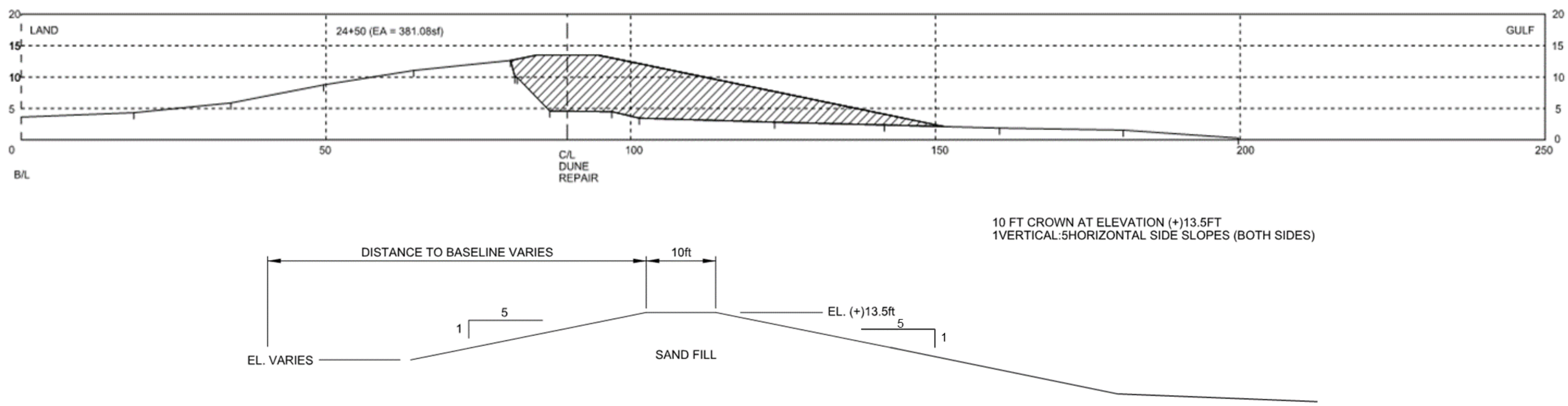


Figure 36. GI-01C Template (top) and detail (bottom). Taken from Grand Isle and Vicinity Hurricane Protection Project Dune Repair and Armoring drawings, dated April 2013.

3.1 Alternative 1: GI-01C + Mitigation Dune + Beach Nourishment

- **Alternative 1A: GI-01C + Mitigation Dune + Beach Nourishment, Sand Only**

Alternative 1A, as shown at the top of Figure 37 and the top of Figure 38, consists of rebuilding the GI-01C template (Figure 36). This alternative is considered to provide a comparison between the USACE design and more robust alternatives.

- **Alternative 1B: GI-01C + Mitigation Dune + Beach Nourishment, Revetment Core**

Alternative 1B, as shown at the bottom of Figure 37 bottom and the bottom of Figure 38, consists of rebuilding the GI-01C template (Figure 36) and adding a revetment core integrated into the dune. While experience has shown the GI-01C template will erode quickly, the rock core extends the protection through the hotspot to stabilize the shoreline from erosion that would impact landward infrastructure.

3.2 Alternative 2: Beach Nourishment

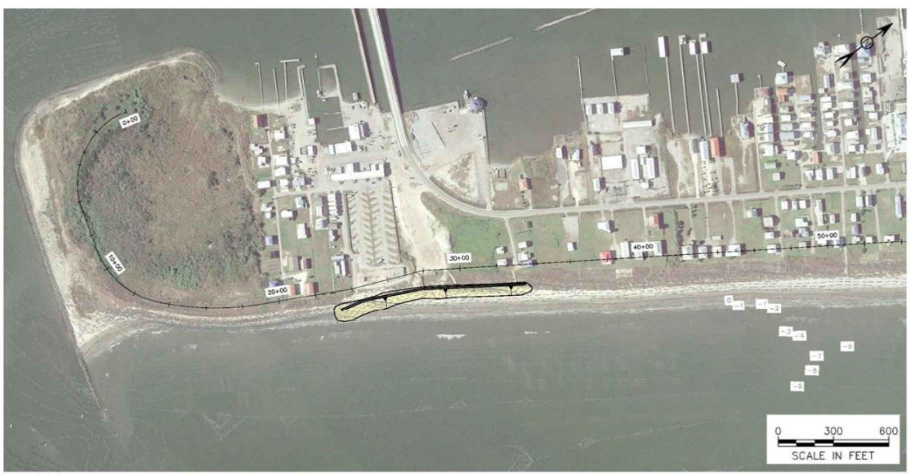
- **Alternative 2A: Beach Nourishment, GI-01A 2009 Template**

Alternative 2A, as show at the top of Figure 39 and the top of Figure 40, consists of a beach nourishment following the GI-01A template (Figure 35). Compared to Alternative 1, the large nourishment is expected to have a longer lifetime and a wider beach and therefore, would have a lower renourishment interval (lower maintenance component).

- **Alternative 2B: Beach Nourishment, GI-01A 2009 Template + Mitigation Fill**

Alternative 2B, as shown on the bottom of Figure 39 and the bottom of Figure 40, consists of a beach nourishment following the GI-01A template (Figure 35) and a mitigation fill, or additional fill to increase the template lifetime. Comparisons between Alt 2A and 2B will help illustrate the benefits and costs of varied fill volumes.

ALT 1.A - GI-01C (2013) + Mitigation Dune + Beach Nourishment Sand Only



ALT 1.B - GI-01C (2013) + Mitigation Dune + Beach Nourishment Revetment Core

Figure 37. Site plan alternative 1: GI-01C + Mitigation Dune + Beach Nourishment. Alternative 1A: Sand Only (top) and Alternative 1B Revetment Core (bottom)

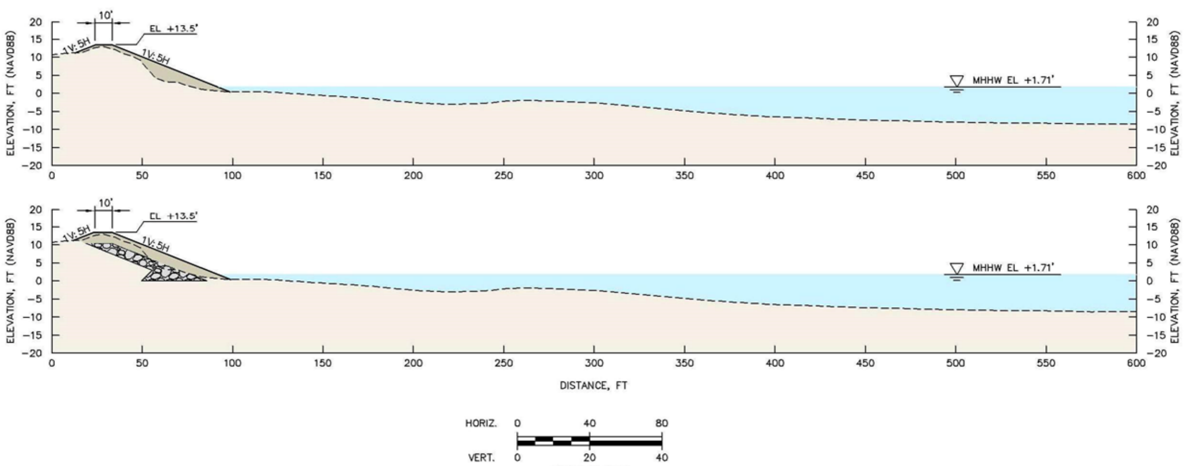
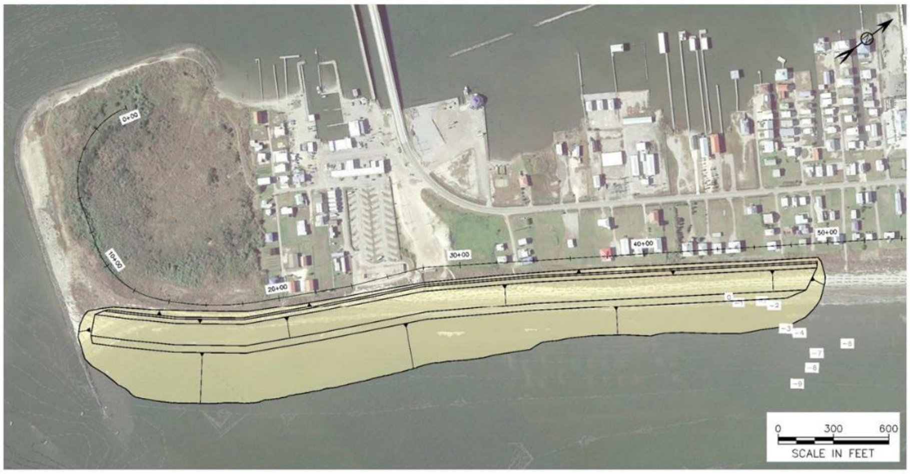
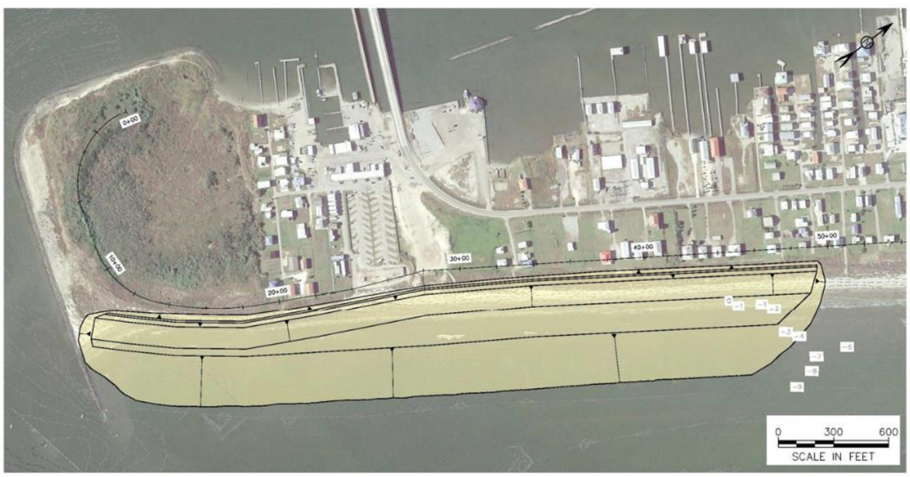


Figure 38. Cross-section alternative 1: GI-01C + Mitigation Dune + Beach Nourishment. Alternative 1A: Sand Only (top) and Alternative 1B Revetment Core (bottom)



ALT 2.A - Beach Nourishment GI-01A (2009) Template



ALT 2.B - Beach Nourishment GI-01A (2009) Template + Mitigation Fill

Figure 39. Site plan alternative 2: Beach Nourishment. Alternative 2A GI-01A 2009 Template (top) and Alternative 2B GI-01A 2009 Template + Mitigation Fill (bottom).

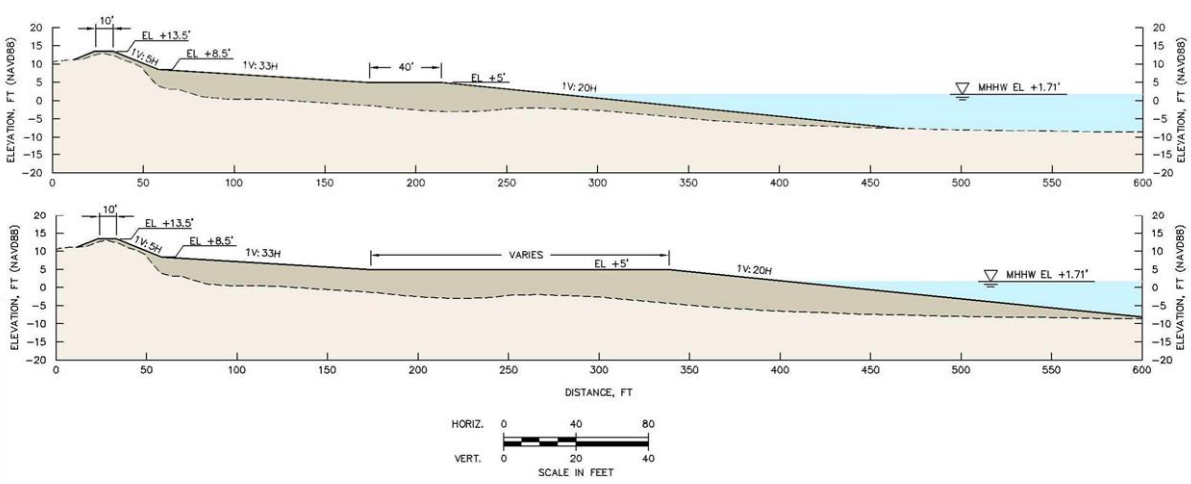


Figure 40. Cross-section alternative 2: Beach Nourishment. Alternative 2A GI-01A 2009 Template (top) and Alternative 2B GI-01A 2009 Template + Mitigation Fill (bottom).

3.3 Alternative 3: Segmented Offshore Breakwaters

- **Alternative 3A Segmented Offshore Breakwaters, GI-01C 2013 Template Replaced**

Alternative 3A (Figure 41 top and Figure 42 top) consists replacing the existing dune with the GI-01C template (Figure 36) in combination with segmented offshore breakwaters. The maintenance component for the beach should be reduced as expected accretion features resulting from the breakwaters such as salient or tombolos on the lee side of the breakwaters would help stabilize the beach. Added benefits of sediment bypassing are expected upon optimization of offshore breakwaters placement location and length/gap ratio on eastern end of breakwater field.

- **Alternative 3B Segmented Offshore Breakwaters, GI-01C 2013+Mitigation Dune+Beach**

Alternative 3B (Figure 41 bottom and Figure 42 bottom) consists of replacing the existing dune with the GI-01C template (Figure 36) in combination with segmented offshore breakwaters and a beach fill. Compared to Alternative 3A, this alternative provides a wider beach at a higher capital cost. Similar to Alternative 3A, the maintenance component for the beach should be significantly less than Alternatives 1 and 2 due to expected benefits of breakwaters.

3.4 Alternative 4: Headland Breakwater

- **Alternative 4.A Headland Breakwaters, GI-01C 2013 Template Replaced**

Alternative 4A, as shown at the top of Figure 43 top and the top of Figure 44, consists replacing the existing dune with the GI-01C template (Figure 36) in combination with headland breakwaters which mimics the effects of a pocket beach. Hence, the maintenance component for the beach should be minimal since the headland breakwaters would be able to stabilize the shoreline; headland / pocket beach features are among the most stable systems as little transport occurs out of the systems.

- **Alternative 4.B Headland Breakwaters, GI-01C 2013 + Mitigation Dune + Beach**

Alternative 4B, as shown at the bottom of Figure 43 bottom and the bottom of Figure 44) consists of replacing the existing dune with the GI-01C template (Figure 36) in combination with headland breakwaters and a beach fill. Compared to Alt 4A, this alternative provides a wider beach at a higher capital cost, but with likely increased protection and recreational use. Similar to Alt 4A, the maintenance component for the beach should be less than alternatives 1 and 2 due to expected stabilization features of the headland breakwaters.

ALT 3.A GI-01C - Segmented Offshore Breakwaters GI-01C (2013) Template Replaced Conceptual Template

ALT 3.B GI-0C - Segmented Offshore Breakwaters GI-01C (2013) + Mitigation Dune + Beach Conceptual Template

Figure 41. Site plan alternative 3: Segmented Offshore Breakwaters. Alternative 3A GI-01C 2013 Template Replaced (top) and Alternative 3B GI-01C 2013 + Mitigation Dune + Beach (bottom).

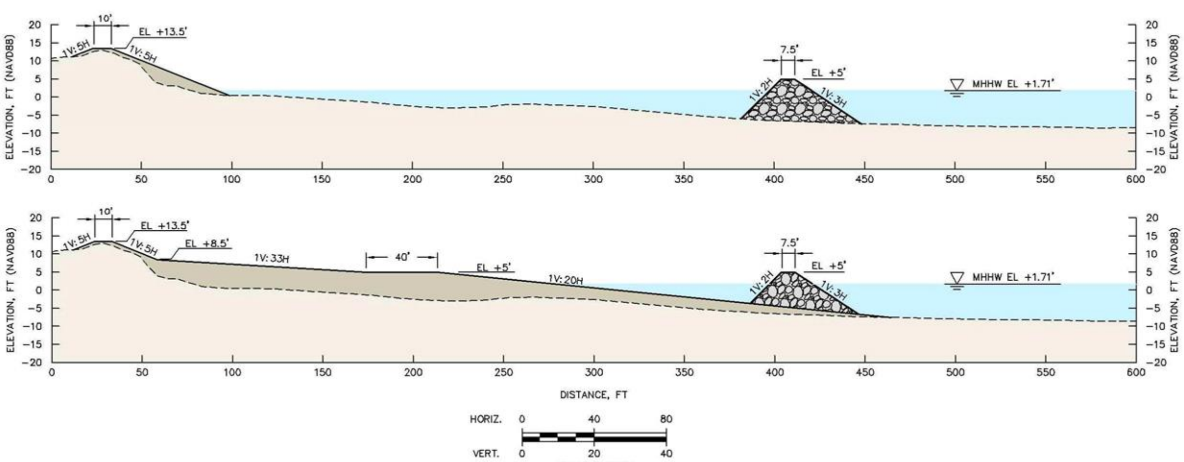
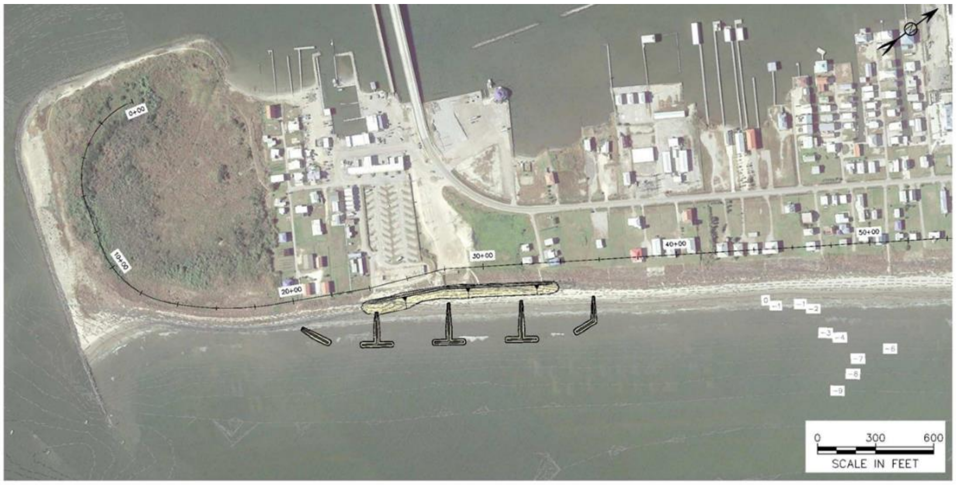


Figure 42. Cross-section alternative 3: Segmented Offshore Breakwaters. Alternative 3A GI-01C 2013 Template Replaced (top) and Alternative 3B GI-01C 2013 + Mitigation Dune + Beach (bottom).



ALT 4.A GI-01C - Headland Breakwaters GI-01C (2013) Template Replaced Conceptual Template

ALT 4.B GI-0C -Headland Breakwaters GI-01C (2013) + Mitigation Dune + Beach Conceptual Template

Figure 43. Site plan Alternative 4: Headland Breakwaters. Alt 4A GI-01C 2013 Template Replaced (top) and Alt 4B GI-01C 2013 + Mitigation Dune + Beach (bottom)

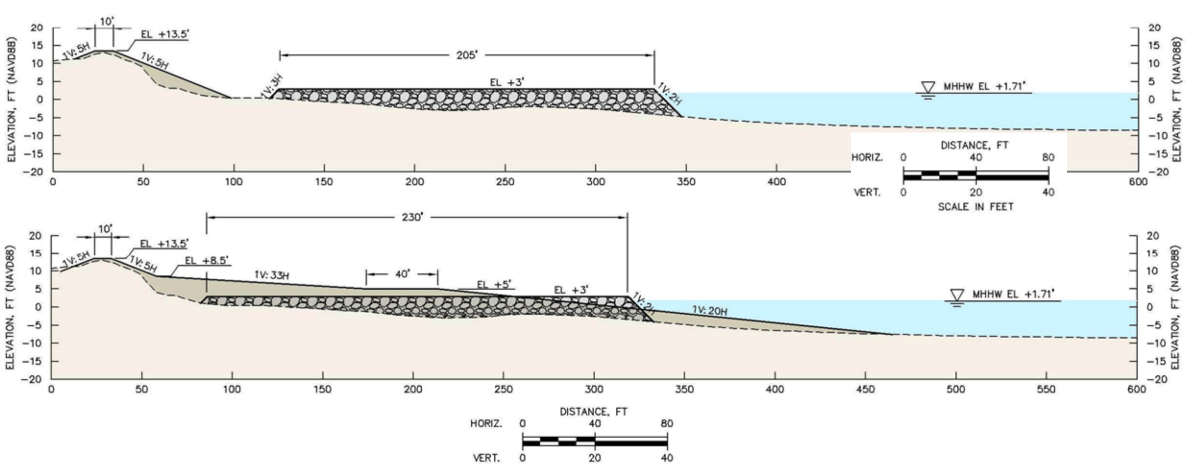


Figure 44. Cross-section Alternative 4: Headland Breakwaters. Alt 4A GI-01C 2013 Template Replaced (top) and Alt 4B GI-01C 2013 + Mitigation Dune + Beach (bottom).

4 Alternative Analysis

To determine the best performing alternative, a direct comparison between the concepts was made. For this comparison, the following criteria were evaluated:

1. Performance criteria
   a. Cross-shore response: dune retreat based on storm impact
   b. Shoreline retreat based on long-term morphology
2. Cost
3. Recreational value

4.1 Performance Criteria

The performance criteria will be used as a basis to analyze how the alternatives perform in meeting the project goals. The primary project goal is to stabilize shoreline change and reduce erosion at the western end of Grand Isle. According to the 2014 USACE OMRRR, (USACE, 2014) action is to be taken when (1) portions longer than 100 linear feet of the geotube, scour apron, or burrito have become exposed or (2) when survey comparisons show a shoreline retreat of 100 feet or more at any elevation. Therefore, the ability of each alternative to maintain the integrity of the dune template and to reduce erosion will be evaluated as a performance criteria.

The dune retreat at any point on the template will be evaluated by assessing the before and after profiles subjected to 1-, 2-, 5-, 10-, 25-, and 50- year storms. The USACE SBEACH model was employed to assess this criterion. Retreat rates for each of the storms were compared for each of the alternatives as a measure of performance. In addition, a statistical approach was used to annualize the dune retreat as a cumulative measure to evaluate performance.

Shoreline retreat was evaluated by modeling the shoreline changes with the GENCADE model. The performance was evaluated based on the time it took for the given alternative to retreat to various trigger lines. These trigger lines were set to resemble the USACE maintenance triggers in the 2014 OMRRR. The first trigger line used for analysis was set at the beginning of encroachment into the GI-01C template (essentially the base of the existing dune). This trigger line was approximated as the vegetation line and was traced from a 2015 aerial. The second trigger line was set 100 feet landward of the seaward extents of the GI-01A template. This metric was only used for alternatives containing a beach fill. Finally, the impacts to the adjacent, downdrift shoreline were quantified.

4.1.1 Cross-shore Response to Storm Impact

Cross-shore morphological modeling was conducted to analyze the cross-shore profile response of different alternatives for different storm events. The cross-shore morphology was simulated using the numerical model SBEACH (Storm-induced BEAch CHange). SBEACH is a 2-D numerical model that simulates cross-shore beach profile changes during storm events. The model assumes all transport is in the cross-shore direction and does not consider downdrift transport or localized effects. SBEACH incorporates overwash processes to simulate landward transport when dunes or beach berms are overtopped (Larson *et al.*, 2004).

Model setup included selecting proper input conditions including cross-section and storm event parameters which include water surface elevation, wave height, and a wave period time series. A cross-sectional profile extracted from the most recent survey along the project site was used for the modeling. The cross-sectional profile extended to 55,000 feet offshore to a depth of 70 ft NAVD88. For the analysis, the numerical model was run with (for visualization) and without (to get the true dune response) the geotube core. The geotube core was assumed as an elliptical tube having a width of 10 feet and height of 5.5 feet with a top elevation of 10.5 feet NAVD88.

The storm hydrograph from Hurricane Danny (7/1997) was selected as the design storm, and is shown in Figure 45. Danny was selected because it is typical of a slow-moving storm that approaches almost directly towards Grand Isle. Danny was a fairly weak storm, however, the storm conditions (water level, wave height, wave period) were scaled to match storm conditions calculated for return periods of 1, 2, 5, 7.5, 10, 15, 20, 25, 50, and 100 years computed previously in the Coastal Processes Analysis task (Section 1) and are shown in Appendix B.

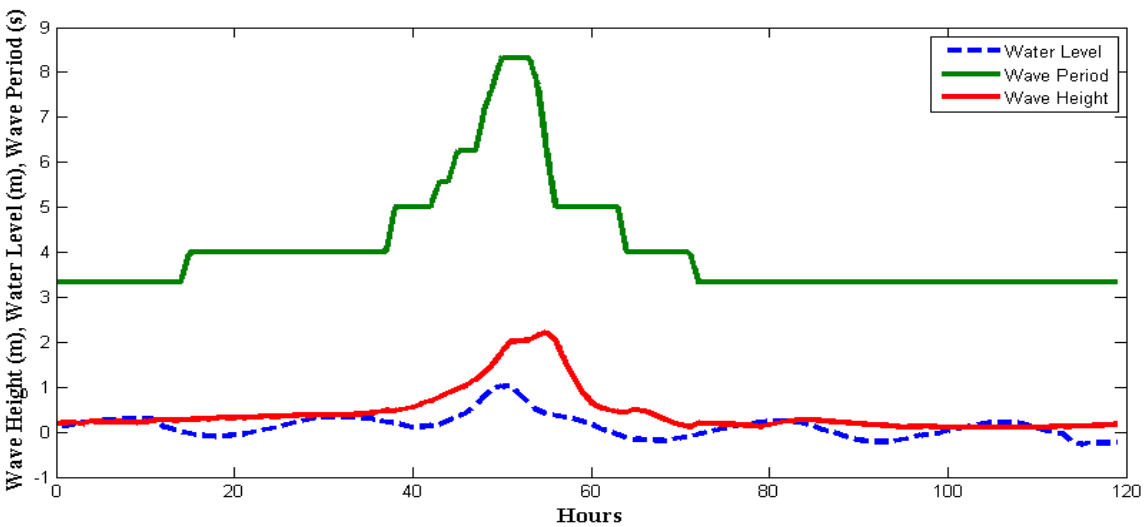


Figure 45. Hydrograph of Hurricane Danny.

The transmission coefficients for different storm events were computed for the hard-structural alternatives (breakwater and t-head groins) and were used to scale the peak of the respective storm events. Since presence of the structure, only alters the wave heights impacting the shoreline, the wave period and water surface elevation time series remains the same for all the hard-structural alternative runs as that for cases without any hard-structural alternative.

The intent of the SBEACH modeling was to simulate the storm impact to determine the duration it will take to expose the geotube core and the amount of sand that will be required to repair the dune to design template. The SBEACH model in this analysis used identical parameters as described in Larson *et al.* (2004) including the sediment transport coefficient, K, which Larson *et al*. recommends as $K = 2.5 \times 10^{-6}$ during overwash conditions. Other parameters were selected based on the calibration run (Alt 1a): match the observed dune erosion at site (2016-2017), where the geotube core was exposed in less than a year of placing the sand dune and are shown in Table 11.

Table 11. Input parameters used in SBEACH modeling

Parameter	Value
Effective grain size (mm)	0.15
Maximum slope prior to avalanching (degrees)	30
Transport Rate Coefficient (m^4/N)	2.5e-006
Overwash transport parameter	0.005
Coefficient for slope dependent terms (m^2/S)	0.002
Transport rate decay coefficient multiplier	0.5
Water temperature (degrees C)	20

SBEACH results were used to calculate contour retreat and associated volume lost from the dune for different alternatives for different storm events. Contour retreat was computed at an elevation of 7.8 feet NAVD88 (elevation where the geotube core is closest to the dune face assuming an elliptical shape for the dune core). For the structural alternatives, to account for the gap in alternatives, the results were computed by taking the average performance of the profile with and without the hard-structure. The results of contour retreat are shown in Table 12 and shown in Appendix B (retreat plots for each alternative for each modeled storm event).

Table 12. Contour retreat (ft) for different storm events using SBEACH.

Alternative	Return Period [yrs]								
	1	2	5	7.5	10	15	20	25	50
Alt 1A	9	14	24	26	67	67	67	67	67
Alt 2A	0	0	4	5	6	8	9	9	87
Alt 2B	0	0	1	1	2	2	2	2	87
Alt 3A	5	9	17	20	42	44	46	48	67
Alt 3B	0	0	2	3	4	5	6	15	87
Alt 4A	5	10	18	21	43	46	50	67	67
Alt 4B	0	0	2	3	4	6	7	10	87

SBEACH modeling results for Alt 1A (existing) for selected storm events are shown in Figure 46. The results show that a storm with a return period of ~ 7.5 yrs will expose the geotube core but as discussed in following sections, this doesn't indicate the annualized contour retreat based on the probabilistic nature of occurrence of other storm events.

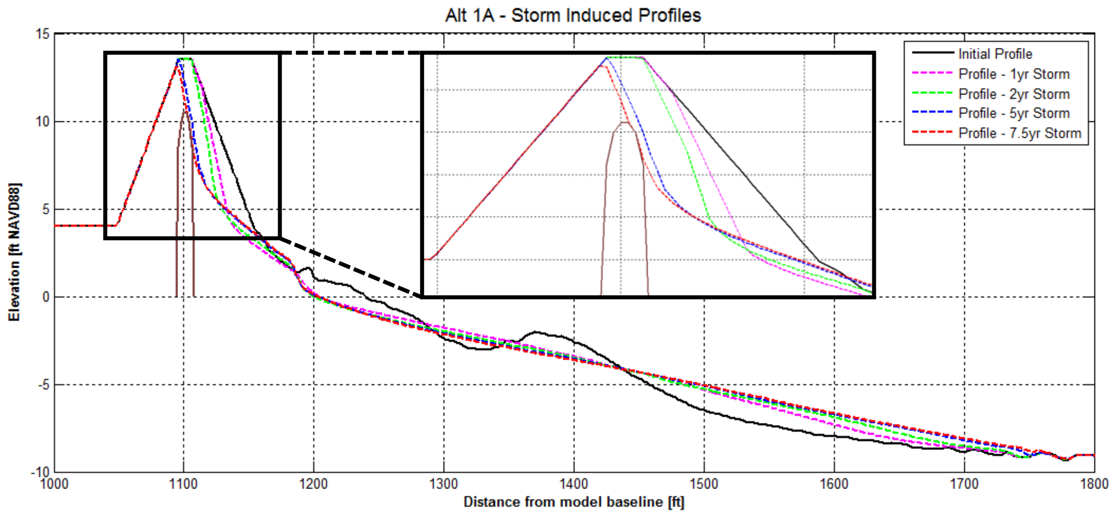


Figure 46. Existing condition (Alt 1A) initial and storm affected profiles from SBEACH modeling for different storm events.

Since the SBEACH results represent results associated with individual storm events, a statistical annualized approach was used to represent progressive erosion of contour and to account for randomization of storms. Under this statistical approach, probability of occurrence for a storm of a given magnitude (return period) is computed in any given year as a binomial distribution given by equation below and used along the shoreline erosion associated with respective storm event (shown in Table 12) to integrate the total contour retreat expected in any given year.

$$P = 100\left(1 - \left(1 - \frac{1}{Tr}\right)^{L}\right)$$

where P is the probability of occurrence, T_r is the return period, and L is the total length of time (here number of years). The probability of occurrence for different return period storms in given in Table 13.

Table 13. Probability of Occurrence of a storm event of a given period over a 50-yr period.

Length of Time [yrs]	Return Period [yrs] 1	2	5	10	25	50
1	100.0%	50.0%	20. 0%	10.0%	4.0%	2.0%
2	100.0%	75.0%	36.0%	19.0%	7.8%	4.0%
3	100.0%	87.5%	48.8%	27.1%	11.5%	5.9%
4	100.0%	93.8%	59.0%	34.4%	15.1%	7.8%
5	100.0%	96.9%	67.2%	41.0%	18.5%	9.6%
.	.	.	.	.	.	.
.	.	.	.	.	.	.
24	100.0%	100.0%	99.5%	92.0%	62.5%	38.4%
25	100.0%	100.0%	99.6%	92.8%	64.0%	39.7%
26	100.0%	100.0%	99.7%	93.5%	65.4%	40.9%
.	.	.	.	.	.	
.	.	.	.	.	.	
48	100.0%	100.0%	100.0%	99.4%	85.9%	62.1%
49	100.0%	100.0%	100.0%	99.4%	86.5%	62.8%
50	100.0%	100.0%	100.0%	99.8%	87.0%	63.4%

The results from the statistical annualized approach for contour retreat at the 7.8ft NAVD88 contour are shown in Table 14. The distance of contour retreat before the geotube core gets exposed for different alternatives is 28ft for alternatives with small to no nourishment (Alt 1A, Alt 3A, and Alt 4A) and 48ft for alternatives with larger nourishment (Alt 2A, Alt 2B, Alt 3B, and Alt 4B).

Table 14. Annualized contour retreat (ft) for different alternatives. Red text indicates exposed core.

Length of Time [yrs]	Alternative 1A	2A	2B	3A	3B	4A	4B
1	31	4	3	20	4	20	4
2	38	8	6	25	7	27	7
3	43	11	8	30	10	31	10
4	48	14	11	33	13	35	13
5	52	16	13	37	15	39	15
.							
.							
24	74	48	44	60	48	63	47
25	74	49	46	60	49	64	48
26	74	50	47	60	50	64	49
27	75	51	48	61	51	65	50
28	75	52	49	61	52	65	51
29	75	53	50	62	53	65	52
30	75	54	51	62	54	66	53

The annualized contour retreat rates shown in Table 14 indicate that in one year, a contour retreat of 30.9 ft can be expected for Alt 1A (existing condition) which implies that the geotube core will exposed because the distance for exposure for this alternative is only 28.5 ft. This matches what has been observed at the site where the geotube core was exposed within a year of rebuilding the dune (June 2016 to March 2017). Note that the time to dune core exposure is a relative metric only, it does not account for the cumulative shoreline erosion which results in more damage from similar storm impacts. Over a period of several years, the beach will have retreated and using the post-construction template is a poor representation. However, the results show the benefits of a wider beach which clearly increase the dune longevity reduce dune maintenance requirements.

Similar to contour retreat, volume lost from the dune was also computed by taking the difference in area above 5' NAVD88 elevation (as that is the designed elevation of the beach berm) for the pre- and post-storm profiles from SBEACH. These were similarly tabulated using the probabilistic approach into annualized volume loss (cy/ft) from the dune for different alternatives and is shown in Table 15.

Table 15. Annualized volume loss (cy/ft) for different alternatives.

Length of Time [yrs]	Alternative						
	1A	2A	2B	3A	3B	4A	4B
1	9	2	1	6	2	6	2
2	11	3	1	8	3	8	3
3	13	4	2	9	4	9	4
4	14	5	2	10	4	11	4
5	15	6	3	11	5	12	5
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
8	18	8	4	13	7	14	7
9	18	8	5	14	7	15	7
10	19	9	5	14	8	15	8
.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.
29	22	15	11	19	15	20	15
30	22	16	11	19	15	21	15

Alt 3A and 4A which are essentially Alt 1A with a segmented breakwater and headland breakwater, respectively show increase in time until the geotube core gets exposed as they reduce the wave energy impacting the dune. Table 14 shows that for both Alt 3A and 4A, the geotube core will get exposed in about a 3 yr time, but the volume lost for Alt 4A when the geotube core is exposed is slightly larger than Alt 3A (9.4 cy/ft v/s 8.8 cy/ft at year 3) as Alt 4A reduces the waves slightly less than Alt 3A. Figure 47 depicts the comparative performance of these three alternatives when impacted by the same storm event.

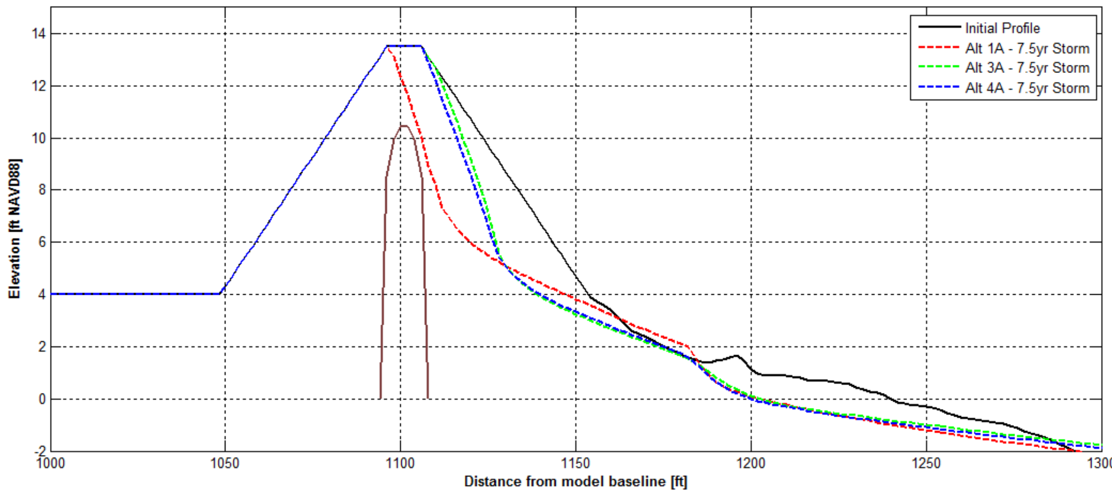


Figure 47. Alt 1A, 3A and 4A performance when impacted by a 7.5 yr storm event using SBEACH.

The results for alternatives with additional beach berm constructed in front of the dune (Alt 2A, Alt 2B, Alt 3B and Alt 4B) show that the time it will take to expose the geotube core is greater than 25 years (Table 14) as the available sand in the beach berm erodes (before the dune gets eroded) due to storm events. Results for Alt 2B impacted by a 25 yr storm event (Figure 48) shows that even though the dune was not eroded, the berm slope has receded by more than 100 feet. This implies that by constructing the beach in front of the dune, the erosional impacts can be directed to the beach and will help in maintaining the dune for a longer duration.

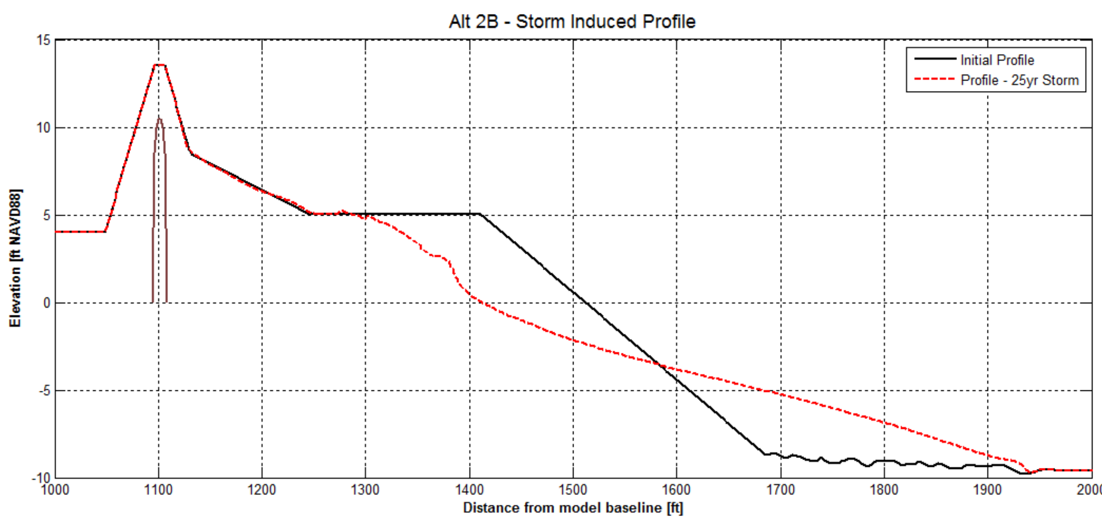


Figure 48. Alt 2B initial and storm affected profile from SBEACH for 25yr storm event.

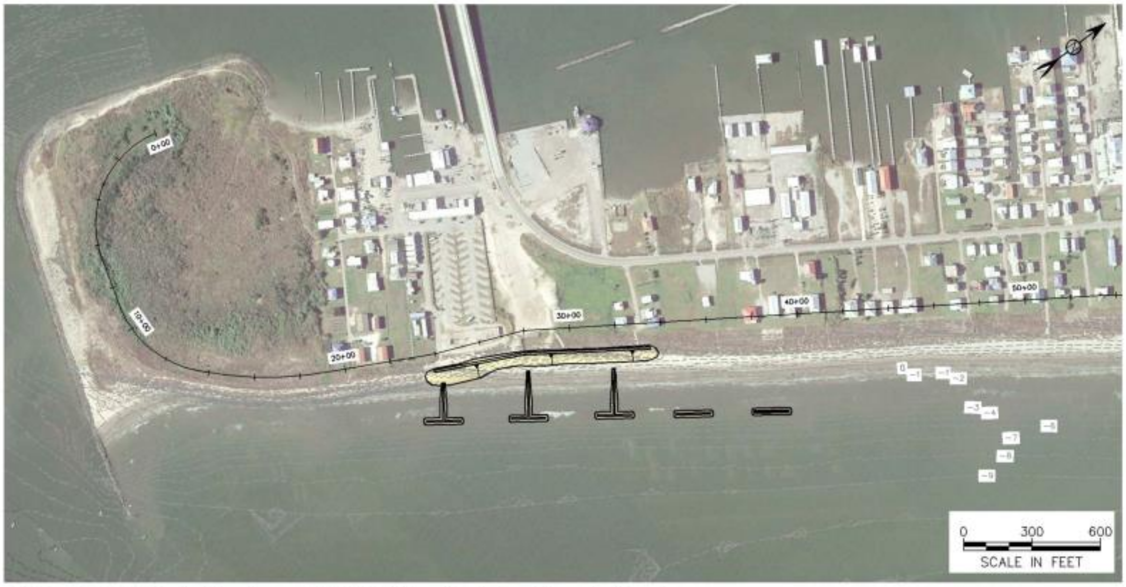
The cross-shore morphology modeling results show that Alt 1A (existing condition) will have its geotube core exposed within a year of replacing the sand dune and therefore, requires some mitigation measures. Alt 1B, which places a revetment face in front of the geotube core, doesn't reduce the retreat rates but provides a more solid defense for shoreline protection that does not require immediate replacement of sand to retain its protective function. Alt 3A and Alt 4A, increases the time required (maintenance cycle) for dune repair due to the reduced wave action by the presence of hard-structures in front of the dune and the existing beach maintained for a longer duration. Beach berms placed in front of the dune (Alt 2A and Alt 2B) increase the maintenance cycle duration considerably as the beach berm (rather than the dune) gets eroded due to the storm impact. Constructing a hard structure in front of the beach berm (Alt 3B and Alt 4B) reduces the wave energies impacting the berm and therefore reduces the overall sand volume lost when compared to similar alternatives without the hard structure (Alt 2A and Alt 2B).

4.1.2 Shoreline Response

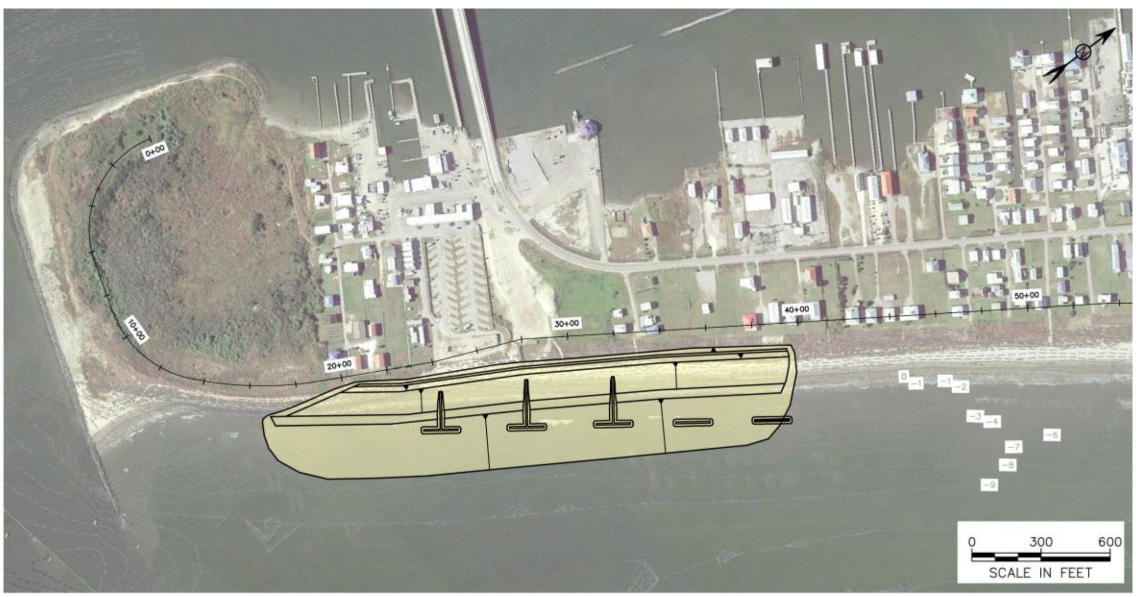
An analysis of the shoreline response resulting from each alternative was conducted using the calibrated and validated GenCade model setup described in Section 1.7.3. The shoreline response for all alternatives, including the Future Without Project (FWOP) scenario, were simulated for a 25-year timeframe, with all beach fills, groins, and breakwaters built into the model setup. Performance metrics were developed to determine the effectiveness of each alternative. Performance metrics include: (1) time to GI-01C template (approximated as the existing vegetation line as traced on the 2015 aerial), (2) time to GI-01A template minus 100 feet of erosion (for applicable alternatives), (3) down drift effects.

The trigger for re-nourishment time was taken as the time for the initial shoreline to reach the existing vegetation line (approximately the GI-01C template). The re-nourishment volume is the volume necessary to restore this eroded shoreline to the initial shoreline for each project. The re-nourishment volumes are incorporated into the maintenance costs described in Section 4.2. Note that Alt 1A and Alt1B were modeled as the FWOP scenario, since GenCade does not allow for inclusion of dunes in the model setup, and no beach fill or rubble mound structures are included in the design of these alternatives.

To provide an initial estimate of project performance, the shoreline positions at year 4 during the model simulation were extracted. The 4-year shoreline was used as a first approximation of alternative performance, as four years is the current 2015 OMRRR recommended maintenance period. The results of the 4-year shoreline analysis were used to compare project performance to the FWOP scenario. These results were used as an initial comparison only, and were not used as a final performance metric. The results of this analysis are summarized in Appendix C. Note that upon initial inspection of the results, Alternative 4A and 4B showed an increased obstruction of longshore transport compared to the other alternatives. This resulted in large downdrift erosion to the east of the project site. Therefore, these two alternatives were modified. These modifications included re-arrangement of the rubble mound structures for Alternative 4A and 4B. The modified alternatives referred to as Alternative 4A_v1 and 4B_v1 (shown in Figure 50) were carried through for further analysis instead of the original alternatives 4A and 4B. The modified alternatives showed less downdrift erosion than the original Alternative 4A and 4B configurations.



ALT 4.A _v1 GI-01C - Headland Breakwaters GI-01C (2013) Template Replaced Conceptual Template



ALT 4.B_v1 GI-0C -Headland Breakwaters GI-01C (2013) + Mitigation Dune + Beach Conceptual Template

Figure 49. Site plan Alternative 4: Headland Breakwaters. Alt 4A_v1 GI-01C 2013 Template Replaced (top) and Alt 4B_v1 GI-01C 2013 + Mitigation Dune + Beach (bottom)

4.1.2.1 Time to GI-01C Template

The GI-01C template was approximated as the vegetation line as shown on the 2015 aerial. This line was used as the trigger for re-nourishment. The time to the vegetation line and the associated re-nourishment volume were used to quantify performance of the alternatives. The re-nourishment volume was computed as the volume necessary to restore the shoreline at the trigger time to initial project shoreline. The results of this analysis are shown below in Table 16. Note that the volume of fill shown is the volume of fill necessary to return the project shoreline to the initial shoreline (alternative template) shown on Figure 50 through Figure 56. The project shoreline is defined from the Caminada Pass Jetty to the easternmost portion of the beach fill or hard structure in each alternative. The volume associated with downdrift erosion is not included in Table 16. A separate analysis of the downdrift erosion volume is conducted in Section 4.1.2.3. If the shoreline accretes past the initial shoreline (seaward of the initial shoreline position) due to structures, this excess volume was not subtracted from the total amount of fill needed. This volume was not included while computing the volume required to restore the project shoreline to its initial condition as it was assumed that any accretion seaward of the initial project shoreline (i.e. salient/tombolo formation) would not be dredged in this renourishment cycle.

Table 16: Time to GI-01C Template (Vegetation Line) and associated volume of fill necessary to restore the beach to the initial project shoreline.

Alternative	Time to Vegetation Line [yrs]	Volume of Fill [cy]
FWOP/Alt. 1A/ Alt. 1B	1.5	89,000
Alt. 2A	7.1	427,000
Alt. 2B	10.3	764,000
Alt. 3A	9.8	86,000
Alt. 3B	13.5	310,000
Alt. 4A_v1	7.1	54,000
Alt. 4B_v1	9.8	231,000

The time to GI-01C template analysis shown in Table 16 illustrates the re-nourishment period and associated volume for each alternative. It is apparent that while the T-Head groin structures (present in Alt 4A_v1 and Alt 4B_v1) have slightly shorter, less desirable re-nourishment cycles than the breakwater alternatives (Alt 3A and Alt 3B). However, Figure 50 through Figure 56 show that despite the shorter re-nourishment cycle, these alternatives hold much of the project shoreline in place.

It is also apparent that placing beach fill without any structures will result in large maintenance costs due to the erosion along the project shoreline that will occur without any hard structures. The lifetime cost of each alternative is further investigated in Section 4.2. Figure 50 through Figure 56 show the shoreline response for each alternative at the time when the modeled shoreline hits the vegetation line, as well as the FWOP shoreline at this timestep in the model simulation.

Figure 50. Shoreline response for FWOP/ Alt 1A/ Alt 1B, at the model timestep when the shoreline hits the vegetation line (1.5 years). Vegetation line (green), FWP/ Alt 1A/ Alt 1B shoreline (red), and FWP initial shoreline (yellow).

Figure 51. Shoreline response for Alt 2A at the model timestep when the shoreline hits the vegetation line (7.1 years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

Figure 52. Shoreline response for Alt 2B at the model timestep when the shoreline hits the vegetation line (10.3 years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

Figure 53. Shoreline response for Alt 3A at the model timestep when the shoreline hits the vegetation line (9.8 years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

Figure 54. Shoreline response for Alt3B, at the model timestep when the shoreline hits the vegetation line (13.5 years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

Figure 55. Shoreline response for Alt 4A_v1 at the model timestep when the shoreline hits the vegetation line (7.1years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

Figure 56. Shoreline response for Alt 4B_v1 at the model timestep when the shoreline hits the vegetation line (9.8 years). Vegetation line (green), FWOP shoreline (purple), FWP shoreline (red), and FWP initial shoreline (yellow).

As described earlier in this section, at the time of re-nourishment (determined by when the FWP shoreline hits the vegetation line), alternatives with structures hold a greater volume of sand within the project site. However, these alternatives also increase downdrift erosion when compared to the FWOP scenario. Further investigation of the downdrift impacts caused by each alternative and the associated volume of erosion is discussed in Section 4.1.2.3.

4.1.2.2 Time to GI-01A Template

The time for all beach fill alternatives to erode 100 feet landward of the GI-01A template was used as a metric to quantify the performance of the beach fill only alternatives. Note that the extents of the GI-01A template are the same as Alt 2A. The trigger line for this alternative was determine by offsetting the GI-01A (Alt 2A) template 100 feet to the landward side. The time to GI-01A was determined by the first instance that each alternative hit this trigger line. This metric was used to compare the ability of the alternatives to hold the beach fill along the project shoreline. Since the line for this criterion is seaward of the present-day shoreline, only alternatives that include a beach fill were analyzed using this metric (Alt. 2A, 2B, 3B, and 4B_v1 were analyzed using the time to GI-01A metric, all others alternatives which lacked a beach fill in the design were not analyzed using this metric). The results of this analysis are shown below in Table 17.

Table 17: Time to GI-01A Template (minus 100 feet of erosion) for all alternatives that include a beach fill in the proposed design.

Alternative	Time to GI-01A (minus 100 feet) [yrs]
Alt. 2A	0.9
Alt. 2B	1.3
Alt. 3B	7.8
Alt. 4B_v1	2.2

Note that this analysis metric was used to compare alternatives that included a beach fill in design. This analysis was conducted over the project footprint only. There is an area of high erosion observed in front of the existing revetment, which are only within the project footprint of Alt 2A and Alt 2B. This partially explains the shorter lifespan of Alt 2A and Alt 2B. Based on these results, a four-year re-nourishment cycle could not have been achieved with the design as constructed at present. Alt 3B shows the best performance, showing decreased erosion of the beach fill due to the breakwater structures and project footprint outside the erosion hotspot. The center of the headlands formed by the T-Head groins in Alt 4B_v1 recede relatively quickly before stabilizing, explaining the short lifespan of Alt 4B_v1. Once stabilized, however, the shorelines for Alt 4B_v1 remain stable.

4.1.2.3 Downdrift Impacts

The downdrift erosion when compared to the FWOP shoreline should be included during alternative analysis to quantify impacts to the shoreline east of each project site. Downdrift impacts were analyzed to determine the shoreline change relative to the FWOP scenario. If increased erosion occurs downdrift of the proposed project, the volume to mitigate this erosion and return the shoreline to the FWOP shoreline at that timestep was included in the cost estimate. The downdrift impacts for each alternative, shown as change in volume from the FWOP scenario, is shown for years 1-15 of the simulation. These results assume that no re-nourishment is conducted during this fifteen-year timeframe. Actual downdrift impacts will vary for each alternative based upon the chosen renourishment cycle. The results of the downdrift impact analysis are shown below in Table 18. Note that results shown past the time when the shoreline impacts the dune (see Table 16) should be considered unreliable as the contribution of the dune to the littoral transport is ignored in the model.

Table 18: Volume deficit downdrift of project site from the FWOP scenario in cubic yards x 10^3. Area taken between shoreline at each year and the FWOP shoreline. Negative numbers indicate erosion compared to FWOP (at each year), positive numbers indicated accretion relative to FWOP (at each year).

	Duration [years]														
Alt.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
Alt. 2A	16	39	54	63	67	29	4	13	27	27	31	42	43	56	55
Alt. 2B	80	104	128	142	175	187	196	202	205	205	17	48	50	51	52
Alt. 3A	-20	-33	-44	-59	-65	-74	-81	-106	-117	-138	-167	-175	-181	-204	-226
Alt. 3B	11	8	25	27	26	24	23	10	3	-12	-36	-43	-43	-65	-86
Alt. 4A_v1	-12	-28	-20	-27	-33	-37	-39	-48	-53	-59	-74	-76	-70	-77	-93
Alt. 4B_v1	11	-3	13	12	10	6	5	-2	-9	-16	-30	-33	-28	-36	-53

The results shown in Table 18 indicate two patterns. First, most alternatives with structures (3A,4A_v1,4B_v1) show increased downdrift erosion when compared to the FWOP scenario. Second, Alt 3B shows decreased erosion when compared to the FWOP scenario for the first nine years of the simulation. Alt 3B includes a beach fill, and breakwaters placed offshore at a sufficient distance that a salient is expected to form along the project shoreline. This allows gradual transport of the beach fill downdrift, reducing shoreline erosion compared to FWOP. Alt 4A_v1, which includes a combination of breakwaters and T-Head groins, shows increased erosion downdrift when compared to FWOP. Alt 4B_v1, which has the same configuration as Alt 4A_v1 with a beach fill added, shows similar performance to Alt 3B. Also, note that Alt 2A and 2B show increased downdrift volume when compared to the FWOP scenario (i.e. less erosion). This is due to the eastward migration of the sand placed within the beach fill template for these alternatives. Although these alternatives show increased sediment downdrift when compared to the FWOP, they do little to promote retention of sand within the project shoreline due to the lack of breakwaters or other hard structures.

Mitigation of downdrift effects could be achieved by downdrift beach nourishments or refinement of the design. Examples of design refinement that could be employed for Alt 3A, Alt 3B, Alt 4A_v1, and Alt 4B_v1 include increasing the porosity of the structure to allow more sediment through the structure, or shorten the structure to allow more bypassing around the tip of the groin, or by adding beach fill to the east to mitigate the downdrift erosion. Refinement of structure components should be conducted during further design of preferred alternatives.

4.1.2.4 Shoreline Response Summary

Alternatives were analyzed for shoreline response performance using the metrics described in Section 4.1.2.1 through Section 4.1.2.3. Alt 2A and Alt 2B involve placing beach fill along the project shoreline. Alt 2A and Alt 2B last 7.1 and 10.3 years respectively, before reaching the existing vegetation line and have minimal downdrift impacts since no hard structures are placed along the project shoreline. Alt 3A and 3B call for five breakwaters placed approximately 500 feet landward of the 2015 shoreline. Alt 3A lasts 9.8 years before hitting the existing vegetation line, while Alt 3B lasts significantly longer (13.5 years) due to the beach fill placed within the project template. Alt 4A_v1 and Alt 4B_v1 involve a combination of T-Head groins and breakwaters near the project site, with alt 4B_v1 adding a beach fill to the design. Alt 4A_v1 lasts 7.1 years before the GI-01C trigger, while Alt 4B_v1 lasts 9.8 years. Refinement of the design of all alternatives during final design could potentially reduce the time to the vegetation line and thereby reduce maintenance costs.

4.2 Cost

The total project cost for each alternative consists of capital (or construction) and maintenance costs. Estimates of probable construction cost at a conceptual level were developed for each of the alternatives. The costs were developed from information gathered from local contractors, previously bid projects in the area (including GI-01, GI-01A, GI-01B, and GI-01C) and existing market conditions. A 35% contingency was added to the cost to account for the conceptual level of design and any unknowns during the final design phase. This cost estimate was primarily developed to compare the relative order of magnitude of cost between various alternatives.

The mobilization was determined for beach nourishment and rock alternatives based on equipment types and expected effort. The surveying was similarly determined for each alternative type on a per day basis cost. The volumes of materials were estimated and unit cost were applied to each alternative based on expected methodologies. Unit costs were derived from similar work in the vicinity and adjusted for the scale of the work based on experience. A summary of all the alternatives total capital cost range from $460K to $12M. A summary of the capital and maintenance costs for each alternative is shown in Table 19. A 50-year lifespan was assumed for calculation of all maintenance costs. These maintenance costs include dune and beach maintenance over the project lifespan. Note that the numbers shown in Table 19 are rounded. Exact costs are shown in the detailed cost estimate in Appendix A. The maintenance cycles and volumes for each alternative over the project lifespan is shown in Table 20.

Table 19. Conceptual cost estimate with 35% contingency on capital and maintenance costs, rounded to nearest hundred in total lifetime cost. Total maintenance costs include dune and beach maintenance over the 50-year lifespan of the project.

Alternative	Capital Cost	Dune Maintenance Cost	Beach Maintenance Cost	Total Maintenance Cost	Total Lifetime Cost
Alt 1.A	$460,000	$86,488,800	$95,814,200	$182,303,000	$182,763,000
Alt 1.B	$1,650,000	$86,488,800	$98,308,100	$184,796,900	$186,446,900
Alt 2.A	$7,800,000	$5,273,000	$109,370,000	$114,643,000	$122,443,000
Alt 2.B	$11,710,000	$1,814,000	$105,603,200	$107,417,200	$119,127,200
Alt 3.A	$3,980,000	$26,077,000	$46,528,300	$72,605,300	$76,585,300
Alt 3.B	$7,930,000	$5,086,600	$44,356,100	$49,442,700	$57,372,700
Alt 4.A_v1	$1,860,000	$29,156,200	$28,776,700	$57,932,900	$59,792,900
Alt 4.B_v1	$5,960,000	$2,262,700	$50,559,400	$52,822,100	$58,782,400

Table 20. Volumes placed for each maintenance cycle broken into: Beach maintenance volume to restore initial project template, beach maintenance volume to restore deficit downdrift of the project when compared to FWOP, and total beach maintenance volume. Note that all volumes shown (Beach & Dune) are at the maintenance interval for each project element.

Alternative	Dune Maintenance Interval [years]	Dune Maintenance Volume [cy]	Beach Maintenance Interval [years]	Beach Maintenance: Project Template [cy]	Beach Maintenance: Downdrift [cy]	Beach Maintenance Volume: Total [cy]
Alt 1.A	0.8	13,530	1.5	89,000	0	89,000
Alt 1.B	0.8	13,530	1.5	89,000	0	89,000
Alt 2.A	24.7	23,430	7.1	427,000	0	427,000
Alt 2.B	27.4	17,655	10.3	764,000	0	764,000
Alt 3.A	2.7	14,025	9.8	86,000	134,000	220,000
Alt 3.B	24.6	22,605	13.5	310,000	54,000	364,000
Alt 4.A_v1	2.4	14,190	7.1	54,000	40,000	94,000
Alt 4.B_v1	25.2	22,770	9.8	231,000	15,000	246,000

Maintenance costs such as re-nourishment are expected to be triggered upon a shoreline retreat of the Grand Isle shoreline to the vegetation line. The beach re-nourishment maintenance costs assume that the total beach maintenance volume consists of the quantities shown in Table 20, which shows volume to restore the shoreline to the initial shoreline in the project template and the downdrift deficit in sand quantity between the alternative and the FWOP scenario. Note that the beach maintenance volumes are the volumes necessary to restore the shoreline to the initial project shoreline as well as mitigate downdrift erosion compared to the FWOP. This volume will need to be placed at the maintenance interval shown in Table 20, which is triggered by the shoreline hitting the vegetation line as described in Section 4.1.2.1. Note that many of the beach maintenance volumes are larger than the initial volume required to place the beach template. This is because the vegetation line is landward of the initial shoreline, so there is additional fill volume needed to restore the project to the initial shoreline on top of replacing the capital volume of sand. Again, note that all cost estimates include a 35% contingency. Refinement of the cost, including maintenance interval and re-nourishment volumes, should be conducted for the selected alternative during final design.

In addition to the sand maintenance volumes shown in Table 20, maintenance costs were considered for repair of any rubble mound structures. It was assumed that every 15 years, half of the initial quantity of stone for each alternative will be replaced. This maintenance cycle was assumed to continue for the full 50-year maintenance period. The total stone weight necessary throughout the project is shown below in Table 21.

Table 21. Capital stone weight, maintenance stone weight, and total stone weight over the course of the 50-year project life. Estimates assume 50% of initial stone amount is replaced every 15 years.

Alternative	Capital Stone Weight [tons]	Maintenance Stone Weight [tons]	Total Stone Maintenance [tons]
Alt 3.A	21,700	32,550	54,250
Alt 3.B	21,700	32,550	54,250
Alt 4.A_v1	7,200	10,800	18,000
Alt 4.B_v1	7,200	10,800	18,000

A detailed breakdown of the costs for each proposed alternative is shown are Appendix A.

4.3 Recreational Value

The recreational value of the beach was assessed by determining the approximate dry beach area along the project site for each alternative. The first 10 years of the model simulation were analyzed and the area between the shoreline and the vegetation line was quantified. This analysis was conducted for the same length of beach for each alternative, extending from the Caminada Pass Jetty to approximately 2500 feet eastward. This length of beach was selected because it covers all alternative footprints. The acreage of each alternative in front of the vegetation line from years 1-10 is shown below in Table 22.

Table 22. Recreational beach area [acres] from

Alt.	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
FWOP/1A/1B	8.2	7.0	6.4	6.5	6.0	5.8	5.5	5.5	4.9	4.5
Alt. 2A	16.8	13.7	12.2	10.8	9.3	8.6	7.7	7.4	6.5	5.9
Alt. 2B	23.9	19.5	17.5	15.6	13.5	12.4	11.1	10.2	8.6	7.8
Alt. 3A	8.2	7.4	7.0	7.0	6.7	6.5	6.2	6.2	5.5	5.1
Alt. 3B	13.4	12.0	11.2	10.5	9.8	9.4	9.0	8.8	7.9	7.4
Alt. 4A_v1	8.2	7.4	7.0	7.2	6.9	6.7	6.5	6.4	5.9	5.6
Alt. 4B_v1	13.4	11.7	10.9	10.2	9.6	9.4	8.9	8.7	7.9	7.5

Alternatives with beach fills (2A, 2B, 3B, 4B_v1) show the greatest recreational value for the first five years of the simulation. The usable acreage compared to the year 1 acreage decreased at a faster rate for alternatives with only beach fills than for those with structures included in the design. Note that the recreational value shown in Table 22 does not include any maintenance performed during this 10-year period.

4.4 Recommendations

It is Mott MacDonald's opinion that the best performing alternative is Alt 3B which is breakwaters and GI-01A beach fill that has a moderate capital cost ($7.9M), long maintenance interval (13.5 years), and moderate total life-cycle costs at $57M. Breakwaters are proven to be effective on Grand Isle, while headland breakwaters have not been employed in Louisiana (they have, however, been shown effective on the Gulf and Atlantic coasts of Florida). The next best alternative is Alt 4B_v1 (headland breakwaters with dune fill and beach fill) which has a low capital cost ($6.4M), moderate maintenance interval (9.8 years) and a relatively low total life-cycle costs at $59M. Both alternatives provide reasonable access to recreational beach through their lifetime.

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A. Detailed Cost Breakdown

MOTT MACDONALD

Grand Isle Cost Estimate

Project:	Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis	PWM	5/03/2017	
Description:	Checked by:	Date:	A
Conceptual Cost Estimate	AA	5/03/2017	

Calculation No:	Rev. No.	Reviewed by:	Date:
1	2	AA	5/03/2017

Alternative 1A

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 30,000	$ 30,000
Surveying	1	LS	$ 11,000	$ 11,000
Environmental Protection	1	LS	$ 9,000	$ 9,000
Beach and Dune Nourishment	5,300	CY	$ 55	$ 291,500
			Subtotal	$ 341,500
			Contingency [35%]	$ 119,525
			Capital Cost	$ 461,025

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
1.5	89,000	$ 18	$ 1,626,090	$ 11,165	$ 9,135	$ 30,451	$ 1,676,842	$ 586,895	$ 2,263,736
3.0	89,000	$ 19	$ 1,650,542	$ 11,333	$ 9,273	$ 30,909	$ 1,702,057	$ 595,720	$ 2,297,777
4.5	89,000	$ 19	$ 1,675,362	$ 11,504	$ 9,412	$ 31,374	$ 1,727,652	$ 604,678	$ 2,332,330
6.0	89,000	$ 19	$ 1,700,555	$ 11,677	$ 9,554	$ 31,846	$ 1,753,631	$ 613,771	$ 2,367,402
7.5	89,000	$ 19	$ 1,726,127	$ 11,852	$ 9,697	$ 32,324	$ 1,780,001	$ 623,000	$ 2,403,002
9.0	89,000	$ 20	$ 1,752,084	$ 12,031	$ 9,843	$ 32,811	$ 1,806,768	$ 632,369	$ 2,439,137
10.5	89,000	$ 20	$ 1,778,431	$ 12,211	$ 9,991	$ 33,304	$ 1,833,937	$ 641,878	$ 2,475,815
12.0	89,000	$ 20	$ 1,805,174	$ 12,395	$ 10,141	$ 33,805	$ 1,861,515	$ 651,530	$ 2,513,045
13.5	89,000	$ 21	$ 1,832,319	$ 12,581	$ 10,294	$ 34,313	$ 1,889,507	$ 661,328	$ 2,550,835
15.0	89,000	$ 21	$ 1,859,872	$ 12,771	$ 10,449	$ 34,829	$ 1,917,921	$ 671,272	$ 2,589,193
16.5	89,000	$ 21	$ 1,887,840	$ 12,963	$ 10,606	$ 35,353	$ 1,946,761	$ 681,366	$ 2,628,128
18.0	89,000	$ 22	$ 1,916,228	$ 13,158	$ 10,765	$ 35,884	$ 1,976,036	$ 691,612	$ 2,667,648
19.5	89,000	$ 22	$ 1,945,043	$ 13,355	$ 10,927	$ 36,424	$ 2,005,750	$ 702,013	$ 2,707,763
21.0	89,000	$ 22	$ 1,974,292	$ 13,556	$ 11,092	$ 36,972	$ 2,035,911	$ 712,569	$ 2,748,481
22.5	89,000	$ 23	$ 2,003,980	$ 13,760	$ 11,258	$ 37,528	$ 2,066,526	$ 723,284	$ 2,789,811
24.0	89,000	$ 23	$ 2,034,115	$ 13,967	$ 11,428	$ 38,092	$ 2,097,602	$ 734,161	$ 2,831,762
25.5	89,000	$ 23	$ 2,064,703	$ 14,177	$ 11,599	$ 38,665	$ 2,129,144	$ 745,200	$ 2,874,345
27.0	89,000	$ 24	$ 2,095,751	$ 14,390	$ 11,774	$ 39,246	$ 2,161,161	$ 756,406	$ 2,917,567
28.5	89,000	$ 24	$ 2,127,265	$ 14,607	$ 11,951	$ 39,836	$ 2,193,659	$ 767,781	$ 2,961,440
30.0	89,000	$ 24	$ 2,159,254	$ 14,826	$ 12,131	$ 40,435	$ 2,226,646	$ 779,326	$ 3,005,973
31.5	89,000	$ 25	$ 2,191,724	$ 15,049	$ 12,313	$ 41,044	$ 2,260,129	$ 791,045	$ 3,051,175
33.0	89,000	$ 25	$ 2,224,682	$ 15,276	$ 12,498	$ 41,661	$ 2,294,116	$ 802,941	$ 3,097,057
34.5	89,000	$ 25	$ 2,258,135	$ 15,505	$ 12,686	$ 42,287	$ 2,328,614	$ 815,015	$ 3,143,628
36.0	89,000	$ 26	$ 2,292,092	$ 15,738	$ 12,877	$ 42,923	$ 2,363,630	$ 827,271	$ 3,190,901
37.5	89,000	$ 26	$ 2,326,559	$ 15,975	$ 13,071	$ 43,569	$ 2,399,173	$ 839,711	$ 3,238,884
39.0	89,000	$ 27	$ 2,361,544	$ 16,215	$ 13,267	$ 44,224	$ 2,435,250	$ 852,338	$ 3,287,588
40.5	89,000	$ 27	$ 2,397,056	$ 16,459	$ 13,467	$ 44,889	$ 2,471,870	$ 865,155	$ 3,337,025
42.0	89,000	$ 27	$ 2,433,101	$ 16,707	$ 13,669	$ 45,564	$ 2,509,041	$ 878,164	$ 3,387,205
43.5	89,000	$ 28	$ 2,469,689	$ 16,958	$ 13,875	$ 46,249	$ 2,546,770	$ 891,370	$ 3,438,140
45.0	89,000	$ 28	$ 2,506,827	$ 17,213	$ 14,083	$ 46,944	$ 2,585,067	$ 904,774	$ 3,489,841
46.5	89,000	$ 29	$ 2,544,523	$ 17,472	$ 14,295	$ 47,650	$ 2,623,940	$ 918,379	$ 3,542,319
48.0	89,000	$ 29	$ 2,582,786	$ 17,734	$ 14,510	$ 48,367	$ 2,663,397	$ 932,189	$ 3,595,587
49.5	89,000	$ 29	$ 2,621,625	$ 18,001	$ 14,728	$ 49,094	$ 2,703,448	$ 946,207	$ 3,649,655
Total	2,937,000	--	$ 68,825,369	$ 472,584	$ 386,659	$ 1,288,865	$ 70,973,477	$ 24,840,717	$ 95,814,194

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
0.8	13,530	55	$ 750,097	$ 13,104	$ 9,072	$ 30,240	$ 802,513	$ 280,879	$ 1,083,392
1.6	13,530	56	$ 756,092	$ 13,209	$ 9,144	$ 30,481	$ 808,927	$ 283,124	$ 1,092,051
2.4	13,530	56	$ 762,135	$ 13,314	$ 9,218	$ 30,725	$ 815,392	$ 285,387	$ 1,100,779
3.2	13,530	57	$ 768,226	$ 13,421	$ 9,291	$ 30,971	$ 821,908	$ 287,668	$ 1,109,576
4	13,530	57	$ 774,365	$ 13,528	$ 9,365	$ 31,218	$ 828,477	$ 289,967	$ 1,118,444
4.8	13,530	58	$ 780,554	$ 13,636	$ 9,440	$ 31,468	$ 835,098	$ 292,284	$ 1,127,382
5.6	13,530	58	$ 786,792	$ 13,745	$ 9,516	$ 31,719	$ 841,772	$ 294,620	$ 1,136,393
6.4	13,530	59	$ 793,081	$ 13,855	$ 9,592	$ 31,973	$ 848,500	$ 296,975	$ 1,145,475
7.2	13,530	59	$ 799,419	$ 13,966	$ 9,668	$ 32,228	$ 855,281	$ 299,348	$ 1,154,629
8	13,530	60	$ 805,808	$ 14,077	$ 9,746	$ 32,486	$ 862,116	$ 301,741	$ 1,163,857
8.8	13,530	60	$ 812,248	$ 14,190	$ 9,824	$ 32,745	$ 869,006	$ 304,152	$ 1,173,159
9.6	13,530	61	$ 818,739	$ 14,303	$ 9,902	$ 33,007	$ 875,952	$ 306,583	$ 1,182,535
10.4	13,530	61	$ 825,283	$ 14,417	$ 9,981	$ 33,271	$ 882,952	$ 309,033	$ 1,191,985
11.2	13,530	61	$ 831,878	$ 14,533	$ 10,061	$ 33,537	$ 890,009	$ 311,503	$ 1,201,512
12	13,530	62	$ 838,527	$ 14,649	$ 10,141	$ 33,805	$ 897,122	$ 313,993	$ 1,211,114
12.8	13,530	62	$ 845,228	$ 14,766	$ 10,222	$ 34,075	$ 904,292	$ 316,502	$ 1,220,794
13.6	13,530	63	$ 851,983	$ 14,884	$ 10,304	$ 34,347	$ 911,519	$ 319,032	$ 1,230,550
14.4	13,530	63	$ 858,793	$ 15,003	$ 10,387	$ 34,622	$ 918,804	$ 321,581	$ 1,240,385
15.2	13,530	64	$ 865,656	$ 15,123	$ 10,470	$ 34,898	$ 926,147	$ 324,151	$ 1,250,298
16	13,530	64	$ 872,574	$ 15,244	$ 10,553	$ 35,177	$ 933,548	$ 326,742	$ 1,260,290
16.8	13,530	65	$ 879,548	$ 15,365	$ 10,638	$ 35,458	$ 941,009	$ 329,353	$ 1,270,363
17.6	13,530	66	$ 886,577	$ 15,488	$ 10,723	$ 35,742	$ 948,530	$ 331,986	$ 1,280,516
18.4	13,530	66	$ 893,663	$ 15,612	$ 10,808	$ 36,028	$ 956,111	$ 334,639	$ 1,290,749
19.2	13,530	67	$ 900,805	$ 15,737	$ 10,895	$ 36,315	$ 963,752	$ 337,313	$ 1,301,065
20	13,530	67	$ 908,004	$ 15,862	$ 10,982	$ 36,606	$ 971,454	$ 340,009	$ 1,311,463
20.8	13,530	68	$ 915,261	$ 15,989	$ 11,069	$ 36,898	$ 979,218	$ 342,726	$ 1,321,945
21.6	13,530	68	$ 922,576	$ 16,117	$ 11,158	$ 37,193	$ 987,044	$ 345,465	$ 1,332,510
22.4	13,530	69	$ 929,949	$ 16,246	$ 11,247	$ 37,490	$ 994,933	$ 348,226	$ 1,343,159
23.2	13,530	69	$ 937,381	$ 16,376	$ 11,337	$ 37,790	$ 1,002,884	$ 351,009	$ 1,353,894
24	13,530	70	$ 944,873	$ 16,507	$ 11,428	$ 38,092	$ 1,010,899	$ 353,815	$ 1,364,714
24.8	13,530	70	$ 952,424	$ 16,638	$ 11,519	$ 38,396	$ 1,018,978	$ 356,642	$ 1,375,621
25.6	13,530	71	$ 960,036	$ 16,771	$ 11,611	$ 38,703	$ 1,027,122	$ 359,493	$ 1,386,615
26.4	13,530	72	$ 967,709	$ 16,905	$ 11,704	$ 39,013	$ 1,035,331	$ 362,366	$ 1,397,697
27.2	13,530	72	$ 975,443	$ 17,041	$ 11,797	$ 39,324	$ 1,043,605	$ 365,262	$ 1,408,867
28	13,530	73	$ 983,239	$ 17,177	$ 11,892	$ 39,639	$ 1,051,946	$ 368,181	$ 1,420,127
28.8	13,530	73	$ 991,097	$ 17,314	$ 11,987	$ 39,956	$ 1,060,353	$ 371,124	$ 1,431,477
29.6	13,530	74	$ 999,018	$ 17,452	$ 12,082	$ 40,275	$ 1,068,827	$ 374,090	$ 1,442,917
30.4	13,530	74	$ 1,007,002	$ 17,592	$ 12,179	$ 40,597	$ 1,077,369	$ 377,079	$ 1,454,449
31.2	13,530	75	$ 1,015,050	$ 17,733	$ 12,276	$ 40,921	$ 1,085,980	$ 380,093	$ 1,466,073
32	13,530	76	$ 1,023,162	$ 17,874	$ 12,374	$ 41,248	$ 1,094,659	$ 383,131	$ 1,477,790
32.8	13,530	76	$ 1,031,339	$ 18,017	$ 12,473	$ 41,578	$ 1,103,408	$ 386,193	$ 1,489,600
33.6	13,530	77	$ 1,039,582	$ 18,161	$ 12,573	$ 41,910	$ 1,112,226	$ 389,279	$ 1,501,505
34.4	13,530	77	$ 1,047,890	$ 18,306	$ 12,674	$ 42,245	$ 1,121,115	$ 392,390	$ 1,513,505
35.2	13,530	78	$ 1,056,265	$ 18,453	$ 12,775	$ 42,583	$ 1,130,075	$ 395,526	$ 1,525,601
36	13,530	79	$ 1,064,707	$ 18,600	$ 12,877	$ 42,923	$ 1,139,107	$ 398,687	$ 1,537,794
36.8	13,530	79	$ 1,073,216	$ 18,749	$ 12,980	$ 43,266	$ 1,148,210	$ 401,874	$ 1,550,084
37.6	13,530	80	$ 1,081,793	$ 18,898	$ 13,084	$ 43,612	$ 1,157,387	$ 405,085	$ 1,562,472
38.4	13,530	81	$ 1,090,439	$ 19,050	$ 13,188	$ 43,960	$ 1,166,637	$ 408,323	$ 1,574,960
39.2	13,530	81	$ 1,099,153	$ 19,202	$ 13,294	$ 44,312	$ 1,175,961	$ 411,586	$ 1,587,547
40	13,530	82	$ 1,107,938	$ 19,355	$ 13,400	$ 44,666	$ 1,185,359	$ 414,876	$ 1,600,234
40.8	13,530	83	$ 1,116,793	$ 19,510	$ 13,507	$ 45,023	$ 1,194,832	$ 418,191	$ 1,613,024
41.6	13,530	83	$ 1,125,718	$ 19,666	$ 13,615	$ 45,383	$ 1,204,381	$ 421,533	$ 1,625,915
42.4	13,530	84	$ 1,134,715	$ 19,823	$ 13,724	$ 45,745	$ 1,214,007	$ 424,902	$ 1,638,909
43.2	13,530	85	$ 1,143,784	$ 19,981	$ 13,833	$ 46,111	$ 1,223,709	$ 428,298	$ 1,652,008
44	13,530	85	$ 1,152,925	$ 20,141	$ 13,944	$ 46,480	$ 1,233,489	$ 431,721	$ 1,665,210
44.8	13,530	86	$ 1,162,139	$ 20,302	$ 14,055	$ 46,851	$ 1,243,347	$ 435,172	$ 1,678,519
45.6	13,530	87	$ 1,171,427	$ 20,464	$ 14,168	$ 47,225	$ 1,253,284	$ 438,649	$ 1,691,934
46.4	13,530	87	$ 1,180,789	$ 20,628	$ 14,281	$ 47,603	$ 1,263,300	$ 442,155	$ 1,705,456
47.2	13,530	88	$ 1,190,226	$ 20,793	$ 14,395	$ 47,983	$ 1,273,397	$ 445,689	$ 1,719,086
48	13,530	89	$ 1,199,738	$ 20,959	$ 14,510	$ 48,367	$ 1,283,574	$ 449,251	$ 1,732,825
48.8	13,530	89	$ 1,209,326	$ 21,126	$ 14,626	$ 48,753	$ 1,293,832	$ 452,841	$ 1,746,673
49.6	13,530	90	$ 1,218,991	$ 12,897	$ 14,743	$ 49,143	$ 1,295,774	$ 453,521	$ 1,749,295
Total	838,860	--	$ 59,889,189	$ 1,037,842	$ 724,320	$ 2,414,400	$ 64,065,750	$ 22,423,013	$ 86,488,763

Total Costs

Description	Total Cost	Total Cost
Capital	$ 461,025	$ -
Beach Maintenance	$ 95,814,194	$ -
Dune Maintenance	$ 86,488,763	$ -
Total Lifetime	$ 182,763,982	$ -

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars)

4. Beach Maintenance mobilization, survey, and environmental protection adjusted for inflation for each maintenance cycle

MOTT MACDONALD

Grand Isle Cost Estimate

Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	4/18/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	4/18/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	3	AA	4/18/2017	

Alternative 1B

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 50,000	$ 50,000
Surveying	1	LS	$ 13,000	$ 13,000
Environmental Protection	1	LS	$ 30,000	$ 30,000
Beach and Dune Nourishment	5,850	CY	$ 55	$ 321,750
Revetment Core	8,100	TON	$ 100	$ 810,000
			Subtotal	$ 1,224,750
			Contingency [35%]	$ 428,663
			Capital Cost	$ 1,653,413

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Revetment Core	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
1.5	89,000	$ 18.27	$ 1,626,090	$ -	$ 13,195	$ 30,451	$ 50,752	$ 1,720,488	$ 602,171	$ 2,322,659
3	89,000	$ 18.55	$ 1,650,542	$ -	$ 13,394	$ 30,909	$ 51,515	$ 1,746,360	$ 611,226	$ 2,357,586
4.5	89,000	$ 18.82	$ 1,675,362	$ -	$ 13,595	$ 31,374	$ 52,290	$ 1,772,621	$ 620,417	$ 2,393,038
6	89,000	$ 19.11	$ 1,700,555	$ -	$ 13,800	$ 31,846	$ 53,076	$ 1,799,277	$ 629,747	$ 2,429,023
7.5	89,000	$ 19.39	$ 1,726,127	$ -	$ 14,007	$ 32,324	$ 53,874	$ 1,826,333	$ 639,217	$ 2,465,550
9	89,000	$ 19.69	$ 1,752,084	$ -	$ 14,218	$ 32,811	$ 54,684	$ 1,853,797	$ 648,829	$ 2,502,625
10.5	89,000	$ 19.98	$ 1,778,431	$ -	$ 14,432	$ 33,304	$ 55,507	$ 1,881,673	$ 658,586	$ 2,540,258
12	89,000	$ 20.28	$ 1,805,174	$ -	$ 14,649	$ 33,805	$ 56,341	$ 1,909,968	$ 668,489	$ 2,578,457
13.5	89,000	$ 20.59	$ 1,832,319	$ -	$ 14,869	$ 34,313	$ 57,188	$ 1,938,689	$ 678,541	$ 2,617,231
15	89,000	$ 20.90	$ 1,859,872	$ -	$ 15,093	$ 34,829	$ 58,048	$ 1,967,842	$ 688,745	$ 2,656,587
16.5	89,000	$ 21.21	$ 1,887,840	$ -	$ 15,320	$ 35,353	$ 58,921	$ 1,997,434	$ 699,102	$ 2,696,535
18	89,000	$ 21.53	$ 1,916,228	$ -	$ 15,550	$ 35,884	$ 59,807	$ 2,027,470	$ 709,614	$ 2,737,084
19.5	89,000	$ 21.85	$ 1,945,043	$ -	$ 15,784	$ 36,424	$ 60,707	$ 2,057,958	$ 720,285	$ 2,778,243
21	89,000	$ 22.18	$ 1,974,292	$ -	$ 16,021	$ 36,972	$ 61,620	$ 2,088,904	$ 731,117	$ 2,820,021
22.5	89,000	$ 22.52	$ 2,003,980	$ -	$ 16,262	$ 37,528	$ 62,546	$ 2,120,316	$ 742,111	$ 2,862,427
24	89,000	$ 22.86	$ 2,034,115	$ -	$ 16,507	$ 38,092	$ 63,487	$ 2,152,200	$ 753,270	$ 2,905,470
25.5	89,000	$ 23.20	$ 2,064,703	$ -	$ 16,755	$ 38,665	$ 64,441	$ 2,184,564	$ 764,597	$ 2,949,161
27	89,000	$ 23.55	$ 2,095,751	$ -	$ 17,007	$ 39,246	$ 65,410	$ 2,217,414	$ 776,095	$ 2,993,509
28.5	89,000	$ 23.90	$ 2,127,265	$ -	$ 17,262	$ 39,836	$ 66,394	$ 2,250,758	$ 787,765	$ 3,038,524
30	89,000	$ 24.26	$ 2,159,254	$ -	$ 17,522	$ 40,435	$ 67,392	$ 2,284,604	$ 799,611	$ 3,084,215
31.5	89,000	$ 24.63	$ 2,191,724	$ -	$ 17,786	$ 41,044	$ 68,406	$ 2,318,959	$ 811,635	$ 3,130,594
33	89,000	$ 25.00	$ 2,224,682	$ -	$ 18,053	$ 41,661	$ 69,435	$ 2,353,830	$ 823,840	$ 3,177,670
34.5	89,000	$ 25.37	$ 2,258,135	$ -	$ 18,324	$ 42,287	$ 70,479	$ 2,389,225	$ 836,229	$ 3,225,454
36	89,000	$ 25.75	$ 2,292,092	$ -	$ 18,600	$ 42,923	$ 71,538	$ 2,425,153	$ 848,804	$ 3,273,957
37.5	89,000	$ 26.14	$ 2,326,559	$ -	$ 18,880	$ 43,569	$ 72,614	$ 2,461,621	$ 861,567	$ 3,323,189
39	89,000	$ 26.53	$ 2,361,544	$ -	$ 19,164	$ 44,224	$ 73,706	$ 2,498,638	$ 874,523	$ 3,373,161
40.5	89,000	$ 26.93	$ 2,397,056	$ -	$ 19,452	$ 44,889	$ 74,814	$ 2,536,211	$ 887,674	$ 3,423,885
42	89,000	$ 27.34	$ 2,433,101	$ -	$ 19,744	$ 45,564	$ 75,939	$ 2,574,349	$ 901,022	$ 3,475,371
43.5	89,000	$ 27.75	$ 2,469,689	$ -	$ 20,041	$ 46,249	$ 77,081	$ 2,613,060	$ 914,571	$ 3,527,632
45	89,000	$ 28.17	$ 2,506,827	$ -	$ 20,343	$ 46,944	$ 78,241	$ 2,652,354	$ 928,324	$ 3,580,678
46.5	89,000	$ 28.59	$ 2,544,523	$ -	$ 20,648	$ 47,650	$ 79,417	$ 2,692,239	$ 942,284	$ 3,634,522
48	89,000	$ 29.02	$ 2,582,786	$ -	$ 20,959	$ 48,367	$ 80,611	$ 2,732,723	$ 956,453	$ 3,689,176
49.5	89,000	$ 29.46	$ 2,621,625	$ -	$ 21,274	$ 49,094	$ 81,823	$ 2,773,816	$ 970,836	$ 3,744,652
Total	2,937,000	--	$ 68,825,369	$ -	$ 558,508	$ 1,288,865	$ 2,148,108	$ 72,820,850	$ 25,487,297	$ 98,308,147

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
0.8	13,530	55	$ 750,097.3	$ 13,104	$ 9,072	$ 30,240	$ 802,513	$ 280,879	$ 1,083,392
1.6	13,530	56	$ 756,092.1	$ 13,209	$ 9,144	$ 30,481	$ 808,927	$ 283,124	$ 1,092,051
2.4	13,530	56	$ 762,134.8	$ 13,314	$ 9,218	$ 30,725	$ 815,392	$ 285,387	$ 1,100,779
3.2	13,530	57	$ 768,225.8	$ 13,421	$ 9,291	$ 30,971	$ 821,908	$ 287,668	$ 1,109,576
4	13,530	57	$ 774,365.5	$ 13,528	$ 9,365	$ 31,218	$ 828,477	$ 289,967	$ 1,118,444
4.8	13,530	58	$ 780,554.2	$ 13,636	$ 9,440	$ 31,468	$ 835,098	$ 292,284	$ 1,127,382
5.6	13,530	58	$ 786,792.4	$ 13,745	$ 9,516	$ 31,719	$ 841,772	$ 294,620	$ 1,136,393
6.4	13,530	59	$ 793,080.5	$ 13,855	$ 9,592	$ 31,973	$ 848,500	$ 296,975	$ 1,145,475
7.2	13,530	59	$ 799,418.8	$ 13,966	$ 9,668	$ 32,228	$ 855,281	$ 299,348	$ 1,154,629
8	13,530	60	$ 805,807.8	$ 14,077	$ 9,746	$ 32,486	$ 862,116	$ 301,741	$ 1,163,857
8.8	13,530	60	$ 812,247.9	$ 14,190	$ 9,824	$ 32,745	$ 869,006	$ 304,152	$ 1,173,159
9.6	13,530	61	$ 818,739.4	$ 14,303	$ 9,902	$ 33,007	$ 875,952	$ 306,583	$ 1,182,535
10.4	13,530	61	$ 825,282.8	$ 14,417	$ 9,981	$ 33,271	$ 882,952	$ 309,033	$ 1,191,985
11.2	13,530	61	$ 831,878.4	$ 14,533	$ 10,061	$ 33,537	$ 890,009	$ 311,503	$ 1,201,512
12	13,530	62	$ 838,526.8	$ 14,649	$ 10,141	$ 33,805	$ 897,122	$ 313,993	$ 1,211,114
12.8	13,530	62	$ 845,228.4	$ 14,766	$ 10,222	$ 34,075	$ 904,292	$ 316,502	$ 1,220,794
13.6	13,530	63	$ 851,983.5	$ 14,884	$ 10,304	$ 34,347	$ 911,519	$ 319,032	$ 1,230,550
14.4	13,530	63	$ 858,792.6	$ 15,003	$ 10,387	$ 34,622	$ 918,804	$ 321,581	$ 1,240,385
15.2	13,530	64	$ 865,656.0	$ 15,123	$ 10,470	$ 34,898	$ 926,147	$ 324,151	$ 1,250,298
16	13,530	64	$ 872,574.4	$ 15,244	$ 10,553	$ 35,177	$ 933,548	$ 326,742	$ 1,260,290
16.8	13,530	65	$ 879,548.0	$ 15,365	$ 10,638	$ 35,458	$ 941,009	$ 329,353	$ 1,270,363
17.6	13,530	66	$ 886,577.4	$ 15,488	$ 10,723	$ 35,742	$ 948,530	$ 331,986	$ 1,280,516
18.4	13,530	66	$ 893,663.0	$ 15,612	$ 10,808	$ 36,028	$ 956,111	$ 334,639	$ 1,290,749
19.2	13,530	67	$ 900,805.2	$ 15,737	$ 10,895	$ 36,315	$ 963,752	$ 337,313	$ 1,301,065
20	13,530	67	$ 908,004.4	$ 15,862	$ 10,982	$ 36,606	$ 971,454	$ 340,009	$ 1,311,463
20.8	13,530	68	$ 915,261.2	$ 15,989	$ 11,069	$ 36,898	$ 979,218	$ 342,726	$ 1,321,945
21.6	13,530	68	$ 922,576.0	$ 16,117	$ 11,158	$ 37,193	$ 987,044	$ 345,465	$ 1,332,510
22.4	13,530	69	$ 929,949.3	$ 16,246	$ 11,247	$ 37,490	$ 994,933	$ 348,226	$ 1,343,159
23.2	13,530	69	$ 937,381.5	$ 16,376	$ 11,337	$ 37,790	$ 1,002,884	$ 351,009	$ 1,353,894
24	13,530	70	$ 944,873.0	$ 16,507	$ 11,428	$ 38,092	$ 1,010,899	$ 353,815	$ 1,364,714
24.8	13,530	70	$ 952,424.5	$ 16,638	$ 11,519	$ 38,396	$ 1,018,978	$ 356,642	$ 1,375,621
25.6	13,530	71	$ 960,036.3	$ 16,771	$ 11,611	$ 38,703	$ 1,027,122	$ 359,493	$ 1,386,615
26.4	13,530	72	$ 967,708.9	$ 16,905	$ 11,704	$ 39,013	$ 1,035,331	$ 362,366	$ 1,397,697
27.2	13,530	72	$ 975,442.9	$ 17,041	$ 11,797	$ 39,324	$ 1,043,605	$ 365,262	$ 1,408,867
28	13,530	73	$ 983,238.7	$ 17,177	$ 11,892	$ 39,639	$ 1,051,946	$ 368,181	$ 1,420,127
28.8	13,530	73	$ 991,096.7	$ 17,314	$ 11,987	$ 39,956	$ 1,060,353	$ 371,124	$ 1,431,477
29.6	13,530	74	$ 999,017.6	$ 17,452	$ 12,082	$ 40,275	$ 1,068,827	$ 374,090	$ 1,442,917
30.4	13,530	74	$ 1,007,001.8	$ 17,592	$ 12,179	$ 40,597	$ 1,077,369	$ 377,079	$ 1,454,449
31.2	13,530	75	$ 1,015,049.8	$ 17,733	$ 12,276	$ 40,921	$ 1,085,980	$ 380,093	$ 1,466,073
32	13,530	76	$ 1,023,162.1	$ 17,874	$ 12,374	$ 41,248	$ 1,094,659	$ 383,131	$ 1,477,790
32.8	13,530	76	$ 1,031,339.3	$ 18,017	$ 12,473	$ 41,578	$ 1,103,408	$ 386,193	$ 1,489,600
33.6	13,530	77	$ 1,039,581.7	$ 18,161	$ 12,573	$ 41,910	$ 1,112,226	$ 389,279	$ 1,501,505
34.4	13,530	77	$ 1,047,890.1	$ 18,306	$ 12,674	$ 42,245	$ 1,121,115	$ 392,390	$ 1,513,505
35.2	13,530	78	$ 1,056,264.9	$ 18,453	$ 12,775	$ 42,583	$ 1,130,075	$ 395,526	$ 1,525,601
36	13,530	79	$ 1,064,706.6	$ 18,600	$ 12,877	$ 42,923	$ 1,139,107	$ 398,687	$ 1,537,794
36.8	13,530	79	$ 1,073,215.8	$ 18,749	$ 12,980	$ 43,266	$ 1,148,210	$ 401,874	$ 1,550,084
37.6	13,530	80	$ 1,081,792.9	$ 18,898	$ 13,084	$ 43,612	$ 1,157,387	$ 405,085	$ 1,562,472
38.4	13,530	81	$ 1,090,438.7	$ 19,050	$ 13,188	$ 43,960	$ 1,166,637	$ 408,323	$ 1,574,960
39.2	13,530	81	$ 1,099,153.5	$ 19,202	$ 13,294	$ 44,312	$ 1,175,961	$ 411,586	$ 1,587,547
40	13,530	82	$ 1,107,937.9	$ 19,355	$ 13,400	$ 44,666	$ 1,185,359	$ 414,876	$ 1,600,234
40.8	13,530	83	$ 1,116,792.6	$ 19,510	$ 13,507	$ 45,023	$ 1,194,832	$ 418,191	$ 1,613,024
41.6	13,530	83	$ 1,125,718.1	$ 19,666	$ 13,615	$ 45,383	$ 1,204,381	$ 421,533	$ 1,625,915
42.4	13,530	84	$ 1,134,714.8	$ 19,823	$ 13,724	$ 45,745	$ 1,214,007	$ 424,902	$ 1,638,909
43.2	13,530	85	$ 1,143,783.5	$ 19,981	$ 13,833	$ 46,111	$ 1,223,709	$ 428,298	$ 1,652,008
44	13,530	85	$ 1,152,924.7	$ 20,141	$ 13,944	$ 46,480	$ 1,233,489	$ 431,721	$ 1,665,210
44.8	13,530	86	$ 1,162,138.9	$ 20,302	$ 14,055	$ 46,851	$ 1,243,347	$ 435,172	$ 1,678,519
45.6	13,530	87	$ 1,171,426.7	$ 20,464	$ 14,168	$ 47,225	$ 1,253,284	$ 438,649	$ 1,691,934
46.4	13,530	87	$ 1,180,788.8	$ 20,628	$ 14,281	$ 47,603	$ 1,263,300	$ 442,155	$ 1,705,456
47.2	13,530	88	$ 1,190,225.7	$ 20,793	$ 14,395	$ 47,983	$ 1,273,397	$ 445,689	$ 1,719,086
48	13,530	89	$ 1,199,738.0	$ 20,959	$ 14,510	$ 48,367	$ 1,283,574	$ 449,251	$ 1,732,825
48.8	13,530	89	$ 1,209,326.4	$ 21,126	$ 14,626	$ 48,753	$ 1,293,832	$ 452,841	$ 1,746,673
49.6	13,530	90	$ 1,218,991.4	$ 12,897	$ 14,743	$ 49,143	$ 1,295,774	$ 453,521	$ 1,749,295
Total Maintenance	838,860	--	$ 59,889,189	$ 1,037,842	$ 724,320	$ 2,414,400	$ 64,065,750	$ 22,423,013	$ 86,488,763

Total Costs

Description	Total Cost
Capital	$ 1,653,413
Beach Maintenance	$ 98,308,147
Dune Maintenance	$ 86,488,763
Total	$ 186,450,323

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed renourishment of dune will occur before damage to revetment core.

4. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars)

5. Beach Maintenance mobilization, survey, rock, and environmental protection adjusted for inflation for each maintenance cycle

Grand Isle Cost Estimate

Project:	Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis	PWM	4/18/2017	
Description:	Checked by:	Date:	A
Conceptual Cost Estimate	AA	4/18/2017	
Calculation No: 1 / Rev. No. 3	Reviewed by: AA	Date: 4/18/2017	

Alternative 2A

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 750,000	$ 750,000
Surveying	1	LS	$ 61,000	$ 61,000
Environmental Protection	1	LS	$ 141,000	$ 141,000
Beach and Dune Nourishment	268,200	CY	$ 18	$ 4,827,600
			Subtotal	$ 5,779,600
			Contingency [35%]	$ 2,022,860
			Capital Cost	$ 7,802,460

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
7.1	427,000	19.3	$ 8,248,636	$ 65,465	$ 151,322	$ 804,902	$ 9,270,325	$ 3,244,614	$ 12,514,939
14.2	427,000	20.7	$ 8,852,458	$ 70,258	$ 162,399	$ 863,823	$ 9,948,938	$ 3,482,128	$ 13,431,066
21.3	427,000	22.2	$ 9,500,482	$ 75,401	$ 174,287	$ 927,057	$ 10,677,227	$ 3,737,029	$ 14,414,256
28.4	427,000	23.9	$ 10,195,943	$ 80,920	$ 187,045	$ 994,920	$ 11,458,828	$ 4,010,590	$ 15,469,418
35.5	427,000	25.6	$ 10,942,313	$ 86,844	$ 200,737	$ 1,067,751	$ 12,297,645	$ 4,304,176	$ 16,601,821
42.6	427,000	27.5	$ 11,743,320	$ 93,201	$ 215,432	$ 1,145,913	$ 13,197,866	$ 4,619,253	$ 17,817,119
49.7	427,000	29.5	$ 12,602,963	$ 100,024	$ 231,202	$ 1,229,797	$ 14,163,986	$ 4,957,395	$ 19,121,381
Total	2,989,000	--	$ 72,086,116	$ 572,112	$ 1,322,423	$ 7,034,164	$ 81,014,815	$ 28,355,185	$ 109,370,000

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
24.7	23,430	70	$ 1,647,680.2	$ 16,622	$ 11,507	$ 38,358	$ 1,714,168	$ 599,959	$ 2,314,127
49.4	23,430	90	$ 2,106,739.5	$ 21,253	$ 14,714	$ 49,045	$ 2,191,751	$ 767,113	$ 2,958,864
Total	46,860	--	$ 3,754,420	$ 37,875	$ 26,221	$ 87,404	$ 3,905,919	$ 1,367,072	$ 5,272,991

Total Costs

Description	Total Cost
Capital	$ 7,802,460
Beach Maintenance	$ 109,370,000
Dune Maintenance	$ 5,272,991
Total Lifetime Cost	$ 122,445,451

Notes

1. Assumed 1% yearly inflation
2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.
3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.
4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.
5. Beach Maintenance mobilization, survey, and environmental protection adjusted for inflation for each maintenance cycle

MOTT MACDONALD	Grand Isle Cost Estimate			
Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	4/18/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	4/18/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	3	AA	4/18/2017	

Alternative 2B

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 1,000,000	$ 1,000,000
Surveying	1	LS	$ 73,000	$ 73,000
Environmental Protection	1	LS	$ 212,000	$ 212,000
Beach and Dune Nourishment	410,300	CY	$ 18	$ 7,385,400
			Subtotal	$ 8,670,400
			Contingency [35%]	$ 3,034,640
			Capital Cost	$ 11,705,040

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
10.3	764,000	$ 20	$ 15,236,177	$ 80,878	$ 234,880	$ 1,107,924	$ 16,659,860	$ 5,830,951	$ 22,490,811
20.6	764,000	$ 22	$ 16,880,533	$ 89,607	$ 260,229	$ 1,227,497	$ 18,457,866	$ 6,460,253	$ 24,918,120
30.9	764,000	$ 24	$ 18,702,356	$ 99,278	$ 288,314	$ 1,359,974	$ 20,449,922	$ 7,157,473	$ 27,607,394
41.2	764,000	$ 27	$ 20,720,797	$ 109,993	$ 319,431	$ 1,506,748	$ 22,656,968	$ 7,929,939	$ 30,586,907
Total	3,056,000	--	$ 71,539,864	$ 379,756	$ 1,102,854	$ 5,202,142	$ 78,224,617	$ 27,378,616	$ 105,603,233

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
27.4	17,655	72	$ 1,275,369.6	$ 17,075	$ 11,821	$ 39,403	$ 1,343,668	$ 470,284	$ 1,813,951
Total	17,655	--	$ 1,275,370	$ 17,075	$ 11,821	$ 39,403	$ 1,343,668	$ 470,284	$ 1,813,951

Total Costs

Description	Total Cost
Capital	$ 11,705,040
Beach Maintenance	$ 105,603,233
Dune Maintenance	$ 1,813,951
Total Lifetime	$ 119,122,224

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.

4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.

5. Beach Maintenance mobilization, survey, and environmental protection adjusted for inflation for each maintenance cycle

M M MOTT MACDONALD	Grand Isle Cost Estimate			
Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	4/18/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	4/18/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	3	AA	4/18/2017	

Alternative 3A

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 350,000	$ 350,000
Surveying	1	LS	$ 43,000	$ 43,000
Environmental Protection	1	LS	$ 72,000	$ 72,000
Beach and Dune Nourishment	7,200	CY	$ 34	$ 244,800
Offshore Breakwaters	21,700	TON	$ 100	$ 2,170,000
Navigation Aids	6	EA	$ 12,000	$ 72,000
			Subtotal	$ 2,951,800
			Contingency [35%]	$ 1,033,130
			Capital Cost	$ 3,984,930

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Rock Wt (ton)	ock Cost ($/tor	Sand	Rock	Survey	Enviro Pro	Mob/Demob	Nav Aids	Subtotal	Contingency	Total
9.8	220,000	$ 20	$ -	$ 110	$ 4,365,607	$ -	$ 47,404	$ 79,375	$ 385,849	$ -	$ 4,878,235	$ 1,707,382	$ 6,585,618
19.6	220,000	$ 22	$ 10,850	$ 122	$ 4,812,759	$ 1,318,647	$ 52,260	$ 87,505	$ 425,370	$ -	$ 6,696,541	$ 2,343,789	$ 9,040,330
29.4	220,000	$ 24	$ 10,850	$ 134	$ 5,305,711	$ 1,453,711	$ 57,613	$ 96,467	$ 468,939	$ -	$ 7,382,441	$ 2,583,854	$ 9,966,295
39.2	220,000	$ 27	$ -	$ 148	$ 5,849,154	$ -	$ 63,514	$ 106,348	$ 516,971	$ -	$ 6,535,986	$ 2,287,595	$ 8,823,581
49.0	220,000	$ 29	$ 10,850	$ 163	$ 6,448,259	$ 1,766,758	$ 70,019	$ 117,241	$ 569,922	$ -	$ 8,972,199	$ 3,140,270	$ 12,112,469
Total	1,100,000	--	$ 32,550	$ 676	$ 26,781,490	$ 4,539,116	$ 290,809	$ 486,936	$ 2,367,051	$ -	$ 34,465,403	$ 12,062,891	$ 46,528,294

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
2.7	14,025	56	$ 792,379.6	$ 13,354	$ 9,245	$ 30,817	$ 845,796	$ 296,028	$ 1,141,824
5.4	14,025	58	$ 813,956.1	$ 13,718	$ 9,497	$ 31,656	$ 868,827	$ 304,089	$ 1,172,916
8.1	14,025	60	$ 836,120.1	$ 14,091	$ 9,755	$ 32,518	$ 892,485	$ 312,370	$ 1,204,854
10.8	14,025	61	$ 858,887.7	$ 14,475	$ 10,021	$ 33,404	$ 916,787	$ 320,875	$ 1,237,663
13.5	14,025	63	$ 882,275.3	$ 14,869	$ 10,294	$ 34,313	$ 941,751	$ 329,613	$ 1,271,364
16.2	14,025	65	$ 906,299.7	$ 15,274	$ 10,574	$ 35,247	$ 967,395	$ 338,588	$ 1,305,984
18.9	14,025	66	$ 930,978.2	$ 15,690	$ 10,862	$ 36,207	$ 993,737	$ 347,808	$ 1,341,546
21.6	14,025	68	$ 956,328.8	$ 16,117	$ 11,158	$ 37,193	$ 1,020,797	$ 357,279	$ 1,378,076
24.3	14,025	70	$ 982,369.7	$ 16,556	$ 11,462	$ 38,206	$ 1,048,593	$ 367,008	$ 1,415,601
27	14,025	72	$ 1,009,119.6	$ 17,007	$ 11,774	$ 39,246	$ 1,077,146	$ 377,001	$ 1,454,148
29.7	14,025	74	$ 1,036,598.0	$ 17,470	$ 12,094	$ 40,315	$ 1,106,477	$ 387,267	$ 1,493,744
32.4	14,025	76	$ 1,064,824.6	$ 17,946	$ 12,424	$ 41,413	$ 1,136,607	$ 397,812	$ 1,534,419
35.1	14,025	78	$ 1,093,819.8	$ 18,434	$ 12,762	$ 42,540	$ 1,167,556	$ 408,645	$ 1,576,201
37.8	14,025	80	$ 1,123,604.5	$ 18,936	$ 13,110	$ 43,699	$ 1,199,349	$ 419,772	$ 1,619,121
40.5	14,025	82	$ 1,154,200.3	$ 19,452	$ 13,467	$ 44,889	$ 1,232,007	$ 431,203	$ 1,663,210
43.2	14,025	85	$ 1,185,629.3	$ 19,981	$ 13,833	$ 46,111	$ 1,265,555	$ 442,944	$ 1,708,499
45.9	14,025	87	$ 1,217,914.0	$ 20,526	$ 14,210	$ 47,367	$ 1,300,016	$ 455,006	$ 1,755,022
48.6	14,025	89	$ 1,251,077.8	$ 21,084	$ 14,597	$ 48,656	$ 1,335,416	$ 467,395	$ 1,802,811
Total	252,450	--	$ 18,096,383	$ 304,979	$ 211,139	$ 703,797	$ 19,316,298	$ 6,760,704	$ 26,077,002

Total Costs

Description	Total Cost
Capital	$ 3,984,930
Beach Maintenance	$ 46,528,294
Dune Maintenance	$ 26,077,002
Total Lifetime	$ 76,590,226

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.

4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.

5. Beach Maintenance mobilization, survey, and environmental protection adjusted for inflation for each maintenance cycle

6. Assumed 50% replacement of rock approximately every 15 years.

MOTT MACDONALD

Grand Isle Cost Estimate

Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	4/18/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	4/18/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	3	AA	4/18/2017	

Alternative 3B

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 700,000	$ 700,000
Surveying	1	LS	$ 55,000	$ 55,000
Environmental Protection	1	LS	$ 142,000	$ 142,000
Beach and Dune Nourishment	152,000	CY	$ 18	$ 2,736,000
Offshore Breakwaters	21,700	TON	$ 100	$ 2,170,000
Navigation Aids	6	EA	$ 12,000	$ 72,000
			Subtotal	$ 5,875,000
			Contingency [35%]	$ 2,056,250
			Capital Cost	$ 7,931,250

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Rock Wt (ton)	Rock Cost ($/ton)	Sand	Survey	Rock Cost	Enviro Pro	Mob/Demob	Nav Aids	Subtotal	Contingency	Total
13.5	364,000	$ 21	$ 10,850	$ 103	$ 7,493,978	$ 62,907	$ 1,113,302	$ 162,415	$ 800,639	$ -	$ 9,633,241	$ 3,371,634	$ 13,004,876
27	364,000	$ 24	$ 10,850	$ 106	$ 8,571,385	$ 71,951	$ 1,146,578	$ 185,766	$ 915,746	$ -	$ 10,891,426	$ 3,811,999	$ 14,703,425
40.5	364,000	$ 27	$ 10,850	$ 109	$ 9,803,689	$ 82,296	$ 1,185,860	$ 212,473	$ 1,047,403	$ -	$ 12,331,721	$ 4,316,102	$ 16,647,823
Total	1,092,000	--	$ 32,550	$ 318	$ 25,869,052	$ 217,155	$ 3,445,740	$ 560,654	$ 2,763,788	$ -	$ 32,856,388	$ 11,499,736	$ 44,356,124

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
24.6	22,605	$ 70.3	$ 1,588,082	$ 16,605	$ 11,496	$ 38,320	$ 1,654,504	$ 579,076	$ 2,233,580
49.2	22,605	$ 89.7	$ 2,028,518	$ 21,211	$ 14,684	$ 48,948	$ 2,113,360	$ 739,676	$ 2,853,037
Total Maintenance	45,210	--	$ 3,616,600	$ 37,816	$ 26,180	$ 87,268	$ 3,767,864	$ 1,318,752	$ 5,086,617

Total Costs

Description	Total Cost
Capital	$ 7,931,250
Beach Maintenance	$ 44,356,124
Dune Maintenance	$ 5,086,617
Total Lifetime	$ 57,373,991

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.

4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.

5. Beach Maintenance mobilization, survey, rock, and environmental protection adjusted for inflation for each maintenance cycle

6. Assumed 50% replacement of rock approximately every 15 years.

MOTT MACDONALD	Grand Isle Cost Estimate			
Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	4/18/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	4/18/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	3	AA	4/18/2017	

Alternative 4A_v1

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 350,000	$ 350,000
Surveying	1	LS	$ 43,000	$ 43,000
Environmental Protection	1	LS	$ 34,000	$ 34,000
Beach and Dune Nourishment	5,300	CY	$ 34	$ 180,200
Headland Breakwaters	7,200	TON	$ 100	$ 720,000
Navigation Aids	4	EA	$ 12,000	$ 48,000
			Subtotal	$ 1,375,200
			[35%]	$ 481,320
			Capital Cost	$ 1,856,520

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Rock Wt (ton)	Rock Cost ($/ton)	Sand	Survey	Enviro Pro	Rock	Nav Aids	Mob/Demob	Subtotal	Contingency	Total
7.1	94,000	$ 19	$ -	$ 107	$ 1,815,859	$ 46,148	$ 36,489	$ -	$ -	$ 375,621	$ 2,274,117	$ 795,941	$ 3,070,057
14.2	94,000	$ 21	$ 3,600	$ 115	$ 1,948,785	$ 49,526	$ 39,160	$ 414,635	$ -	$ 403,117	$ 2,855,223	$ 999,328	$ 3,854,551
21.3	94,000	$ 22	$ -	$ 124	$ 2,091,441	$ 53,151	$ 42,027	$ -	$ -	$ 432,627	$ 2,619,246	$ 916,736	$ 3,535,982
28.4	94,000	$ 24	$ 3,600	$ 133	$ 2,244,540	$ 57,042	$ 45,103	$ 477,562	$ -	$ 464,296	$ 3,288,543	$ 1,150,990	$ 4,439,533
35.5	94,000	$ 26	$ -	$ 142	$ 2,408,847	$ 61,218	$ 48,405	$ -	$ -	$ 498,284	$ 3,016,753	$ 1,055,863	$ 4,072,616
42.6	94,000	$ 28	$ 3,600	$ 153	$ 2,585,181	$ 65,699	$ 51,948	$ 550,038	$ -	$ 534,760	$ 3,787,626	$ 1,325,669	$ 5,113,295
49.7	94,000	$ 30	$ -	$ 164	$ 2,774,423	$ 70,508	$ 55,751	$ -	$ -	$ 573,905	$ 3,474,587	$ 1,216,106	$ 4,690,693
Total	658,000	--	$ 10,800	$ 938	$ 15,869,075	$ 403,292	$ 318,882	$ 1,442,235	$ -	$ 3,282,610	$ 21,316,094	$ 7,460,633	$ 28,776,727

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
2.4	14,190	56	$ 799,312.1	$ 13,314	$ 9,218	$ 30,725	$ 852,569	$ 298,399	$ 1,150,968
4.8	14,190	58	$ 818,630.0	$ 13,636	$ 9,440	$ 31,468	$ 873,174	$ 305,611	$ 1,178,785
7.2	14,190	59	$ 838,414.9	$ 13,966	$ 9,668	$ 32,228	$ 894,277	$ 312,997	$ 1,207,274
9.6	14,190	61	$ 858,677.9	$ 14,303	$ 9,902	$ 33,007	$ 915,890	$ 320,562	$ 1,236,452
12	14,190	62	$ 879,430.6	$ 14,649	$ 10,141	$ 33,805	$ 938,025	$ 328,309	$ 1,266,334
14.4	14,190	63	$ 900,684.9	$ 15,003	$ 10,387	$ 34,622	$ 960,696	$ 336,244	$ 1,296,939
16.8	14,190	65	$ 922,452.8	$ 15,365	$ 10,638	$ 35,458	$ 983,914	$ 344,370	$ 1,328,284
19.2	14,190	67	$ 944,746.9	$ 15,737	$ 10,895	$ 36,315	$ 1,007,694	$ 352,693	$ 1,360,386
21.6	14,190	68	$ 967,579.7	$ 16,117	$ 11,158	$ 37,193	$ 1,032,048	$ 361,217	$ 1,393,265
24	14,190	70	$ 990,964.4	$ 16,507	$ 11,428	$ 38,092	$ 1,056,991	$ 369,947	$ 1,426,937
26.4	14,190	72	$ 1,014,914.3	$ 16,905	$ 11,704	$ 39,013	$ 1,082,536	$ 378,888	$ 1,461,424
28.8	14,190	73	$ 1,039,442.9	$ 17,314	$ 11,987	$ 39,956	$ 1,108,699	$ 388,045	$ 1,496,744
31.2	14,190	75	$ 1,064,564.4	$ 17,733	$ 12,276	$ 40,921	$ 1,135,494	$ 397,423	$ 1,532,918
33.6	14,190	77	$ 1,090,293.1	$ 18,161	$ 12,573	$ 41,910	$ 1,162,937	$ 407,028	$ 1,569,965
36	14,190	79	$ 1,116,643.5	$ 18,600	$ 12,877	$ 42,923	$ 1,191,043	$ 416,865	$ 1,607,909
38.4	14,190	81	$ 1,143,630.8	$ 19,050	$ 13,188	$ 43,960	$ 1,219,829	$ 426,940	$ 1,646,769
40.8	14,190	83	$ 1,171,270.3	$ 19,510	$ 13,507	$ 45,023	$ 1,249,310	$ 437,258	$ 1,686,568
43.2	14,190	85	$ 1,199,577.8	$ 19,981	$ 13,833	$ 46,111	$ 1,279,504	$ 447,826	$ 1,727,330
45.6	14,190	87	$ 1,228,569.5	$ 20,464	$ 14,168	$ 47,225	$ 1,310,427	$ 458,649	$ 1,769,076
48	14,190	89	$ 1,258,261.8	$ 20,959	$ 14,510	$ 48,367	$ 1,342,098	$ 469,734	$ 1,811,832
Total	283,800	--	$ 20,248,063	$ 337,273	$ 233,497	$ 778,323	$ 21,597,155	$ 7,559,004	$ 29,156,159

Total Costs

Description		Total Cost
Capital	$	1,856,520
Beach Maintenance	$	28,776,727
Dune Maintenance	$	29,156,159
Total Lifetime	$	59,789,406

Notes

1. Assumed 1% yearly inflation

2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.

3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.

4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.

5. Beach Maintenance mobilization, survey, rock, and environmental protection adjusted for inflation for each maintenance cycle

6. Assumed 50% replacement of rock approximately every 15 years.

MOTT MACDONALD

Grand Isle Cost Estimate

Project:		Prepared by:	Date:	Appendix
Grand Isle Coastal Engineering Analysis		PWM	5/2/2017	
Description:		Checked by:	Date:	A
Conceptual Cost Estimate		AA	5/2/2017	
Calculation No:	Rev. No.	Reviewed by:	Date:	
1	4	AA	5/2/2017	

Alternative 4B_v1

Capital Costs

Description	Est. Quantity	Units	Unit Price	Extended Price
Mobilization and Demobilization	1	LS	$ 700,000	$ 700,000
Surveying	1	LS	$ 79,000	$ 79,000
Environmental Protection	1	LS	$ 108,000	$ 108,000
Beach and Dune Nourishment	152,000	CY	$ 18	$ 2,736,000
Headland Breakwaters	7,200	TON	$ 100	$ 720,000
Navigation Aids	6	EA	$ 12,000	$ 72,000
			Subtotal	$ 4,415,000
			Contingency [35%]	$ 1,545,250
			Capital Cost	$ 5,960,250

Beach Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Rock	Rock/ton	Sand	Rock	Survey	Enviro Pro	Nav Aids	Mob/Demob	Subtotal	Contingency	Total
9.8	246,000	$ 20	$ -	$ 110	$ 4,881,543	$ -	$ 87,092	$ 119,062	$ -	$ 771,698	$ 5,859,394	$ 2,050,788	$ 7,910,182
19.6	246,000	$ 22	$ 3,600	$ 122	$ 5,381,540	$ 437,524	$ 96,012	$ 131,257	$ -	$ 850,740	$ 6,897,073	$ 2,413,975	$ 9,311,048
29.4	246,000	$ 24	$ 3,600	$ 134	$ 5,932,749	$ 482,337	$ 105,846	$ 144,701	$ -	$ 937,878	$ 7,603,512	$ 2,661,229	$ 10,264,742
39.2	246,000	$ 27	$ -	$ 148	$ 6,540,417	$ -	$ 116,688	$ 159,522	$ -	$ 1,033,941	$ 7,850,569	$ 2,747,699	$ 10,598,268
49	246,000	$ 29	$ 3,600	$ 163	$ 7,210,326	$ 586,205	$ 128,640	$ 175,862	$ -	$ 1,139,844	$ 9,240,877	$ 3,234,307	$ 12,475,184
Total	1,230,000	--	$ 10,800	$ 676	$ 29,946,575	$ 1,506,066	$ 534,277	$ 730,404	$ -	$ 4,734,102	$ 37,451,425	$13,107,999	$ 50,559,424

Dune Maintenance Costs

Year	Sand Vol (cy)	Sand Cost ($/cy)	Sand	Survey	Enviro Pro	Mob/Demob	Subtotal	Contingency	Total
25.2	22,770	71	$ 1,609,253	$ 16,705	$ 11,565	$ 38,550	$ 1,676,072	$ 586,625	$ 2,262,698
Total	22,770	--	$ 1,609,253	$ 16,705	$ 11,565	$ 38,550	$ 1,676,072	$ 586,625	$ 2,262,698

Total Costs

Description	Total Cost
Capital	$ 5,960,250
Beach Maintenance	$ 50,559,424
Dune Maintenance	$ 2,262,698
Total Lifetime	$ 58,782,371

Notes

1. Assumed 1% yearly inflation
2. Assumed $55/cy (present dollars) for dune fill maintenance material. Adjusted for inflation in future years.
3. Assumed $30,000 mobilization/demobilization, $13,000 survey, and $9,000 environmental protection costs for dune renoursihment (present dollars). Adjusted for inflation in future years.
4. Assumed $18/cy for beach fill maintenance material (present dollars). Cost adjusted for inflation in future years. Decreased cost due to large quantity necessary for beach fill.
5. Beach Maintenance mobilization, survey, rock, and environmental protection adjusted for inflation for each maintenance cycle
6. Assumed 50% replacement of rock approximately every 15 years.

B. Supplemental SBEACH results

B.1 Scaled storm hydrographs

As discussed in Section 4.1.1, the storm hydrograph from Hurricane Danny was scaled to match storm conditions calculated for return periods of 1, 2, 5, 7.5, 10, 15, 20, 25, 50, and 100 years. Figure 57 shows the scaled wave height, wave period and water level for Alt 1A, 1B, 2A and 2B (no hard structure). For Alt 3 and 4, due to the presence of hard structure (and transmissivity of waves over the structure) the wave heights gets reduced. Figure 58 shows the wave height graphs applicable to no hard structure alternatives, Alt3, and Alt 4 alternatives.

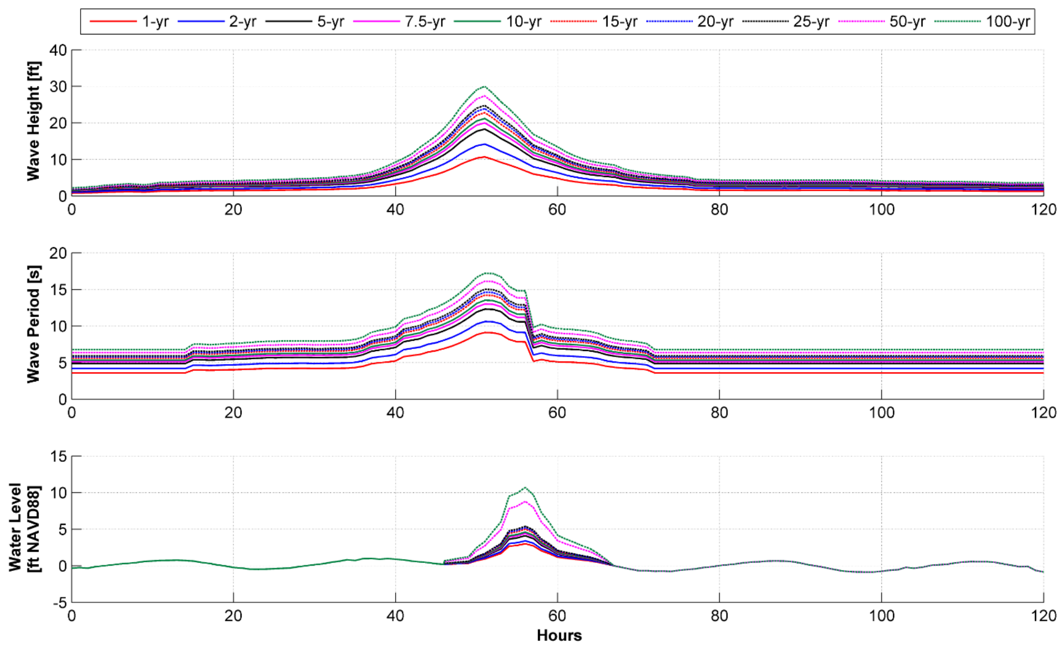

Figure 57. Scaled storm hydrographs for different return period events

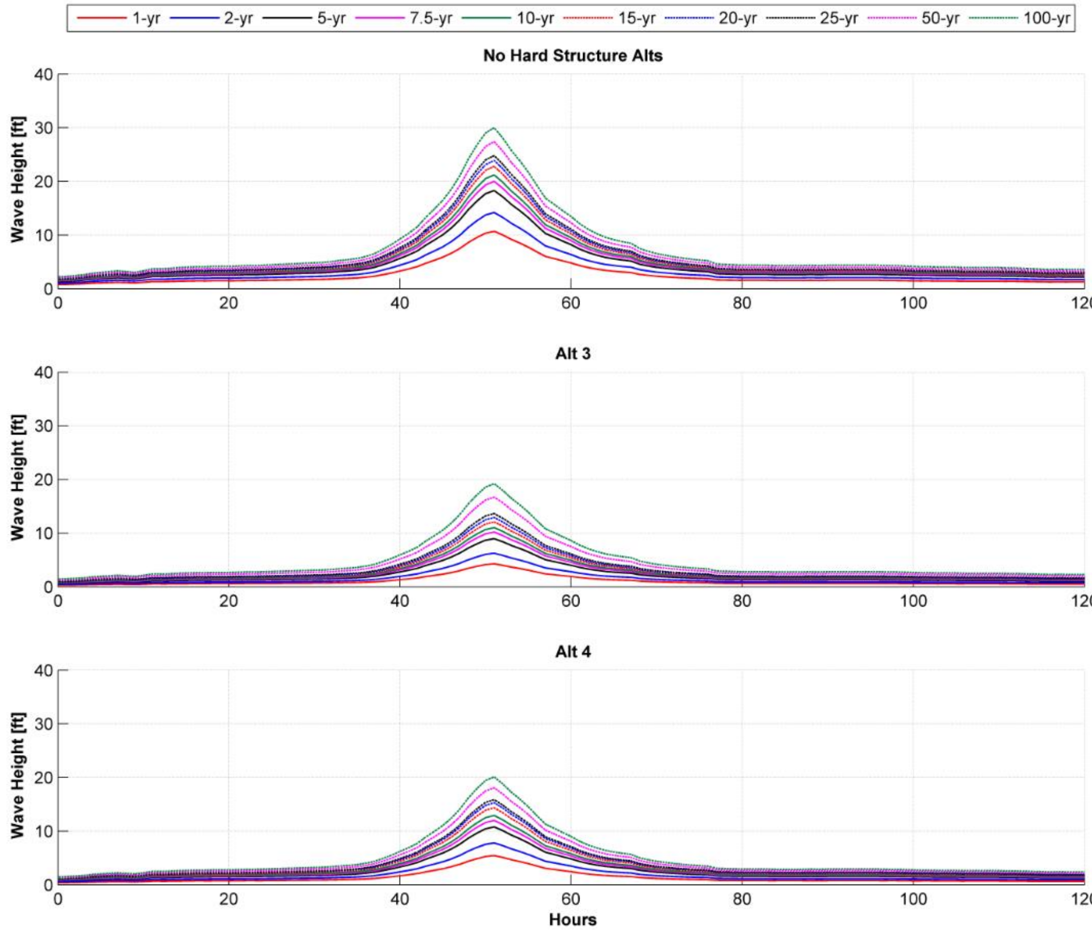

Figure 58. Scaled storm wave heights for no hard structure, Alt3, And Alt4 alternatives

B.2 Contour retreat plots

The following figures show the storm impacted profiles for different alternatives for different return period storm event. It should be noted that for Alt 3A, 3B, 4A, and 4B contour retreat was computed using the average of contour retreat for the profile behind the alternative and the profile in the gaps. The figures below show the profile evolution behind the alternative segmented and headland breakwater for Alt 3A/3B and Alt 4A/4B, respectively.

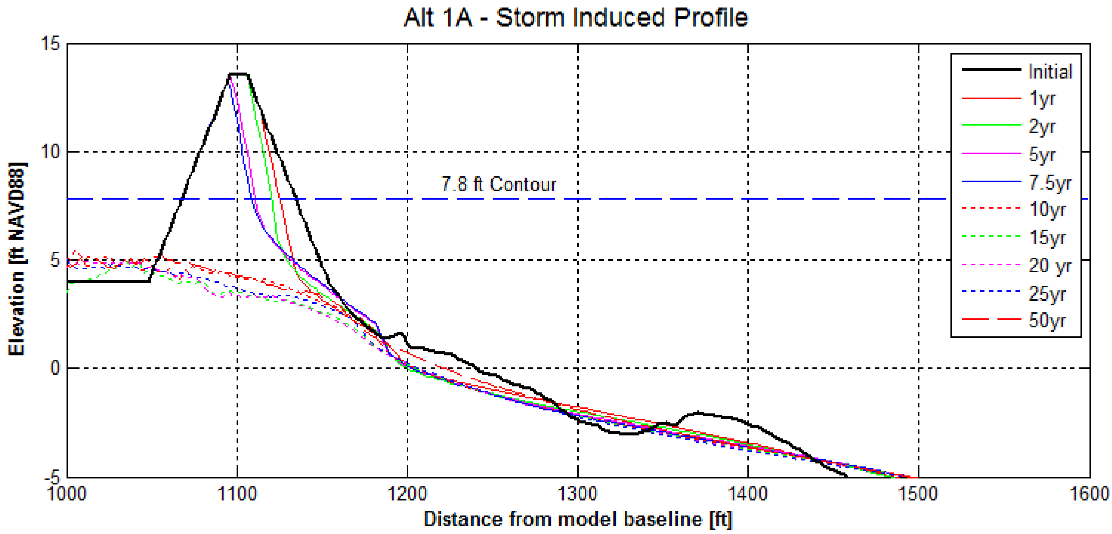

Figure 59. Contour retreat plot for Alt 1A

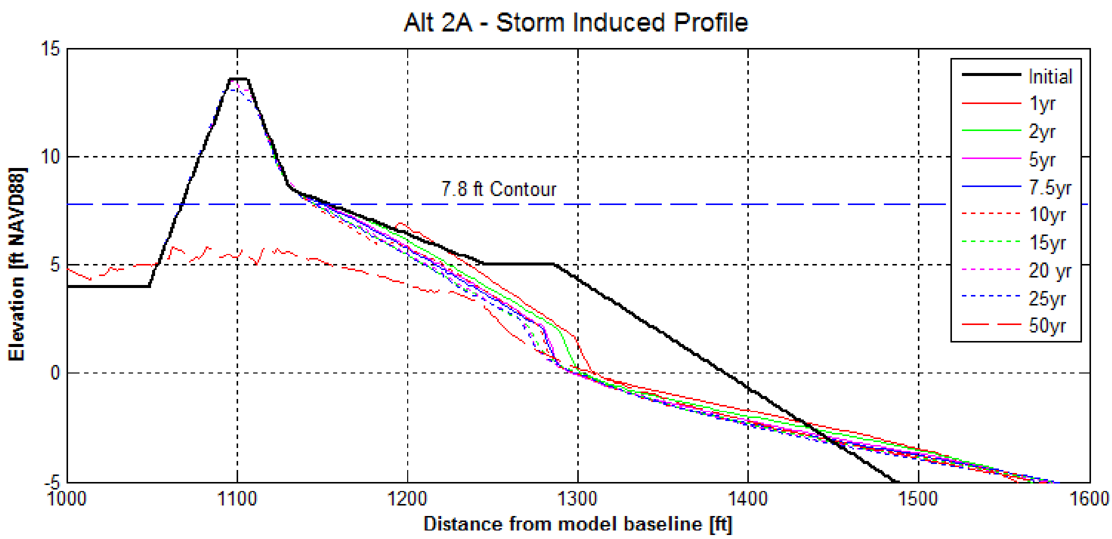

Figure 60. Contour retreat plot for Alt 2A

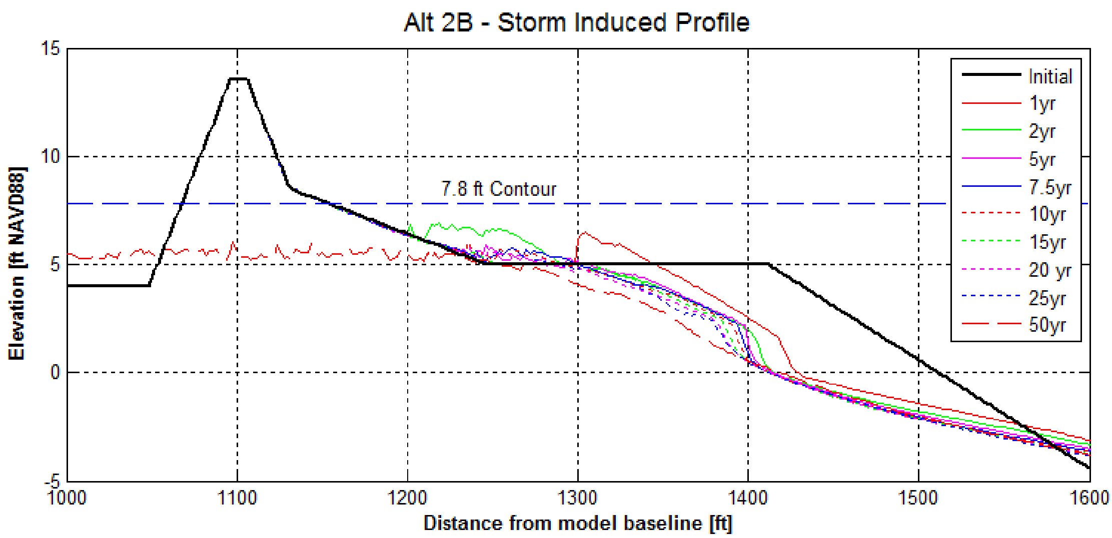

Figure 61. Contour retreat plot for Alt 2B

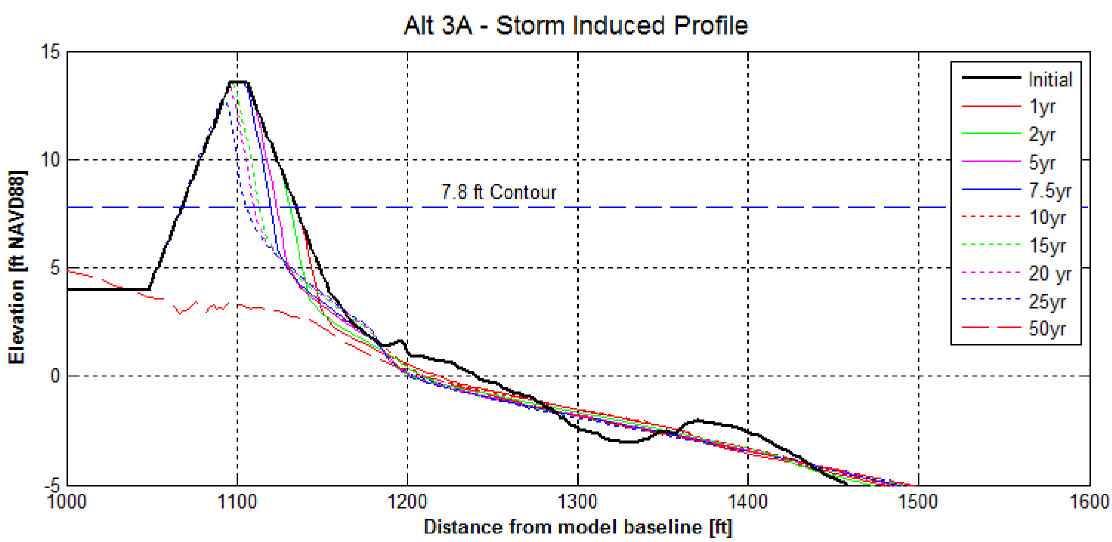

Figure 62. Contour retreat plot for Alt 3A (profile behind the breakwater)

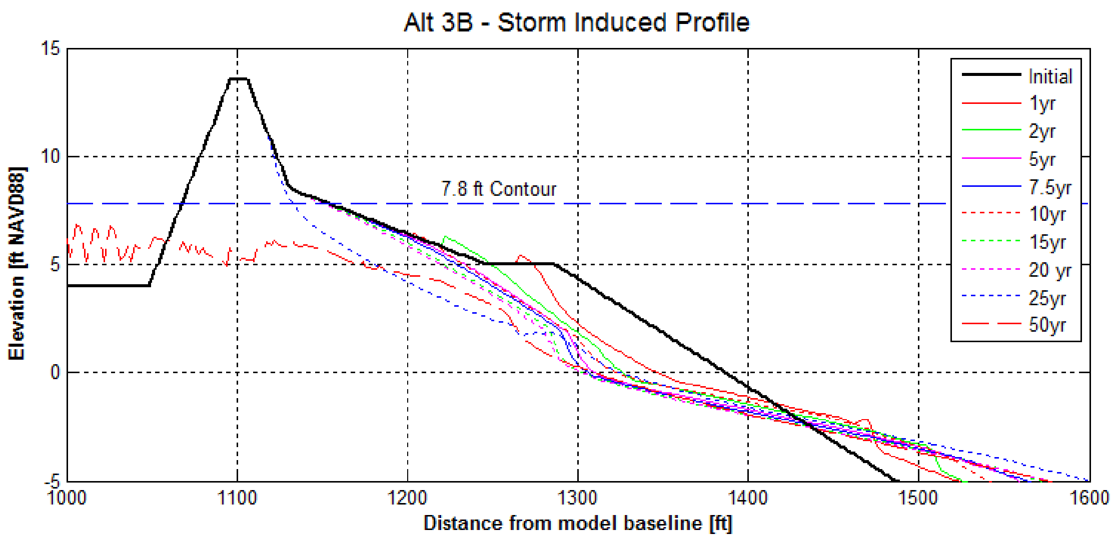

Figure 63. Contour retreat plot for Alt 3B (profile behind the breakwater)

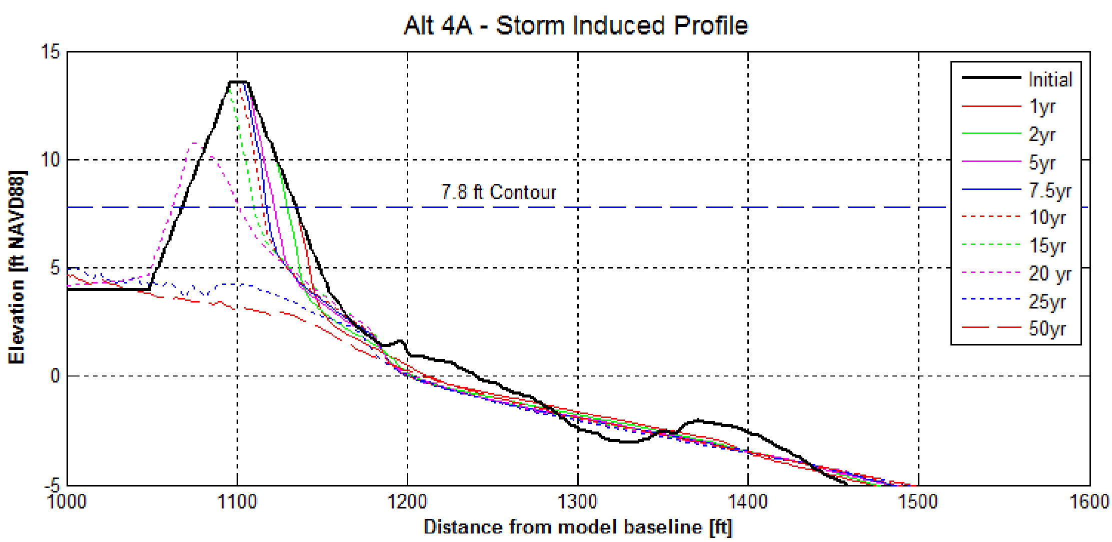

Figure 64. Contour retreat plot for Alt 4A (profile behind T-head groin)

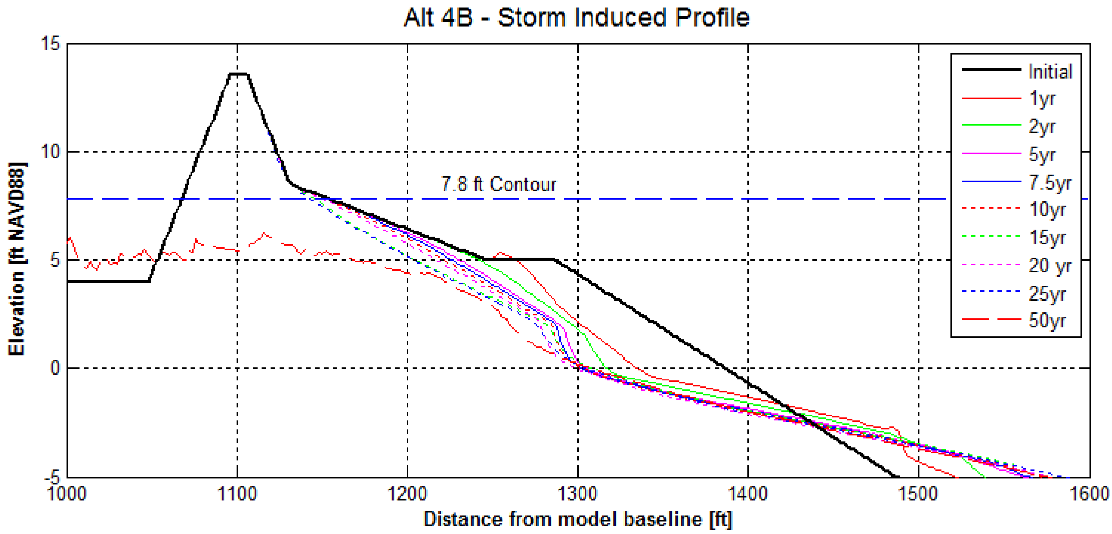

Figure 65. Contour retreat plot for Alt 4B (profile behind T-head groin)

C. Shoreline Response: 4-Year Shoreline Results

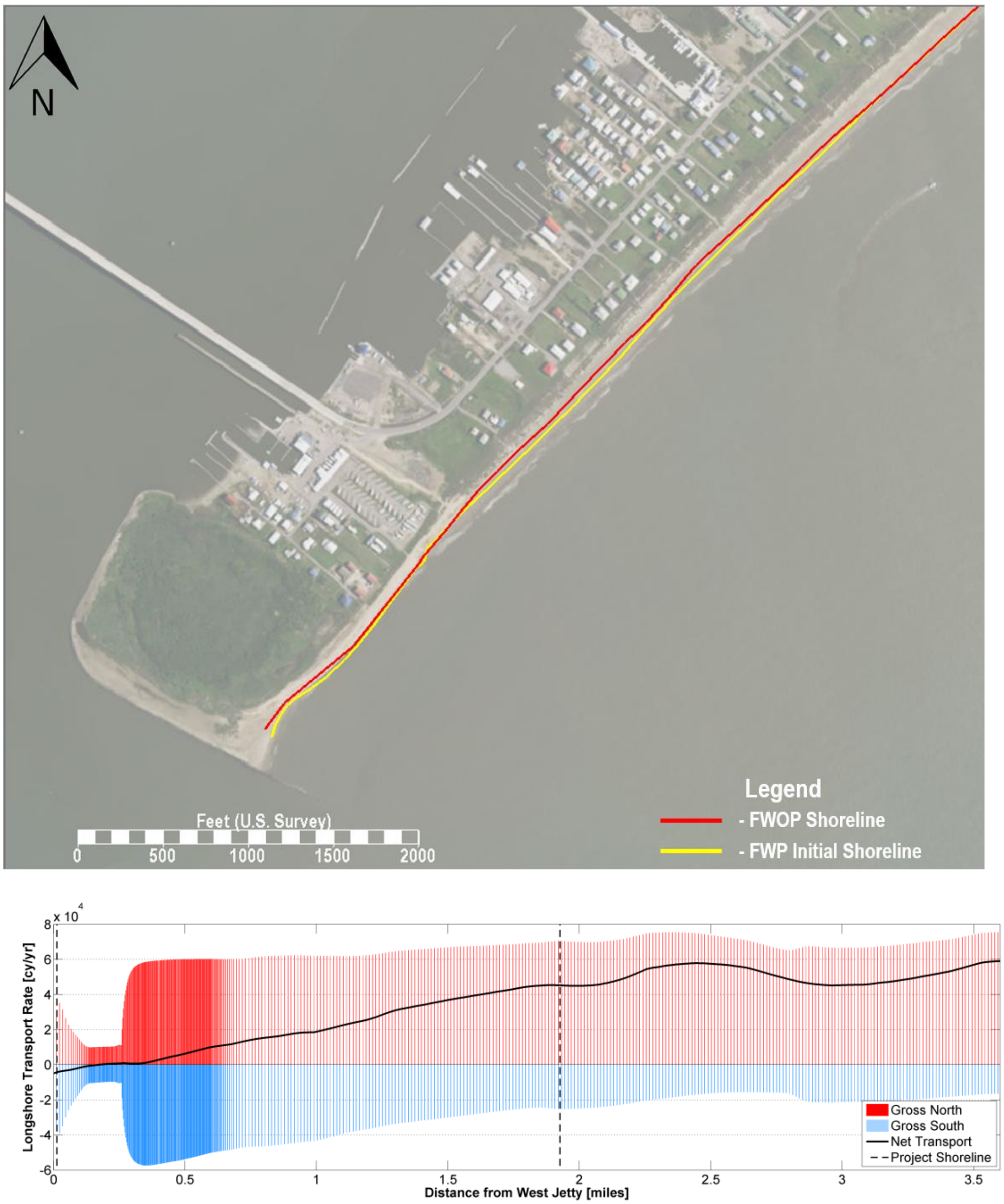

Figure 66. Top: Shoreline response after 4 years for FWOP/1A/1B scenarios (red) and FWOP/1A/1B initial shoreline (yellow). Bottom: Average LST rates for first 4 years for FWOP/1A/1B scenarios.

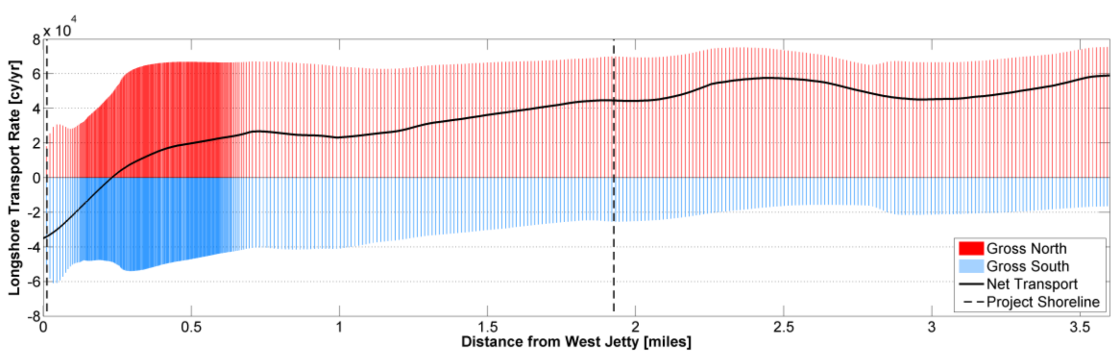

Figure 67. Top: Shoreline response after 4 years for Alt 2A (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 2A.

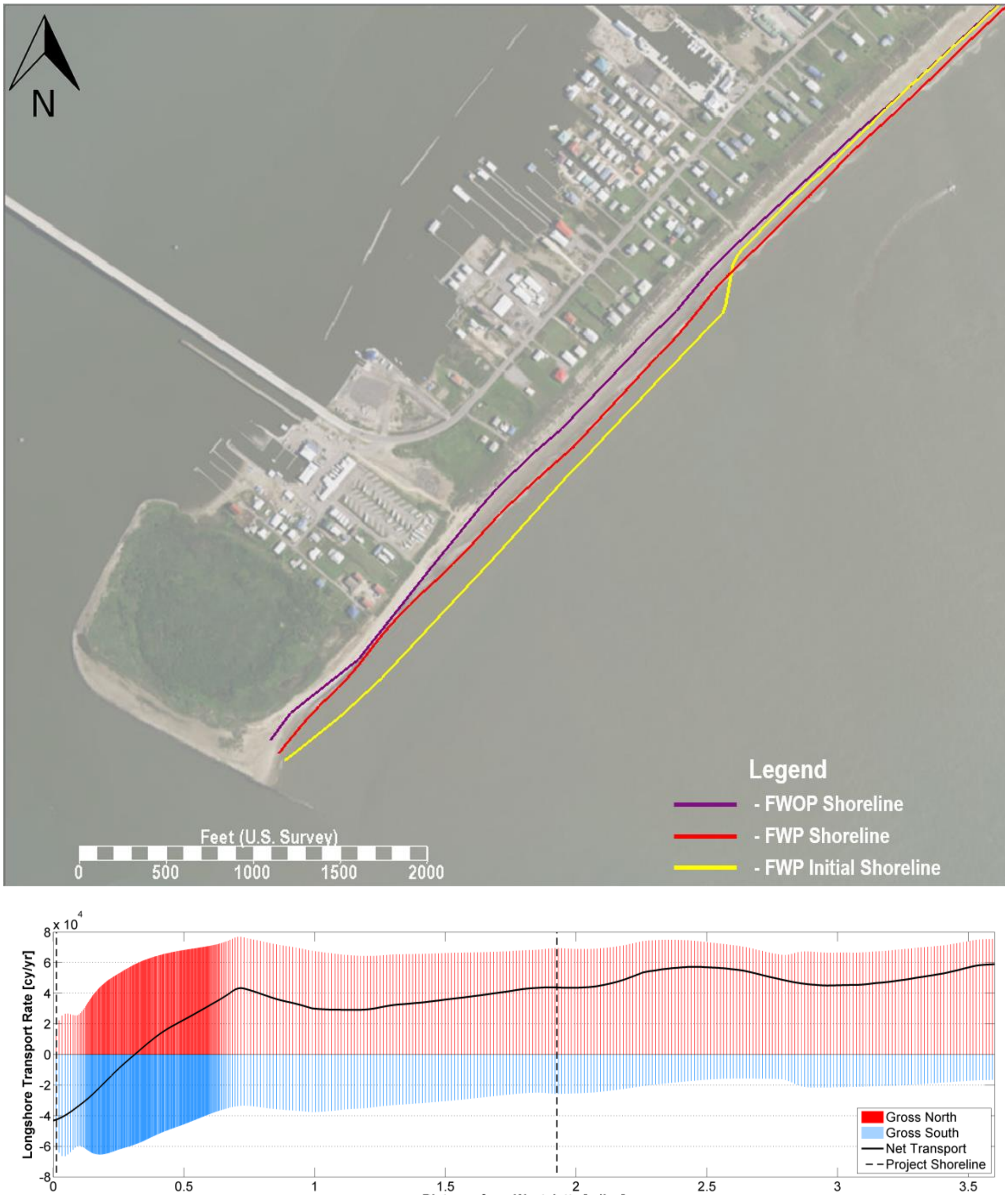

Figure 68. Top: Shoreline response after 4 years for Alt 2B (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 2B.

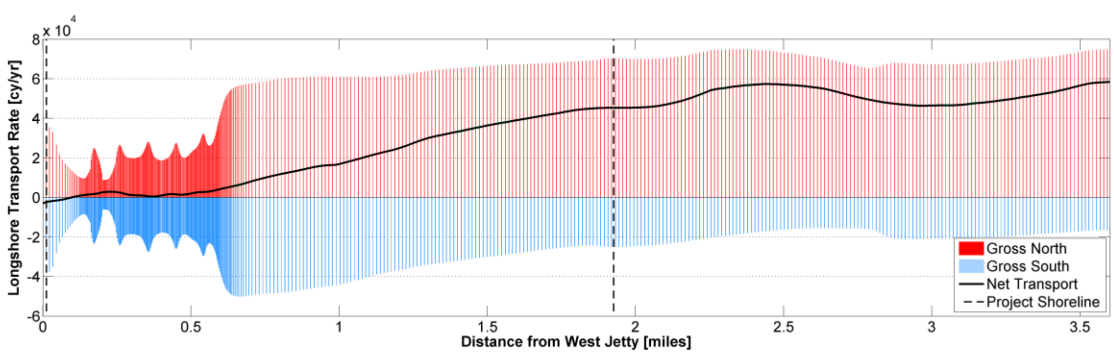

Figure 69. Top: Shoreline response after 4 years for Alt 3A (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 3A.

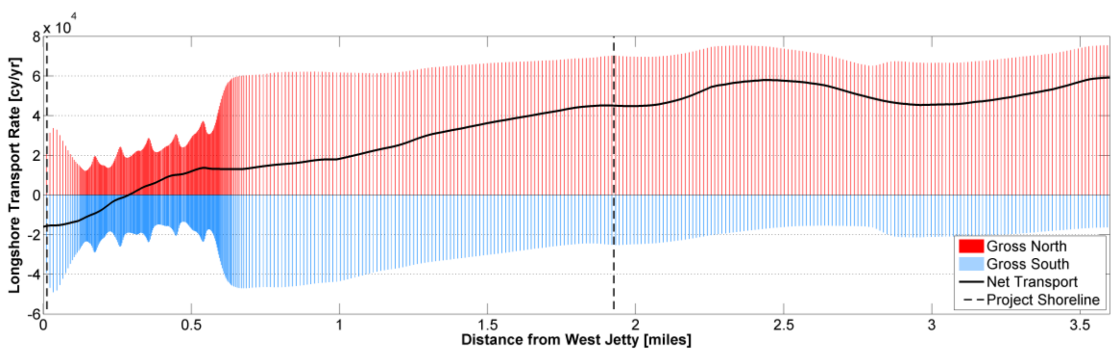

Figure 70. Top: Shoreline response after 4 years for Alt 3B (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 3B.

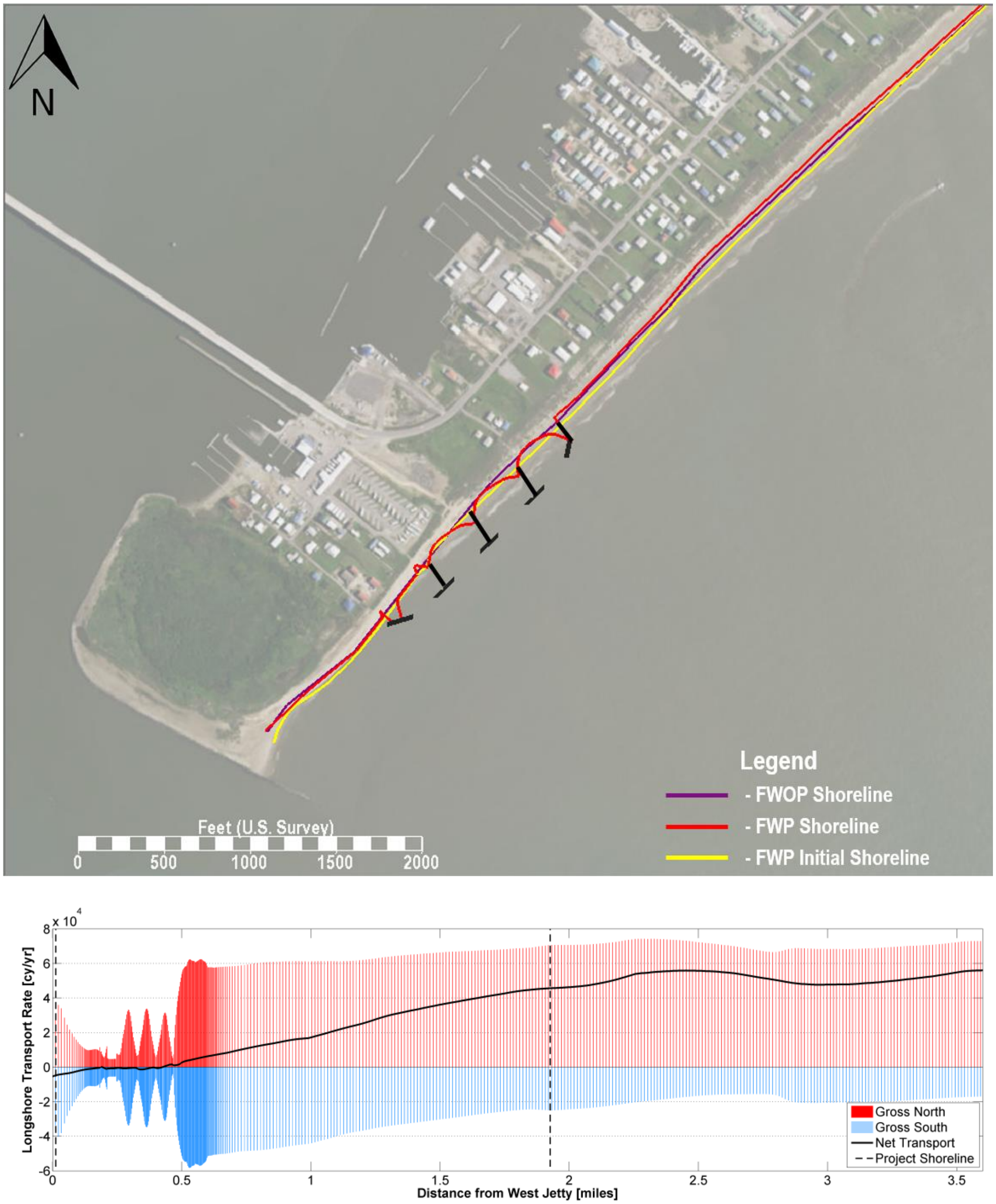

Figure 71. Top: Shoreline response after 4 years for Alt 4A (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 4A.

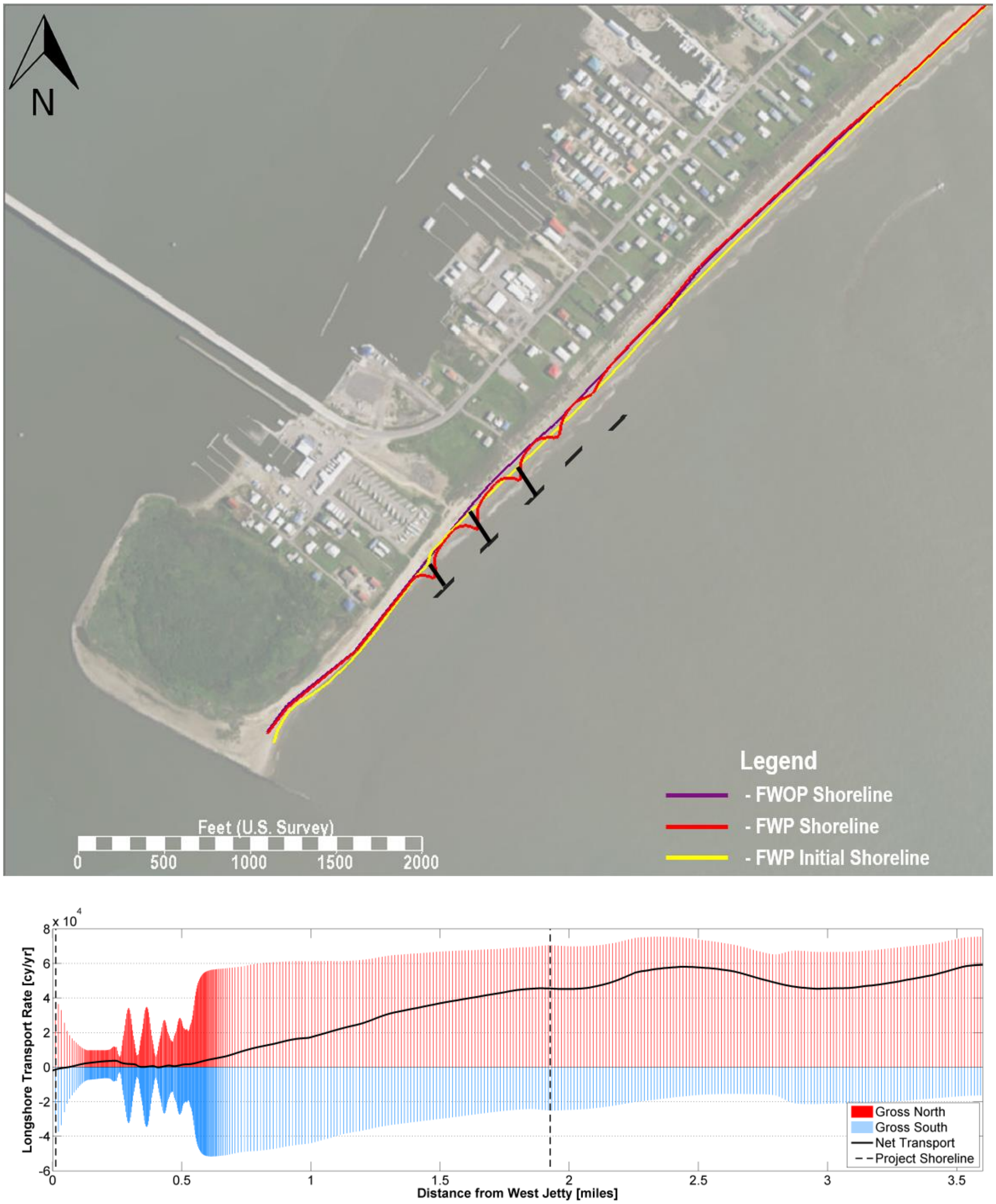

Figure 72. Top: Shoreline response after 4 years for Alt 4A_v1 (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 4A_v1.

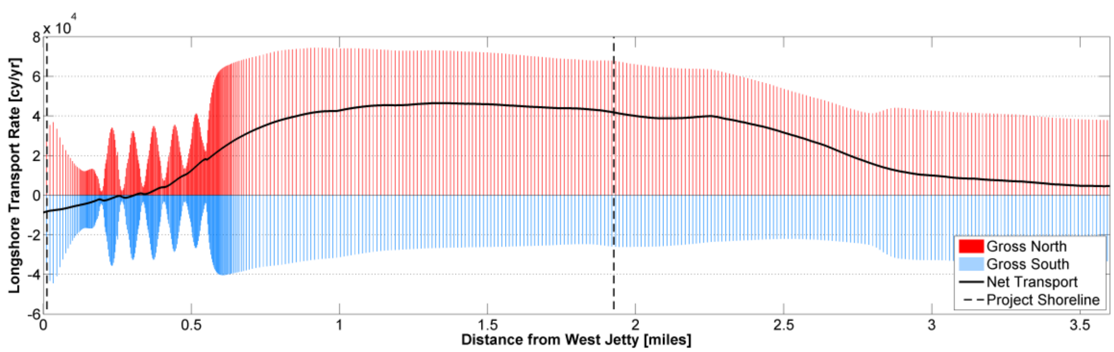

Figure 73. Top: Shoreline response after 4 years for Alt 4B (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 4B.

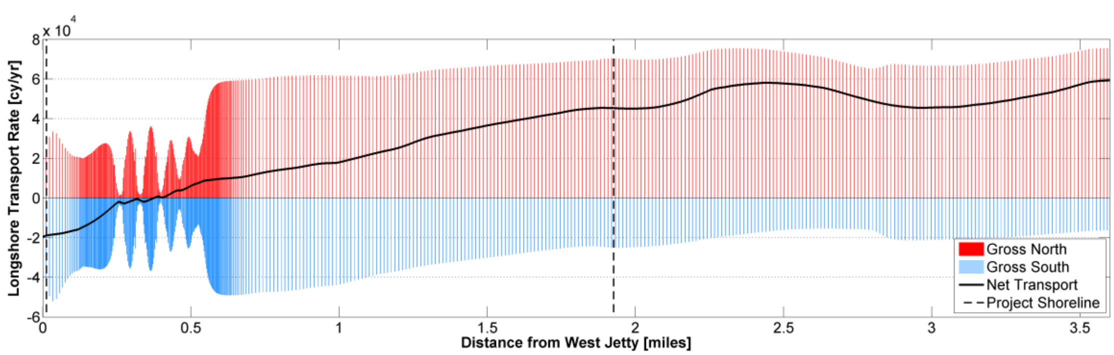

Figure 74. Top: Shoreline response after 4 years for Alt 4B_v1 (red). Also shown is the initial shoreline (yellow) and FWOP shoreline after 4 years (purple). Bottom: Average LST rates for first 4 years for Alt 4B_v1.

Grand Isle and Vicinity Breakwater Design

Draft Report

March 25, 2019

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Grand Isle and Vicinity Breakwater Design

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March 25, 2019

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Issue and revision record

Revision	Date	Originator	Checker	Approver	Description
0	3/25/19	V.Curto C. Harter	V.Curto C.Day	J.Carter	Draft report submitted to CPRA for comments
1	3/28/19	V.Curto C.Harter	V.Curto	J.Carter	Draft report submitted to USACE for comments

Document reference: 400269 **|** 1 **|** a

Information class: Standard

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Executive summary

This report has been prepared in accordance with CPRA IDIQ Contract No. 4400012419 for work performed under Task 3 – Grand Isle and Vicinity Breakwater Design. This report extends the analysis of the Grand Isle Levee Dune and Beach Stabilization and Beach Nourishment Project conducted by Mott MacDonald (2017). The 2017 analysis concluded with a preferred alternative consisting of Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill. The analysis presented in this report includes the optimization of breakwater field components and assessing the impacts of using the Caminada Pass ebb shoal as a borrow source for beach fill.

The goal of the coastal engineering and alternatives analysis is to understand the coastal dynamics of the Caminada Pass ebb shoal and the Grand Isle shoreline. The ultimate objective of this study is to optimize the proposed breakwater field on the southwestern end of the Island so that the breakwaters do not interfere with the natural bypass of sand from the Caminada Headland onto Grand Isle.

A review of existing data was conducted as part of the coastal engineering analysis, and included tidal datums, statistical and extreme value analyses of water surface elevations, winds, and waves, along with bathymetric data sources and sediment size. Much of the existing data has been referenced from the Grand Isle Levee Dune and Beach Stabilization and Beach Nourishment Project (Mott MacDonald, 2017). The recent bathymetric data sources included data from 2005, 2015, 2016, 2017, and 2018.

To better understand the Caminada Pass shoal dynamics and the associated sediment bypassing, a 2D numerical model was developed. The intent of the numerical model was to have a tool that simulated transport across Caminada Pass ebb shoal and evaluated the impacts on the pass dynamics and the project's shoreline from (a) implementing breakwaters and (b) utilizing Caminada Pass ebb shoal as a borrow source. The breakwaters and dredge borrow pits impacts have been evaluated based on a relative comparison, i.e. without versus with project conditions. The intent of the model is not to robustly simulate all sediment transport and morphological processes in Caminada Pass ebb shoal and Grand Isle shoreline but to understand the impacts the proposed alternatives have on the coastal dynamics to optimize the design.

Numerical modeling was conducted using the process-based numerical model suite Delft3D with nested model domains. The global model included the coupling of circulation and waves covering the full extent of Barataria Bay. The nested model included the coupling of circulation, waves, and sediment transport with higher resolution at the project site. The global bathymetric surface was based on the 2005 Barataria Bay model surface in combination with the 2015 BICM data. Hydrodynamic calibration was performed using ADCP water surface elevation and current velocity data collected in 2005. A reduced time series of environmental conditions between June 1, 2015 and June 1, 2018 was used as environmental forcing. To account for the scale in which morphological changes occur versus the hydrodynamic time scales, a time-varying Morphological Acceleration Factor (MORFAC) was used. An existing-conditions simulation using the 2015 bathymetry as the initial condition provided the basis for the alternative comparative analysis.

The results of the existing condition model indicated a net transport field directed toward the northeast with increasing sediment transport in the center portion of the island. Results illustrated the sediment bypassing from Elmer's Island over the Caminada Pass ebb shoal onto Gran Isle. The analysis also indicated the presence of a divergence node on the western end of

Grand Isle resulting in an erosional hot spot. The erosional area extends between 0 mi to approximately 0.6 mi from the jetty where the Grand Isle shoreline stabilizes. It has been noted the 0.6 mi location matches the eastern end of the 2017 revetment at station 51+00. The model results agree with field observations and the previous analysis conducted by Mott MacDonald, (2017).

Two different breakwater fields consisting of 5 and 10 breakwaters have been evaluated using the Delft3D model. The results have shown that both alternatives have no negative impacts on sediment bypassing from the Caminada Headlands to Grand Isle. The model results also indicated the 5-breakwater field performs better than the 10-breakwater field alternative and therefore, the 5-breakwater field is the recommended alternative.

Delft3D results indicate the 5-breakwater alternative does not reach the location where the sediment bypassing attaches onto the Grand Isle shoreline, while the 10-breakwater alternative extends to the location where the sediment bypassing reaches the island. Both alternatives show improvements by reducing erosion from 0 mi to approximately 0.6 mi from the jetty. However, a larger downdrift erosive extent is observed for the 10-breakwater alternative than for the 5-breakwater alternative. These results are further quantified using Gencade one-line model developed in the work by Mott MacDonald, 2017.

The Gencade analysis showed that with the 5-breakwater alternative, the shoreline position for the entire western end of Grand Isle is seaward of the future without project shoreline and the alternative has no negative impact when compared to the future without project condition; no increase in erosion was observed. On the contrary, the 10-breakwater field resulted in significant downdrift erosion with respect to the future without project. At year 5, for the 5-breakwater field, the beach is at or seaward of the initial shoreline position for the area of interest; however, the 10-breakwater field shows nearly 75 ft of shoreline retreat downdrift of the end of the breakwater field, with downdrift erosion extending for nearly three quarters of a mile. After 10 years, both the 5 and 10-breakwater alternatives still retain beach fill seaward of the initial shoreline for much of the breakwater field, but the downdrift effects of the 10-breakwater field increase erosion by 35 ft.

Three potential borrow sites at Caminada Pass were defined and their impacts on Caminada inlet processes were evaluated. Two alternatives were located on the western lobe of the Caminada Pass ebb shoal (Pits A and B) and a third located eastern lobe (Pit C). The borrow source impacts on Caminada Pass inlet processes were evaluated using the Delft3D model.

The model results suggest that Pit C located on the eastern lobe of the ebb shoal will result in the greatest increases in scour and shoreline erosion in the interior of the inlet but the smallest increases in nearshore erosion on the Gulf-front shorelines on either side of the inlet, however, it will also reduce the sand transported to the Gulf Shoreline through bypassing by nearly 18%, while Pits A and B have negligible changes to overall bypassing.

Generally, the model results indicate that the impacts of dredging the ebb shoal may be mild to moderate and the Caminada Pass is a feasible borrow source that should be considered. We recommend that a more detailed geotechnical investigation be performed to develop a better understanding of the sand body geometry, and then develop a more precise borrow site for further evaluation.

Further evaluation of the project geometry may be warranted based on expected project budget for construction and cost estimates for the breakwater construction and beach nourishment after more refined design. In addition, we recommend additional effort to minimize downdrift impacts. This may include variation of the beach nourishment template to achieve a smooth transition between the breakwater field and the end of the revetment based on available funds.

In addition, downdrift erosion at the end of breakwater field and existing revetment is expected for any number of the breakwaters. We recommend consideration of additional sand placed as beach and/or dune nourishment to further reduce downdrift erosion at the transition between the end of the breakwater field and the end of the existing revetment.

1 Project Background

1.1 Introduction

This report discusses work completed under CPRA Contract Number 4400012419, Task Order 3, for Grand Isle and Vicinity – Breakwater Design. The purpose of the overall Grand Isle Levee Dune Beach Stabilization and Beach Nourishment Project is to work with the project partners to develop a design that stabilizes the western end of Grand Isle, protecting the Levee Dune and landward infrastructure, while maintaining a recreational beach. The project is needed to address the recent gulf shoreline erosion and diminished protection against storm surge. The project should not interfere with the downdrift shoreline or disrupt longshore transport from the Caminada Headlands to Grand Isle.

The goals of this project are to evaluate the location and length of the breakwater field installed seaward of a proposed beach nourishment on the southwestern end of the Island so that the proposed breakwater field does not interfere with the natural bypass of sand from the Caminada Headland onto Grand Isle, and to evaluate the impact of a proposed sand borrow site in the Caminada Pass ebb shoal on the Grand Isle shoreline.

Work on this project included developing an understanding of coastal processes, creation of a sediment transport model at the Caminada Pass inlet and western end of Grand Isle and to evaluate proposed alternatives on project performance using the model. Finally, recommendations are developed on the number of breakwaters as well as the breakwater length, spacing, and distance offshore, along with the impacts of proposed borrow sites.

1.2 Project History

This section describes the Grand Isle project history, the previous coastal engineering analyses, and the current project goals. The existing understanding of coastal processes will be used as the basis for assessing the Caminada Pass coastal dynamic processes and the impacts the proposed alternatives have on the Grand Isle shoreline.

Grand Isle is located in Jefferson Parish, Louisiana (Figure 1). Grand Isle is the only inhabited barrier island in Louisiana. It is also part of a barrier island chain that separates Barataria Bay from the Gulf of Mexico. For more than 60 years, the Grand Isle shoreline has been subjected to multiple projects and hurricane events as shown on Figure 2. For a detailed history of the project site and a summary of projects executed along the project shoreline, please refer to Coast & Harbor Engineering (CHE, 2005).

Figure 1. Grand Isle project vicinity.

USACE projects GI-01, GI-01A, GI 01B, GI-01C, and GI-01D represent the Grand Isle and Vicinity Hurricane Protection Project and consist of a 7.5 mile vegetated sand dune extending the length of Grand Isle's gulf shoreline, a jetty to stabilize the western end of the island at Caminada Pass, an offshore breakwater system, and dune walkovers. The majority of the levee dune consists of a vegetated sand dune with a geotextile tube core with an anchor tube, a scour apron, and a sand cap (see Figure 3 and Figure 4).

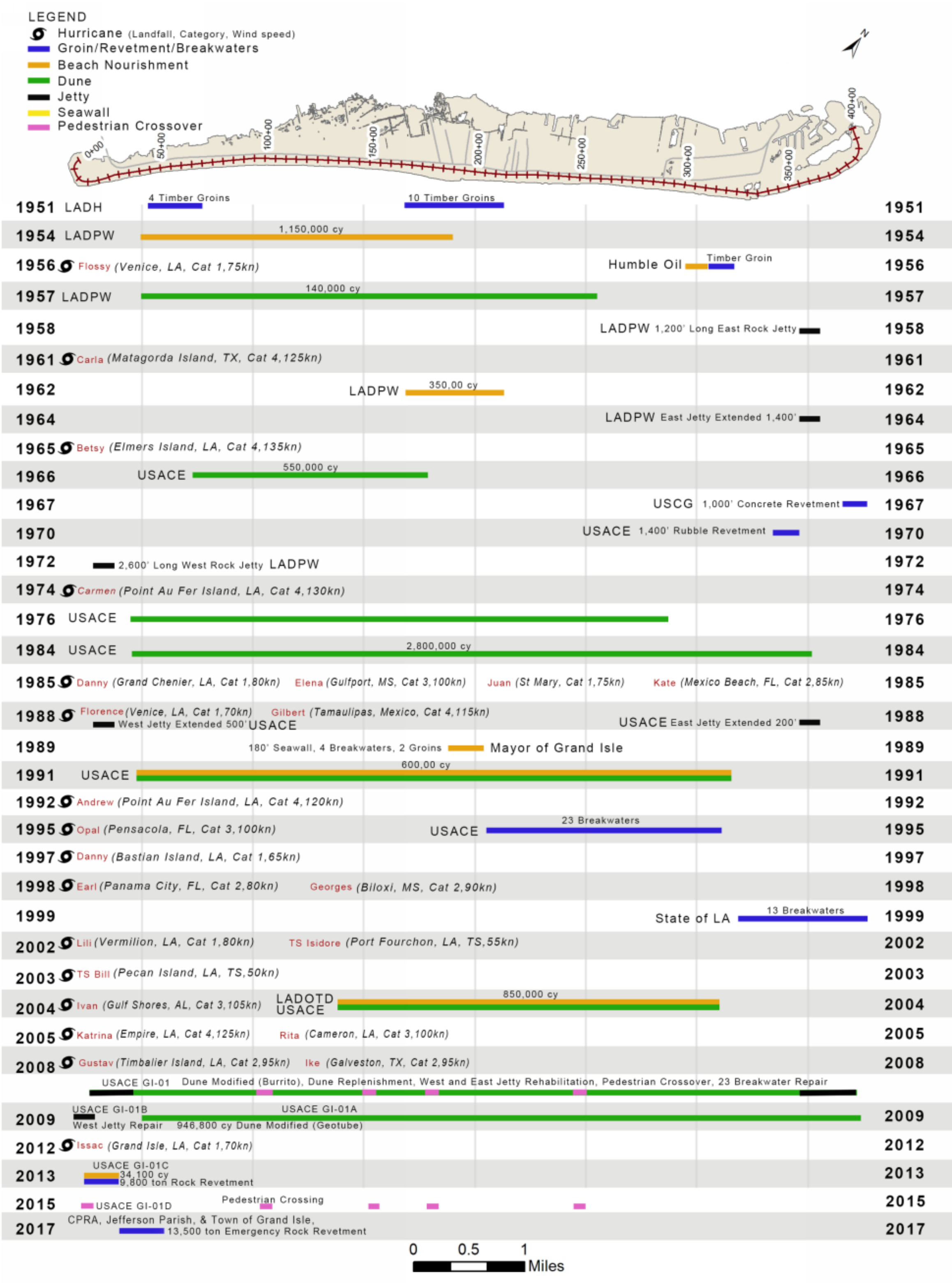

Figure 2. Engineering projects and hurricane history at Grand Isle, LA.

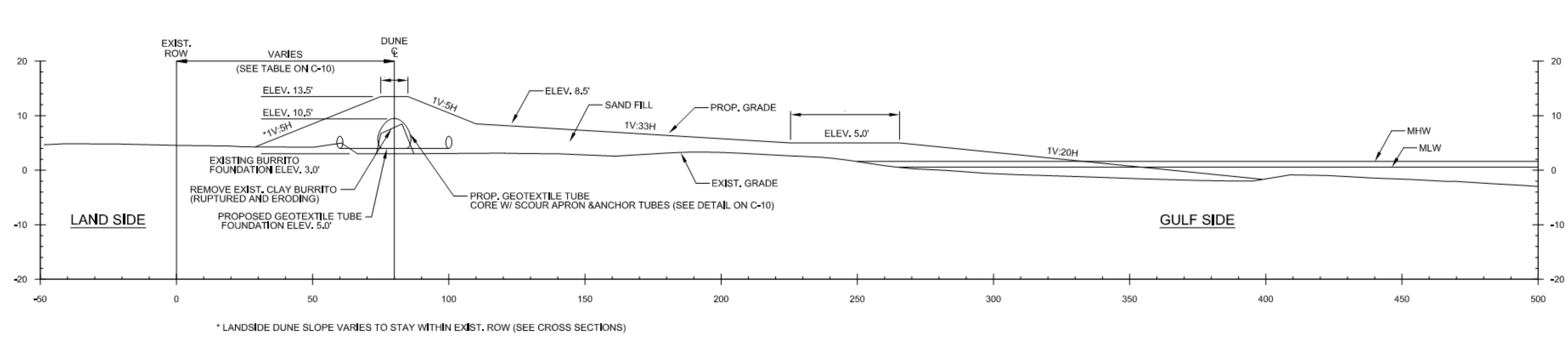

Figure 3. GI-01A Template. Taken from Grand Isle and Vicinity Hurricane Protection Project Station 0+00 to 386+00 along Grand Isle Beach. Rehabilitation of Hurricane Gustav and Hurricane Ike Damage drawings, dated February 2009.

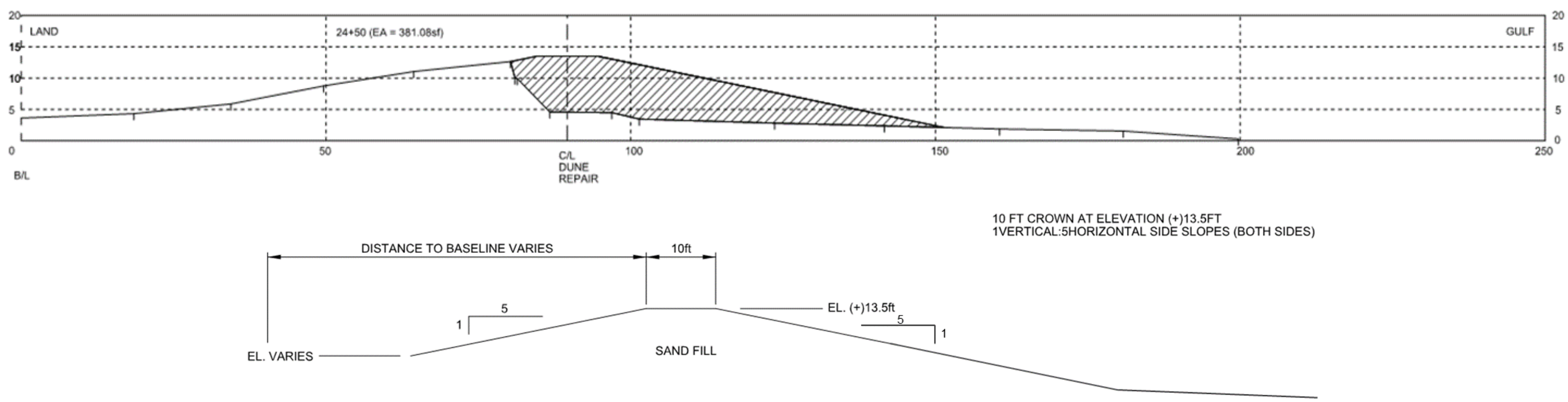

Figure 4. GI-01C Template (top) and detail (bottom). Taken from Grand Isle and Vicinity Hurricane Protection Project Dune Repair and Armoring drawings, dated April 2013.

In recent years, the southwest part of the island has been subjected to severe erosion even though no major hurricanes have impacted the area. In March 2016 a weather event occurred on Grand Isle and produced sustained southerly winds which generated increased wave action on the Grand Isle beach and dune. As a result, the beach and dune sustained heavy erosion on the southwest end scarping the dune and exposing the geotextile tube and scour apron/tube in some areas.

This section of the sand dune was previously subject to GI-01C Dune Repair and Armoring (Stations 0+00 to 30+00) in 2013 to repair damages from Hurricane Isaac; a portion of the latter section had been previously repaired after Hurricanes Katrina and Gustav (GI-01 and GI-01a). With subsequent storms in 2017, the dune erosion, scarping, and exposing of the scour apron/geotube severely progressed northwards of station 30+00. Field pictures shown in Figure 5 illustrate the erosion on the west side of the island.

Due to severe erosion and threat to the existing geotube dune core, emergency repairs were conducted during Spring and Summer of 2017 to reinforce the dune. In September 2017 rock armoring was installed along the southwest portion of the dune from the west end jetty to station 51+00. The extent of revetment is shown in Figure 6.

Figure 5. Field pictures show the increase in erosion on western end of Grand Isle. Pictures taken from west walkover looking toward the west jetty in June 2016 (top) and April (2017).

Figure 6. Extent of west revetment shown in gray along Grand Isle baseline.

1.3 Previous Coastal Engineering Analysis

In May 2017 Mott MacDonald completed a Coastal Processes Analysis and Alternatives Development for CPRA, referred to as Phase 1 of the project. The objective of the study was to understand the causes of the erosion in the southwest end of the island, ultimately proposing four stabilization alternatives. For a detailed coastal engineering analysis, assessment of the Grand Isle Federal Levee Project, alternative development, and alternative analysis, please refer to Mott MacDonald (2017).

A numerical SWAN wave model was used to transform the waves from offshore to nearshore to determine the longshore transport along the project shoreline and to drive the Gencade shoreline morphology model, which formed the basis of a sediment budget along the shoreline. Wave modeling indicated that the Caminada Pass ebb shoal modifies the wave transformation near the west end of the island, resulting in a nodal point (divergent node), with a localized sediment transport reversal at the west end of Grand Isle in spite of the overall net sediment transport towards the northeast.

This divergent node results in an erosional hot spot which has led to severe erosion at that nodal point and localized accretion on the West Jetty. A half mile east of the jetty, sediment bypassing across Caminada Pass attaches to the shoreline. Along the middle section of Grand Isle, (between 2-3.5 miles from western jetty) a relatively uniform sediment transport rate results in a shoreline that is relatively stable. Along the eastern end of the island, a decreasing sediment transport rate, likely due to the presence of offshore breakwaters, results in shoreline accretion.

Shoreline change analysis confirmed the presence of the erosional hot spot. The analysis showed that prior to the construction of the rock revetment, the erosional hot spot lied around 0.3-0.4 miles east of West Jetty. After the construction of rock revetment in 2013, the erosional hot spot has shifted downdrift of the revetment (0.3-0.6 miles east of West Jetty), and as shown in Figure 5, the erosion continued extending downdrift.

The erosional hotspot present along the western end of the Grand Isle shoreline has impacted the Federal projects with erosion rates higher than the planned maintenance rate. The GI-01C project (revetment) was successful in protecting the Levee Dune in its immediate lee but does not alleviate erosion adjacent to the structure. Several alternatives were proposed to alleviate the erosion in the western end.

The alternatives were designed to provide sand at some maintenance interval, add structures to retain the sand at the site, or some combination to achieve a stable shoreline for the lowest project life cost. The best performing alternative, shown in Figure 8 and Figure 9, was found to be a combination of offshore breakwaters, mitigation dune, and beach fill. The breakwater field of 5 breakwaters was analyzed at the conceptual design level; detailed breakwater field design was not part of the scope of work and was not provided by Mott MacDonald (2017).

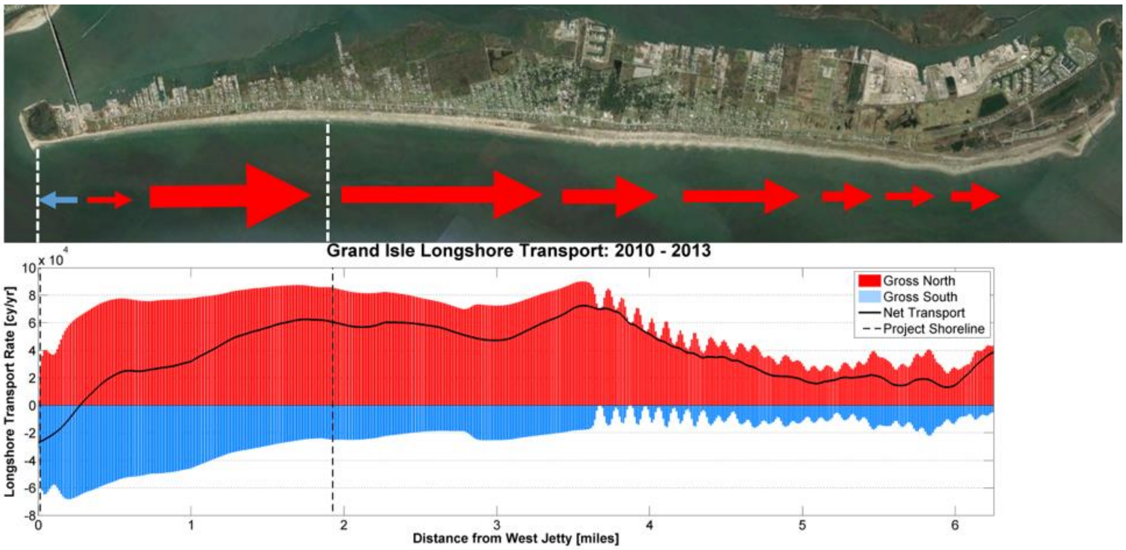

Figure 7. Computed LST rates from 2010 to 2013. Gross transport directed toward the southwest is shown with blue bars, gross transport directed toward the northeast is shown in red bars, and the thick black line shows the net longshore transport rate.

Figure 8. Preferred alternative site plan: Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill.

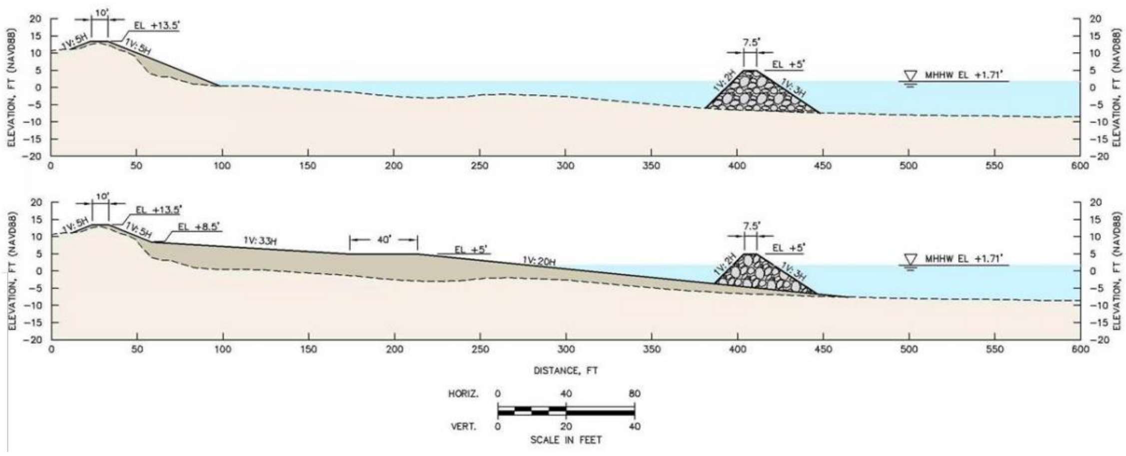

Figure 9. GI-01C 2013 Template Replaced (top) and preferred alternative cross-section Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill (bottom).

1.4 Project Goals

Project Stakeholders advocated to build breakwaters along much of the western end of the project shoreline. However, the sediment bypassing Caminada Pass may be modified by a large breakwater field in a way detrimental to Grand Isle shoreline stability. As a result, Mott MacDonald has been tasked to design a breakwater field that does not interfere with the Caminada Pass shoal dynamics particularly the natural sediment bypassing. The goal of this Grand Isle and Vicinity Breakwater Design study is twofold:

1. Optimize the location and length of the breakwater field installed seaward of a proposed beach nourishment on the southwestern end of the Island so that the proposed breakwater field does not interfere with the natural bypass of sand from the Caminada Headland onto Grand Isle.
2. Evaluate the impacts dredge borrow pits located on the Caminada Pass shoal would have on the Grand Isle shoreline.

2 Data Review

This section describes the existing physical characteristics at the site and presents existing and new data that has been collected in order to inform the project. Existing hydrodynamic and bathymetric data have been mostly referenced from Grand Isle Levee Dune and Beach Stabilization and Beach Nourishment Project Report; where necesseray, additional existing data have been reviewed and compiled. This section covers water surface elvations, wind, waves, bathymetry, and sediments size data descriptions.

2.1 Statistical and Extreme Value Analysis

Statistical and extreme value analyses of waves, winds, and water levels were conducted in Mott MacDonald, 2017 to develop an understanding of coastal processes and how they impact the project shoreline. Relevant data were collected from available sources (WIS, NHC, and NOAA) near the project site and are shown in Figure 10. Since no major storm has impacted Grand Isle from 2017 to 2019, it is reasonable to assume that the statistical analyses are still valid. Thus, the results shown in this section are referenced from Mott MacDonald (2017).

Figure 10. Data sources and locations used for coastal processes analysis.

2.1.1 Tidal Datums and Water Levels

Tidal datums were referenced from Mott MacDonald (2017) which were obtained from the NOAA Station 8761724, Grand Isle (NOAA, 2015) located within the project vicinity referenced to the 2007-2011 tidal epoch; these elevations are shown in Table 1. New bathymetric survey collected by the USACE recorded changes in tidal datums; however, the tidal and vertical datums from the 2016 and 2019 have differences that have not been able to be reconciled And therefore, the work presented in this report used the tidal datums shown in Table 1 throughout this analysis. In general, the tide range is low, with a spring tide range of 1.1 feet.

Table 1. Tidal datums at location near the project site at NOAA station 8761724 Grand Isle based on the 2007-2011 epoch.

Water Surface Elevation	[ft NAVD88]
Mean Higher High Water (MHHW)	1.71
Mean high Water (MHW)	1.70
Mean Sea Level (MSL)	1.19
Mean Lower Hater (MLW)	0.66
Mean Lower Low Water (MLLW)	0.65

Extreme value analysis was conducted for NOAA station 8761724 water level data only up to the 25-year return period due to the unreliability of the instrument to record higher water levels during storms. For return periods higher than 50-year, extreme value water levels were based on a previous study (Resio, 2007). The extreme water levels are provided in Table 2 (Mott MacDonald, 2017).

Table 2. Extreme surge plus tide (storm tide) near the project site.

Return Period [yr]	Storm Tide [ft NAVD 88]
2	4.0
5	4.6
10	5.0
20	5.3
25	5.4
50	8.8
75	9.8
100	10.7
500	13.7

2.1.2 Wind

Statistical and extreme value analyses for Grand Isle winds were performed using two different data sources: WIS station 73130 and National Hurricane Center (NHC) database.

WIS

Statistical analyses for Grand Isle winds was performed using wind data from Wave Information Studies (WIS) (USACE, 2010). As shown in Figure 11, a wind rose was developed using the historical WIS wind data from 1980 to 2014. The wind rose indicates a varied offshore wind distribution, with no predominant direction. The highest wind speeds are observed coming from the northeast and northwest directions; such wind speeds are associated with strong winter cold fronts. For winds coming from onshore directions, more energetic winds come from south-southeast to south direction compared to the east-southeast to east-northeast directions (Mott MacDonald, 2017).

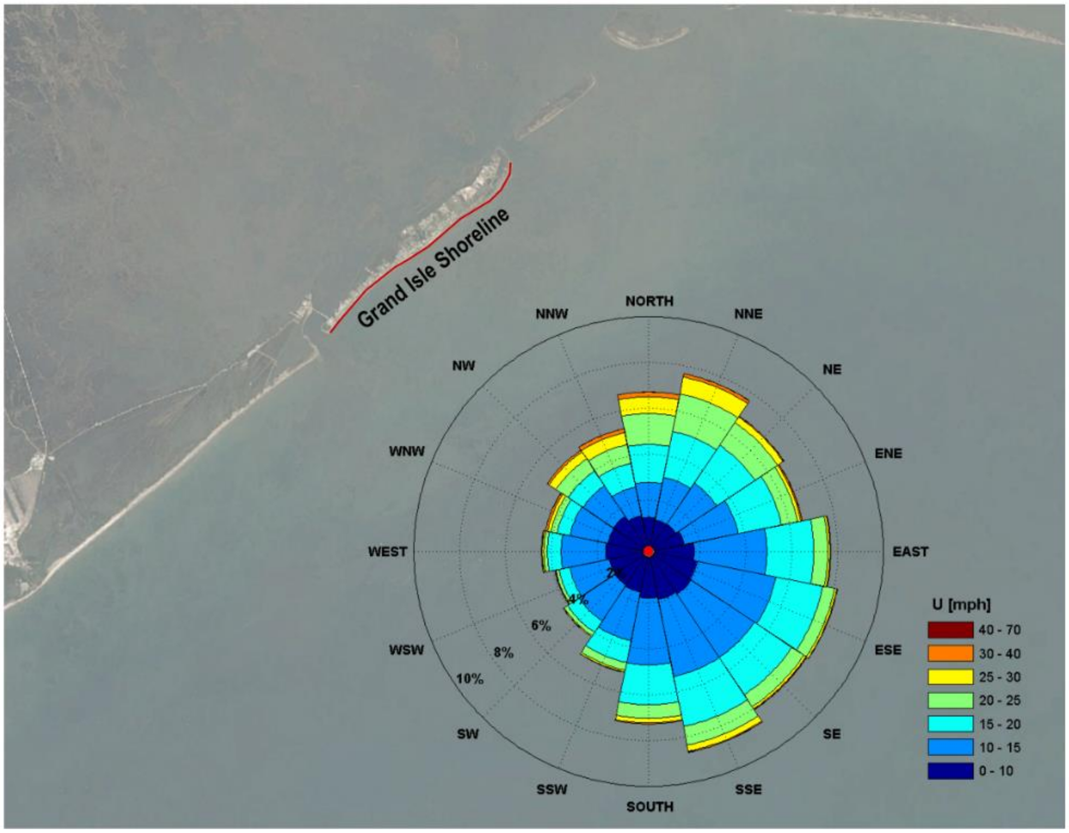

Figure 11. Grand Isle wind rose from WIS station 73130.

National Hurricane Center (NHC)

Mott MacDonald (2017) performed an extreme value analysis on all hurricanes influencing the project site using the National Hurricane Center (NHC) database from 1842 to 2014. Maximum wind speeds were extracted for all storms passing within 75 nautical miles of the project site during the data record (total of 86 storms). An extreme value distribution was fit to these maximum wind speeds and the results are shown in Table 3.

Table 3. Extreme wind speeds near Grand Isle based on NHC data.

Return Period [yr]	Wind Speed [mph] 2-min averaging	Wind Speed [mph] 10-min averaging
5	93.7	84.1
10	118.7	97.7
15	131.8	106.6
20	140.7	118.4
25	147.3	126.4
50	166.9	132.3
75	177.8	150.0
100	185.3	159.8

2.1.3 Waves

Similar to WIS wind analysis, a wave rose from WIS Station 73130 was developed and is shown in Figure 12. The predominant offshore wave direction is southeast to south-southeast. Similar to the wind rose, the wave rose also shows a more energetic environment from south-southeast to south compared to east-southeast to east directions. The time series WIS wave data was analyzed to produce the extreme value wave statistics presented in Table 4 (Mott MacDonald, 2017).

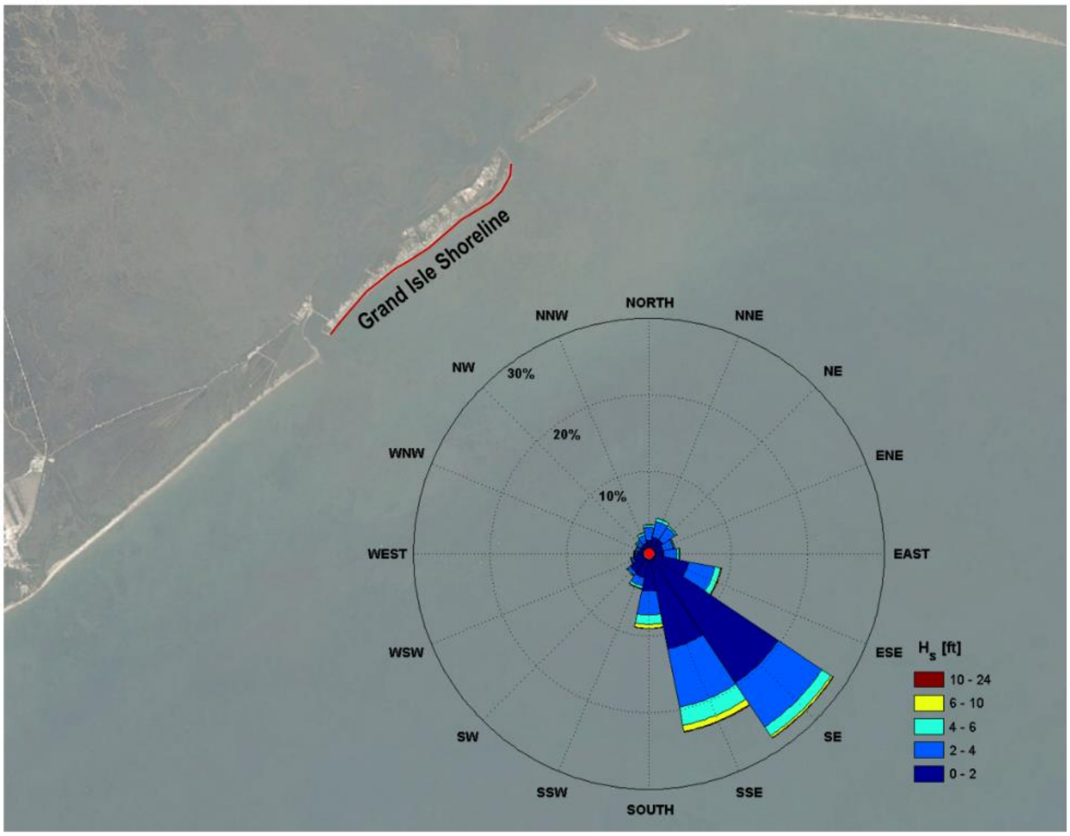

Figure 12. Grand Isle wave from WIS station 73130.

Table 4. Extreme wave heights and periods from WIS station 73130.

Return Period [yrs]	Hs [ft]	Tp [sec]
1	10.7	9.1
2	14.2	10.6
5	18.3	12.3
10	21.2	13.5
25	24.8	15.0
50	27.4	16.1
100	30.0	17.2

2.2 Bathymetry Sources

A bathymetric surface model that covers a wide region is required for circulation, wave, and sediment transport modeling. The different bathymetric data sources used in this study are shown below.

- **2005:** bathymetric surface created by Grand Isle Barrier Shoreline Stabilization Study Task 2 - Summary of existing Data and New Field Data Collection Plan (CHE, 2005). It was comprised of 2005 field data supplemented with a range of other sources including Coastal Relief Model (CRM) data. Data extents shown in Figure 13.
- **2015**: hydrographic survey lines consisting of a point data set obtained from Barrier Island Comprehensive Monitoring (BICM) (CPRA, 2019) (CPRA, 2016) with November 2015 as the survey date. Data extents shown in Figure 14.
- **2016:** hydrographic survey transects consisting of a point data set collected by HydroTerra Technologies as part of the Mott MacDonald (2017) study with December 2016 as the survey date (Mott MacDonald, 2017). Data extents shown in Figure 15.
- **2017:** hydrographic survey transects downloaded from CIMS spatial viewer website (CPRA, 2019) with June 2017 as the survey date. Data extents shown in Figure 16.
- **2018:** two sets of hydrographic survey transects taken by the USACE with September 2018 and December 2018 as survey dates and one set of hydrographic survey transects consisting of a point data set collected by HydroTerra Technologies for CPRA. Data extents shown in Figure 16.

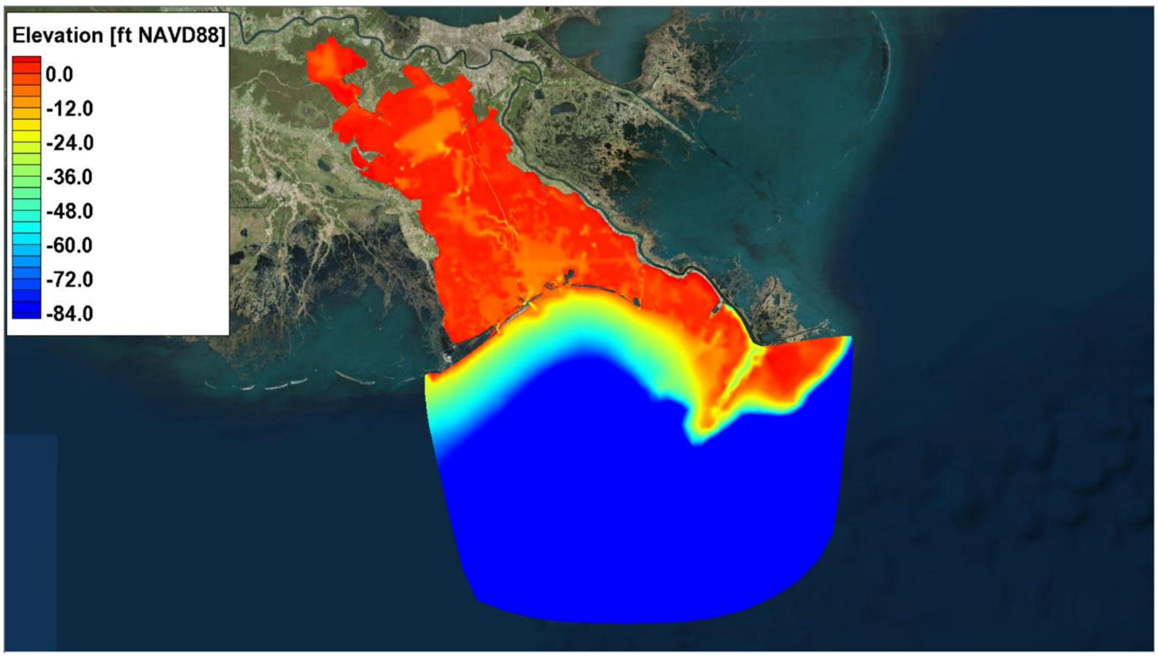

Figure 13. 2005 model bathymetry extents.

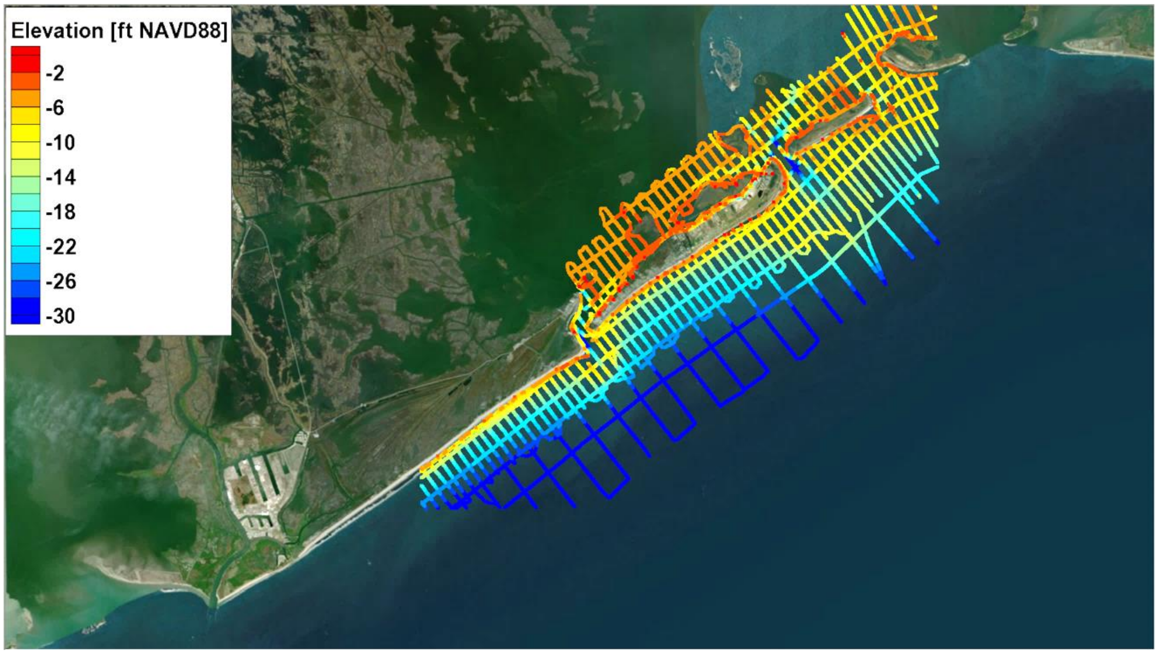

Figure 14. 2015 BICM survey extents.

Figure 15. 2016 survey extents from Mott MacDonald, 2017.

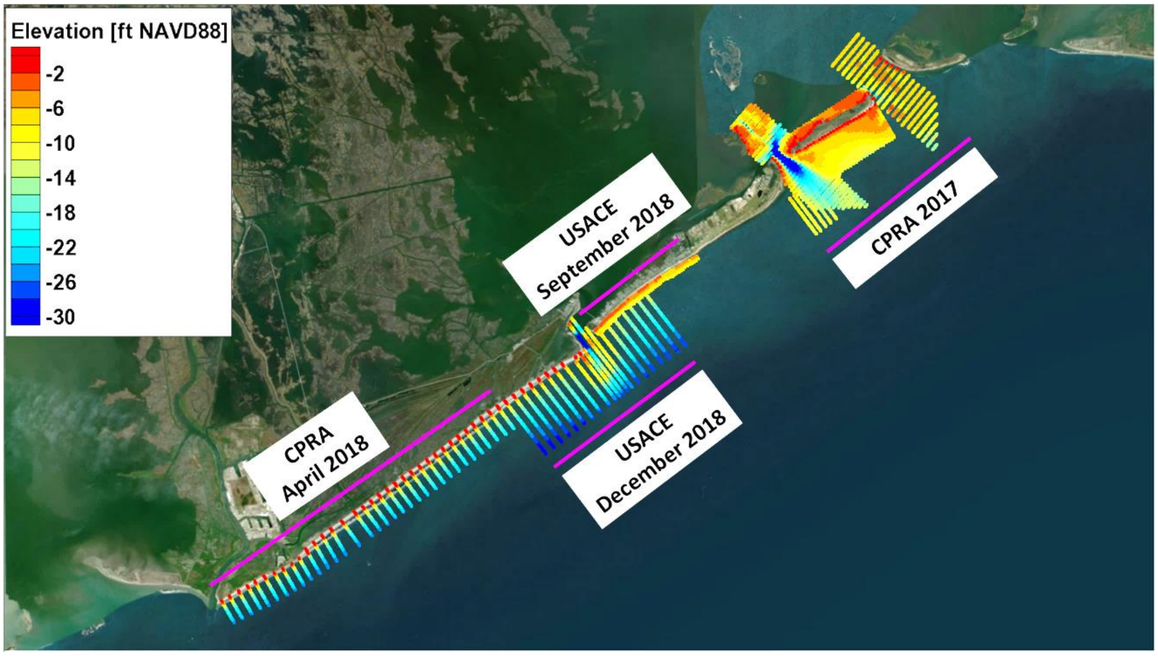

Figure 16. 2017 CPRA survey extents, 2018 CPRA survey extents, and 2018 USACE survey extents.

2.3 Sediment Size

Sediment size data at the project site, including Caminada Headlands, Caminada Pass ebb shoal, and Grand Isle, was obtained from three historic datasets. Three datasets include: BICM data for years 2008 and 2015 (CPRA, 2019) which span the entire sandy coast of Louisiana, and a 2017 data collection effort by UNO. These datasets/periods are substantial because they occur before, during, and after the Caminada Headland Beach and Dune Restoration Projects (Georgiou, et al., 2018).The existing data includes d_{50} and percent sand; available data is shown in Figure 17 and Figure 18.

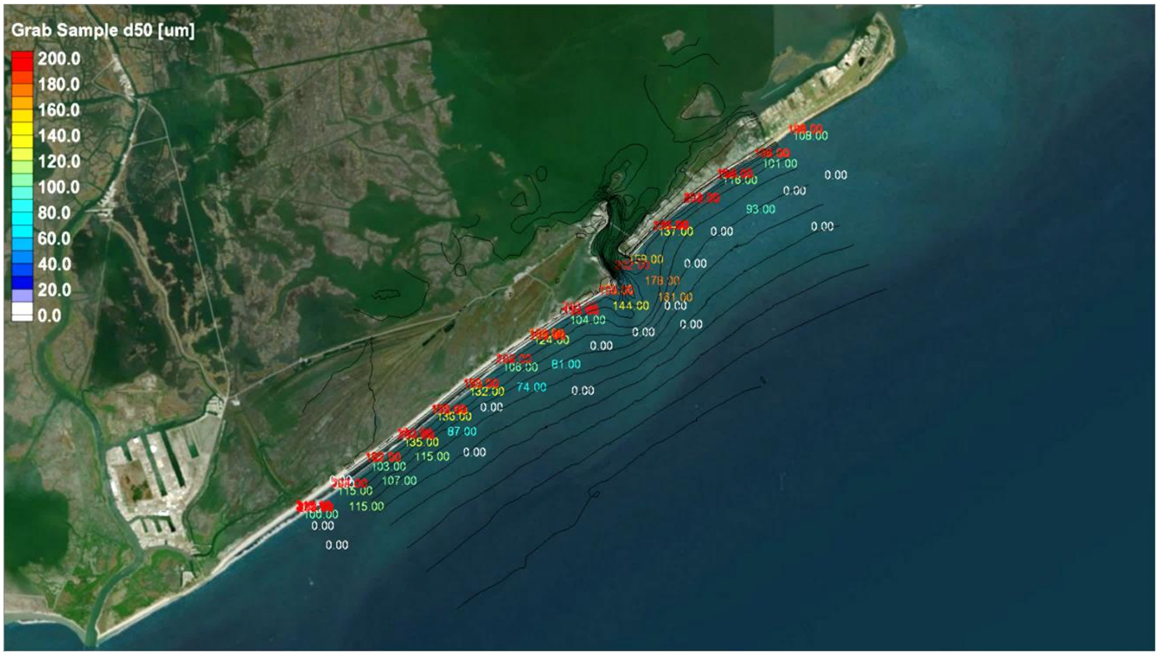

Figure 17. Existing sediment grain size data, d_{50}.

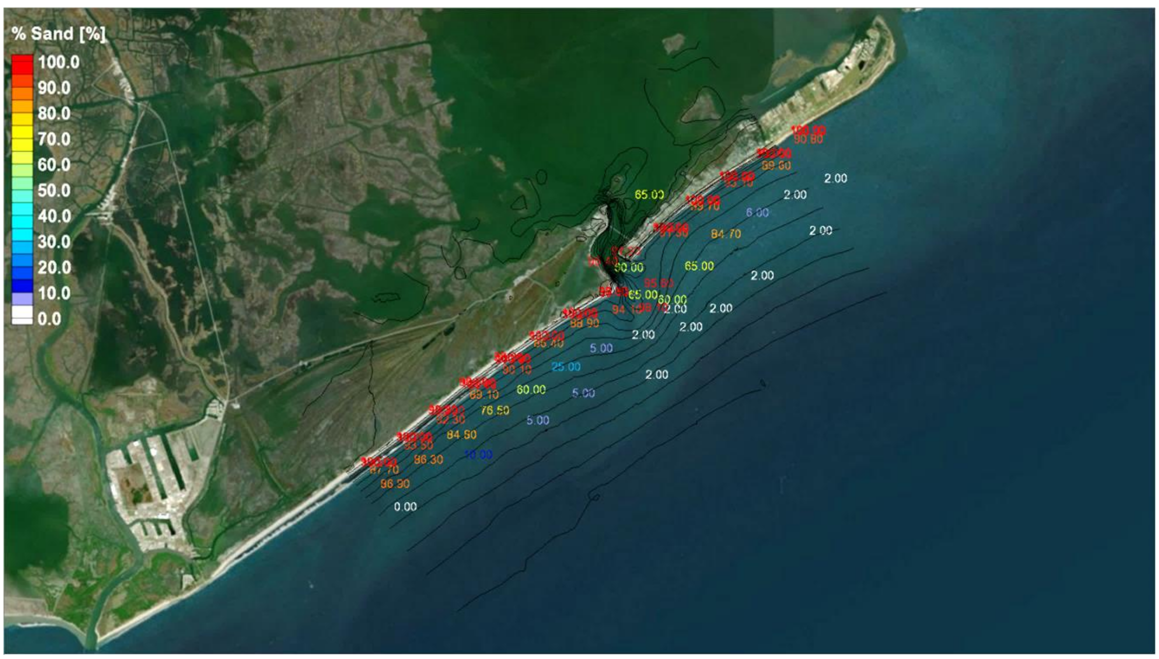

Figure 18. Existing sediment grain size data, percent sand.

3 Caminada Pass Inlet Processes

To better understand the Caminada Pass shoal dynamics and the associated sediment bypassing, tidal circulation, wave transformation, longshore currents, and longshore sediment transport were simulated using Delft3D, a morphological modeling package that couples a flow and sediment transport model with the Simulation Waves Nearshore (SWAN 40.72ABCDE) model. Delft3D can be applied in a wide variety of coastal environments, including complex geomorphological features such as Caminada Pass.

The intent of the numerical model is to simulate transport across the Caminada Pass ebb shoal and evaluate the impacts and benefits of implementing breakwaters and dredging the Caminada Pass ebb shoal on the pass dynamics and the project's shoreline. The effects of the breakwaters and dredge borrow pits are evaluated based on a relative comparison, i.e. without versus with project conditions. The intent of the model is not to robustly quantify all sediment transport and morphological process in Caminada Pass shoal and Grand Isle but to understand the effects the proposed alternatives have on the coastal dynamics, and therefore the model was established to provide a comparative analysis between various project configurations.

3.1 Numerical Model

3.1.1 Model Settings

Model Description

Numerical modeling was conducted using the process-based numerical model suite Delft3D. The model is composed of different modules that can compute the hydrodynamics, waves, sediment transport, and morphology. The base of the model is the hydrodynamic module, FLOW, that solves the unsteady shallow water equations in two (depth-averaged) or three dimensions (Lesser et al., 2004). This analysis employs the 2-dimensioanl (2D) model version.

Waves are simulated using the SWAN spectral wave model. SWAN is a 2D, spectral (phase-averaged) wave transformation model that can be used to generate wind-waves and transform offshore wave conditions to the nearshore project area (Delft University of Technology, 2012). The SWAN model was coupled with the Delft3D-FLOW – currents and updated bathymetry from the Delft3D-FLOW model are sent the SWAN model hourly, with wave information from the SWAN model sent back into the Delft3D-FLOW model to estimate longshore currents and sediment transport.

The sediment transport was calculated using the Van Rijn et al. (2000) sediment transport equation. The gradients on the transport rates are used to calculate the bed level changes. The bathymetry is then updated for the calculations in the next time step. Since morphological changes have much longer time scales compared to the hydrodynamics and transport processes, the bed changes are multiplied by a morphological time scale factor (MORFAC) to allow long-term, faster simulations (Lesser et al., 2004; Roelvink and Stive, 2006; Ranasinghe et al., 2011).

Grids and Bathymetry

The Delft3D domain setup is shown in Figure 19. The global model captures the overall interaction of the hydrodynamics and waves of Barataria Bay and the proper hydrodynamics at Caminada Pass but does not estimate sediment transport or morphological change. The global model curvilinear grid resolution at the project site in the alongshore direction consists of 330 ft. The global model grids and bathymetry are shown in Figure 20. The nested model captures the detailed Caminada Pass dynamics, including hydrodynamics, waves, sediment transport, and morphology, which affect the project shoreline. The nested model curvilinear grid resolution at the project site consist of 65 ft. The nested model grids and bathymetry are shown in Figure 21.

The global bathymetric surface was based on the 2005 Barataria Bay model surface shown in Figure 13 in combination with the 2015 BICM data shown in Figure 14. The nested bathymetric surface was mainly based on the 2015 BICM data. Thus, the bathymetric surfaces used throughout this analysis, shown in Figure 20 and Figure 21, are based on 2015 nearshore data. The numerical model was developed to show changes on the Caminada Pass ebb shoal and the Grand Isle shoreline resulting from the proposed project alternatives i.e. breakwaters and dredging; the purpose was not to quantify the associated changes. Therefore, as the datum conversion between the different surveys have not been reconciliated (see section 2.1.1), the 2015 bathymetric surface was used for all the modeling.

Boundary Conditions

The south (seaward) boundary condition of the global FLOW domain was prescribed as water levels. At the upcoast and downcoast boundaries of the global FLOW domain, Neumann conditions were applied, with longshore gradients in water levels and currents assumed to negligible. On the nested FLOW domain, the seaward and landward boundary conditions were prescribed as water levels extracted from the global FLOW domain. At the cross-shore boundaries of the nested FLOW domain Neumann conditions were applied.

The east, west, and south boundaries of the global WAVE domain were prescribed with significant wave height, wave period, and wave direction. The nested WAVE used the global WAVE domain results to define boundary conditions for the finer grid domain. The global and nested WAVE domains were also forced with wind speed and wind directions

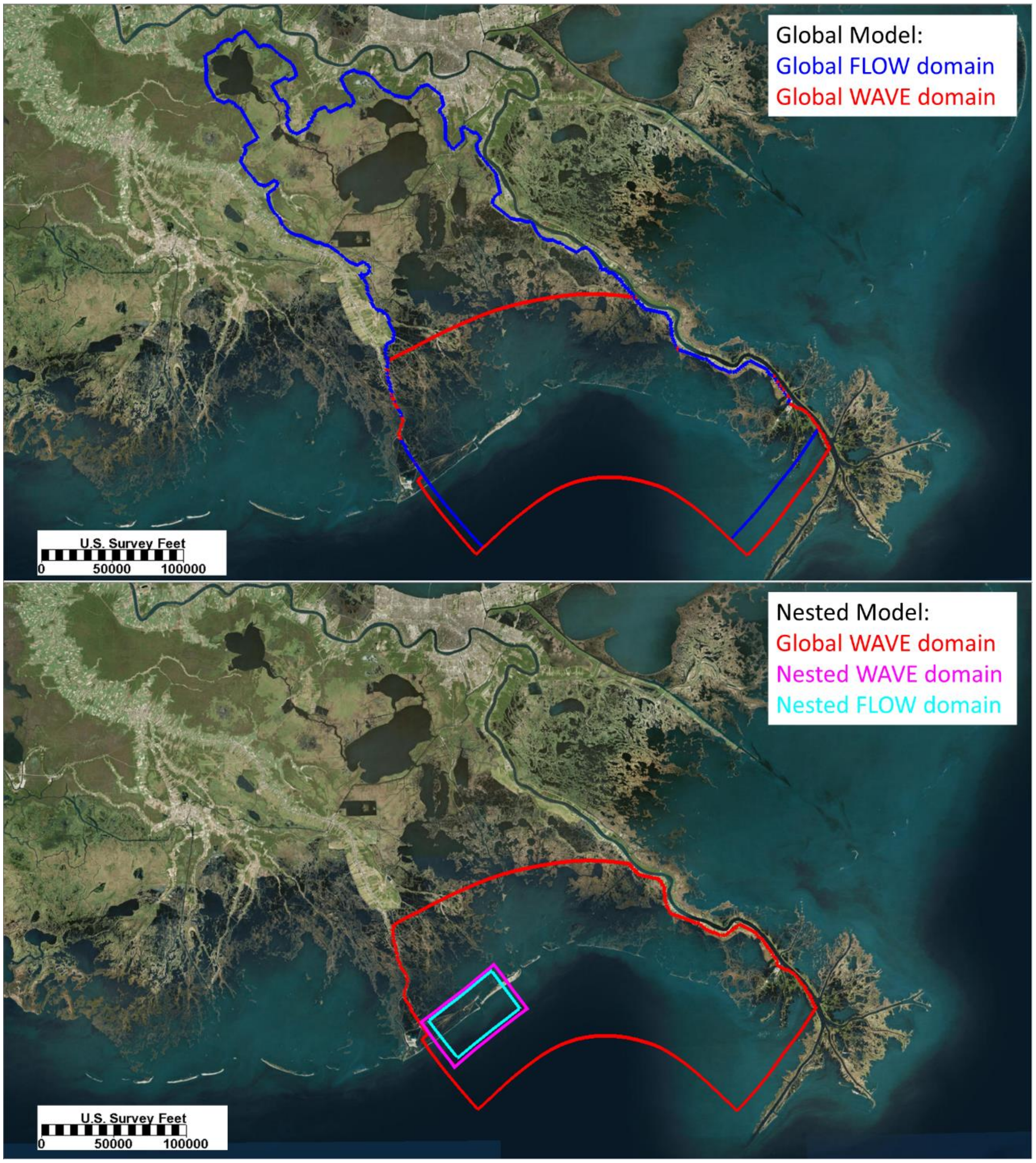

Figure 19. Delft3D model set up: top figure, Global Model consisting of global FLOW (blue) and global WAVE (red) domains; bottom figure, Nested Model consisting of global WAVE (red), nested WAVE domain (magenta), and nested FLOW (teal) domains.

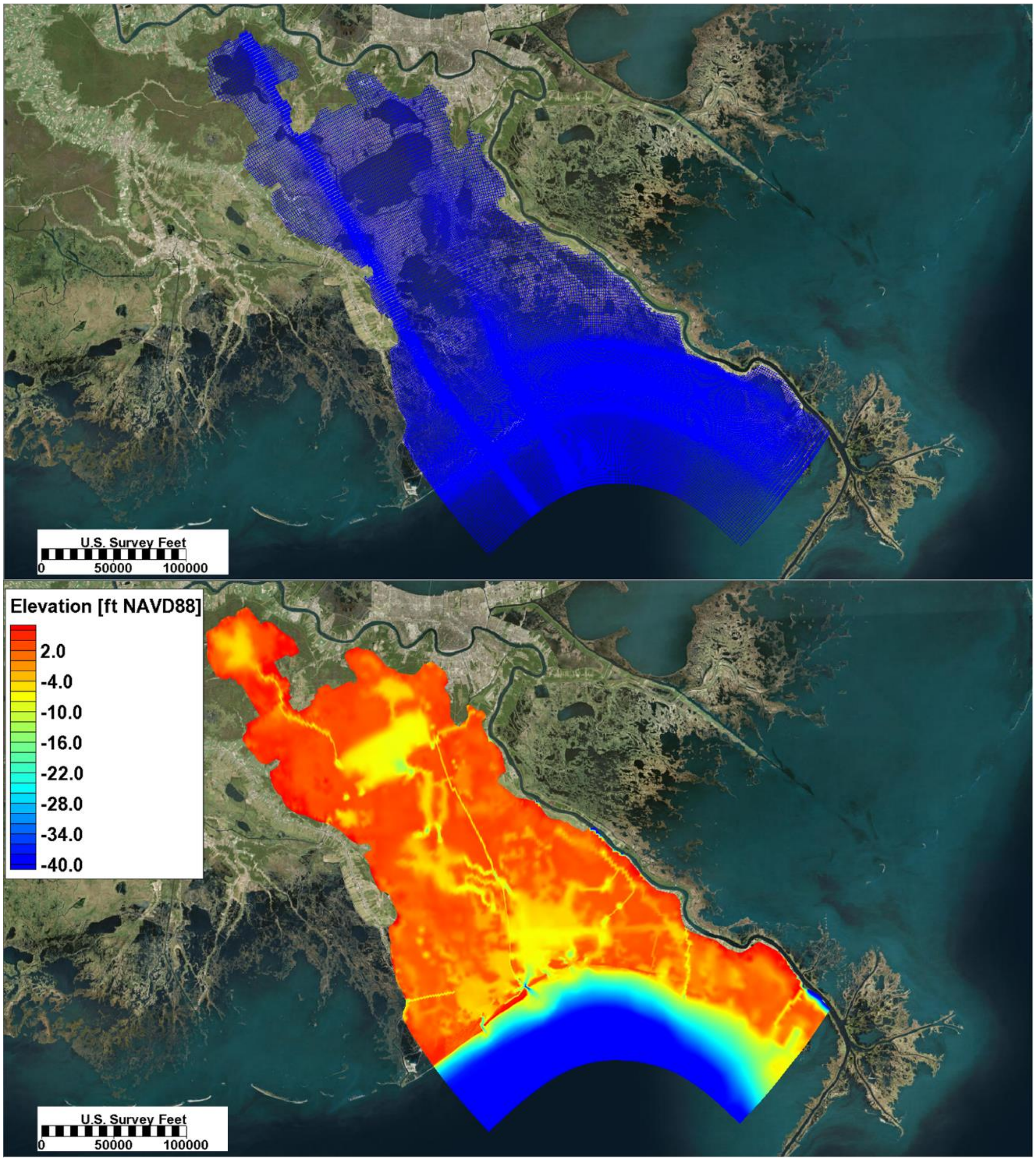

Figure 20. Global FLOW model grid (top) and associated 2015 surface (bottom).

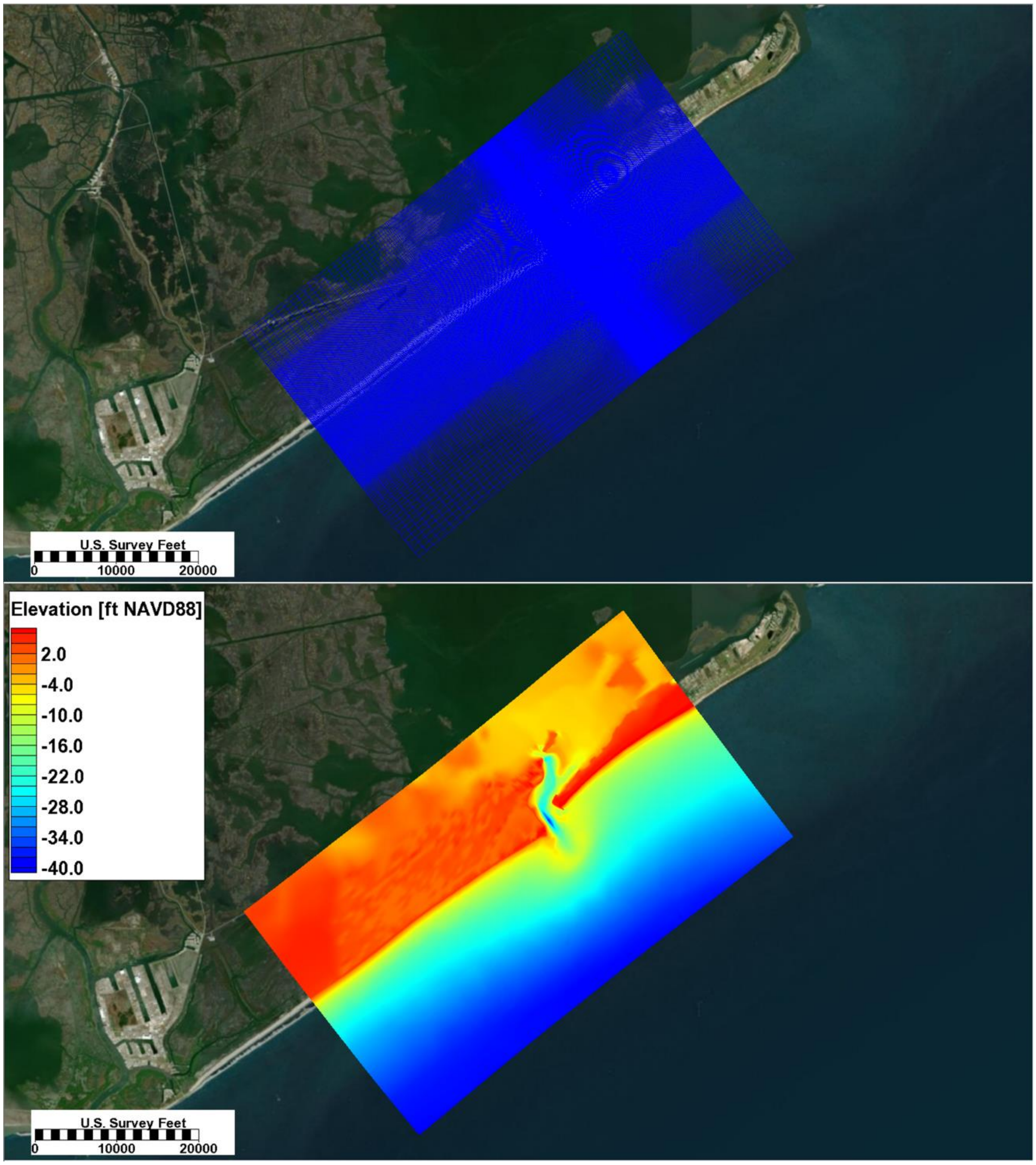

Figure 21. Nested FLOW model grid (top) and associated 2015 bathymetry (bottom).

3.1.2 Hydrodynamic Calibration

The model calibration process consists of comparing modeled velocities with measured data. For this project, the 2005 data including bathymetry and measured ADCP data was used for calibration. As shown in Figure 10, an ADCP gage was deployed on 8/10/05 at Caminada Pass and was recovered on 11/12/05. During this time period, Hurricanes Katrina and Rita passed nearby Grand Isle. The gage failed to record during the peak of the storm. Shortly after the storm passed, the CP gage began recording again, and continued to record until it was recovered (CHE, 2005).

The period selected for model calibration was 14 days prior to Hurricane Katrina, from 8/10/2005 00:00 UTC to 8/24/2006 00:00 UTC. The ADCP recorded water surface elevations were used as boundary conditions while the current velocities were used for calibrations. The model requires some spin-up time, as it starts from zero water level and zero velocities. Therefore, the model was started with an artificial ramp time. Initial model results with default model parameters showed poor agreement between modeled and measured data. Through a series of testing, the uniformly spaced Manning's n roughness coefficient was set at 0.015 for the hydrodynamic optimum calibration set up. The measured and calibrated model water surface elevations (WSE) and velocities are shown in Figure 22.

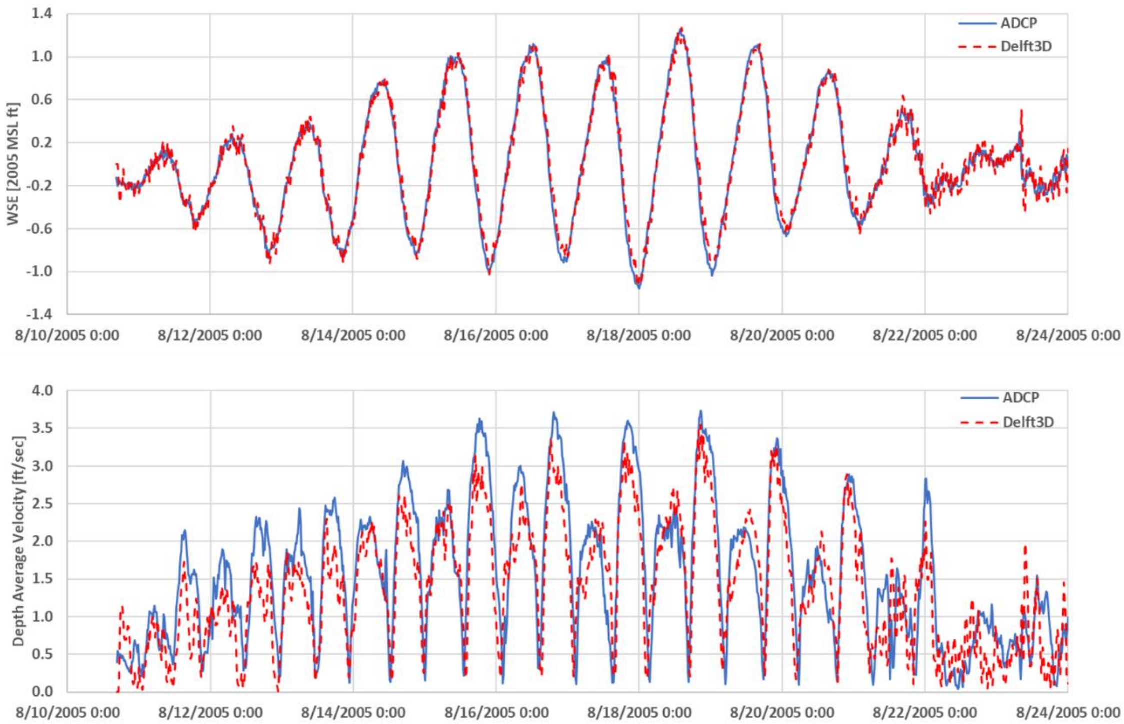

Figure 22. ADCP measured vs. Delft3D simulated water surface elevations (top) and velocities (bottom).

The WSE matches well in both magnitude and phase. The velocity is somewhat under-predicted during the first half of the simulation but matches well on the second half of the simulation. Based on these results, the model is considered reasonably calibrated for flows in the immediate vicinity of Grand Isle.

3.1.3 Environmental Forcing

Two- and three-dimensional sediment transport and morphology models are computationally intensive. Furthermore, while flows change on an hourly basis, the morphology changes occur on a scale of months to years. Thus, the Delft3D model was run for a shorter period of time, using a reduced number of wave cases to approximate the general wave climate during the period of interest. The number of wave cases are chosen to produce sediment transport patterns that would be similar to those based on the full time series of offshore waves (i.e.: Lesser, et al., 2004; Benedet and List, 2008; CPE, 2013).

The offshore wave climate during the calibration period was based on the time series of hindcast waves at WIS Station 73130 between June 1, 2015 and June 1, 2018. The primary wave cases were selected from the waves originating from the seaward direction bands (53° to 233°), which covered 63 percent of the wave record by time. These wave records were divided into wave height and direction classes, with each wave class containing an equal amount of wave energy. This method is known as the Energy Flux Method, which characterizes each wave record based the longshore energy flux. The longshore energy fluxes over the 3 year period were approximated using methods detailed in the Shore Protection Manual (USACE, 1984)."

Based on the energy estimates, the offshore waves were divided into 3 height classes with roughly equal amounts of wave energy. Each height class was then divided into 4 direction bands representing equal amounts of wave energy, for a total of 12 wave cases. To account for periods during which the offshore waves were propagating from the landward directions (233° to 360° and 0° to 54°), a 13th wave case was added, representing calm conditions.

Since higher, more energetic waves occurred less often than lower waves, the various wave cases did not represent an equal portion of the wave record with respect to time. To account for the percent occurrence of each wave case and the duration of the study period, a variable Morphological Acceleration Factor (MORFAC) was used as described in Lesser et al (2004) and Benedet and List (2008). The wave case distributions are shown in Figure 23 and Table 5.

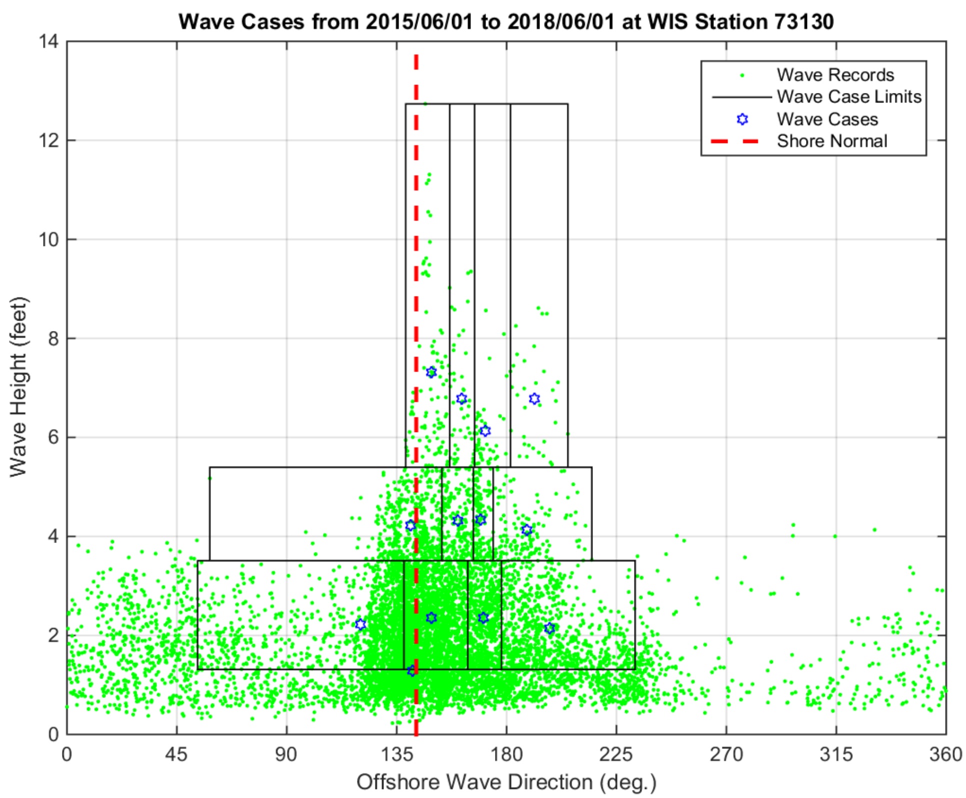

Figure 23. Wave cases based on WIS Station 73130 between June 1, 2015 and June 1, 2018.

Table 5. Wave cases and associated MORFAC based on WIS Station 73130 between June 1, 2015 and June 1, 2018.

Wave Class	Hs Range [ft]		Wave Dir Range [°TN]		Hs [ft]	Tp [sec]	Wave Dir. [°TN]	Wind [ft/sec]	Wind Dir. [°TN]	Hydrodynamic Time [hr]	MORFAC [-]	Morphological Time [hr]
1	1.31	3.51	54	138	2.22	5.3	120	17.43	73	144	26.1	3751.5
2	1.31	3.51	138	164	2.36	5.8	149	11.25	114	144	37.4	5383.5
3	1.31	3.51	164	178	2.36	5.2	171	9.60	154	144	12.3	1770
4	1.31	3.51	178	233	2.15	4.9	198	11.44	235	144	15.9	2283
5	3.51	5.40	59	154	4.22	7.0	141	21.16	115	72	18.2	1311
6	3.51	5.40	154	166	4.31	6.6	160	15.20	149	72	7.5	543
7	3.51	5.40	166	175	4.34	6.5	170	14.88	178	72	5.0	363
8	3.51	5.40	175	215	4.13	6.3	188	18.19	256	72	5.6	402
9	5.40	12.74	139	157	7.31	8.5	149	29.34	140	72	4.1	294
10	5.40	12.74	157	167	6.77	7.9	162	23.01	175	72	2.0	141
11	5.40	12.74	167	182	6.13	7.2	171	28.11	191	72	2.0	141
12	5.40	12.74	182	205	6.79	7.5	192	33.02	241	72	1.3	96
Calm	(Remaining waves)				1.29	4.0	142	11.24	11	144	68.2	9825

To avoid artificially biasing the morphological downscaling of hydrodynamic conditions toward certain tide conditions, the morphological acceleration factor was assigned to vary synchronously with tidal cycles (as shown in Figure 24). However, to capture a representative range of tidal forcing conditions over the three-year analysis period (June 2015 – June 2018), three tide ranges were selected from a histogram of historical tide range that represent a large proportion of the observable tides. The tides were input to the model as perfect sinusoid waves with a period of 24 hours.

The time series of reduced wave cases, water surface elevations, and MORFAC was used in the global model as environmental forcing conditions; the time series is shown in Figure 25.

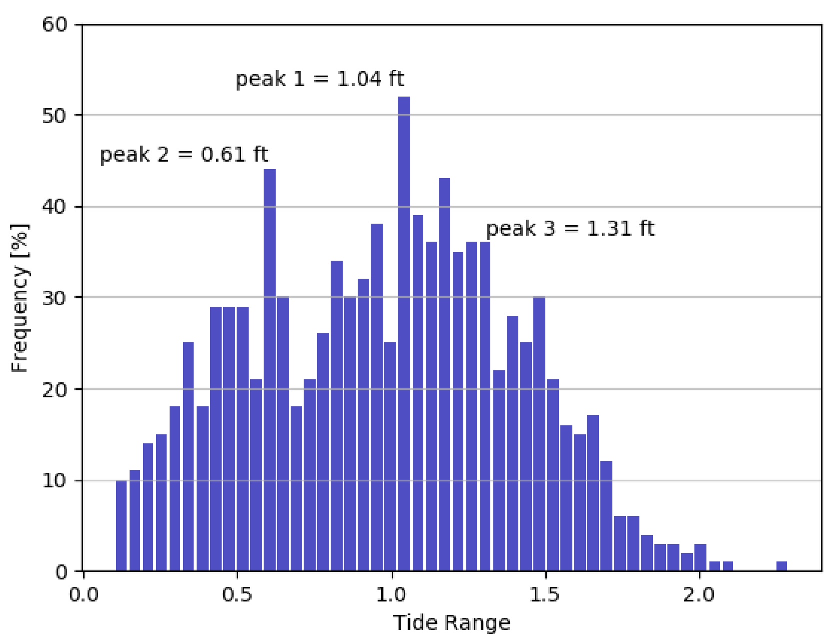

Figure 24. Distribution of tide range in feet at Grand Isle from 2015 to 2018.

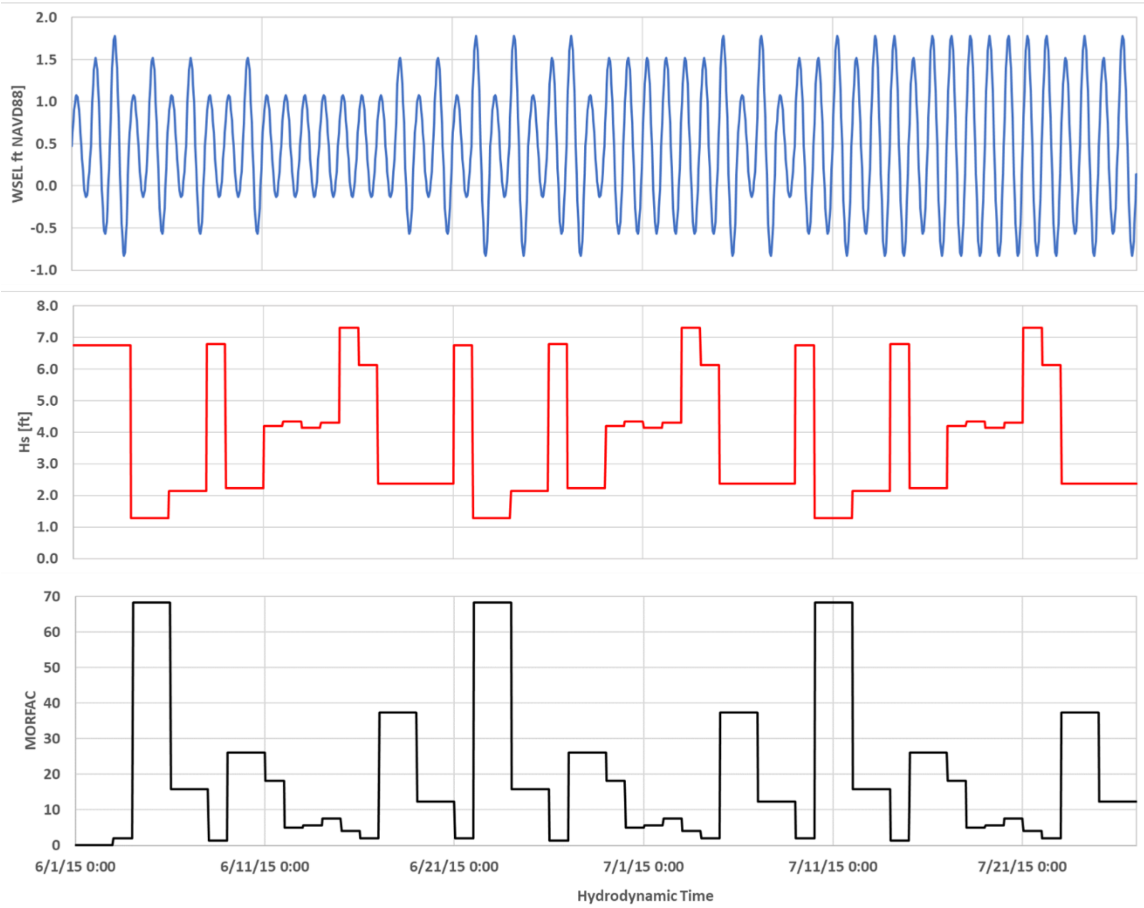

Figure 25. Time series of environmental forcing conditions: water surface elevations (top), wave height (middle), and MORFAC (bottom).

3.1.4 Sediment class distributions

Sediment transport rates are dependent on the sediment size. Based on the existing grain size data described in section 2.3, three different spatially varying sediment classes were employed in the nested FLOW module to calculate compute sediment transport patterns. Extensive sensitivity analysis was performed using different sediment sizes and the associated sediment class spatial distributions. The three sediment classes used in the analysis consisted of: (1) fine sand with d_{50} equal to 200 µm, (2) very fine sand with d_{50} equal to 100 µm, (3) mud. As shown in Figure 26, the grain size decreases form the nearshore to offshore.

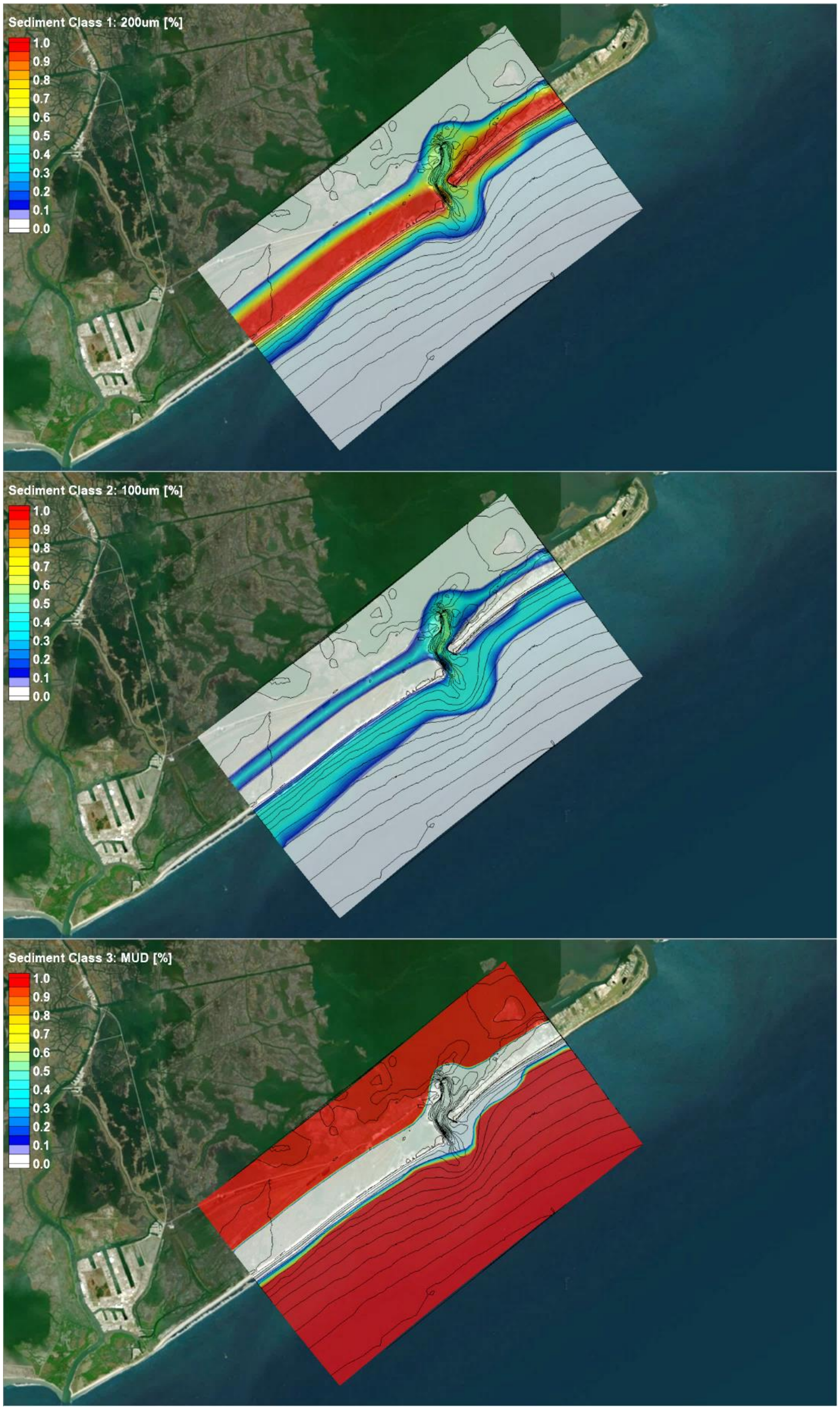

Figure 26. Three sediment classes used in the analysis: medium sand d50= 200 µm (top), fine sand d50 = 100 µm (middle), and mud (bottom). Scale is fraction of sediment where 1.0 = 100%.

3.2 Sediment Transport Results

The goal of the sediment transport analysis is to understand the sediment transport patterns along Caminada Pass and Grand Isle with an emphasis on sediment bypassing over the ebb shoal.

Figure 27 shows the sediment transport patterns along Caminada Pass ebb shoal based on the Delft3D model described in Section 3.1, using the 2015 bathymetric surface, and the reduced time series shown in Figure 25 as the environmental forcing conditions. Throughout the work presented in this report, the numerical analysis and its results are based on the 2015 bathymetric surface for consistency across the model (see Sections 2.2 and 3.1.1 for details).

The sediment transport vector field indicates a net transport field directed toward the northeast (from Elmer's Island to Grand Isle) with increasing sediment transport in the center portion of the island. Results illustrate the sediment bypassing from Elmer's island over the Caminada Pass ebb shoal onto Grand Isle. The analysis also indicates the presence of a nodal point on the western end of Grand Isle resulting in an erosional hot spot, which has been observed on the island since 2016.

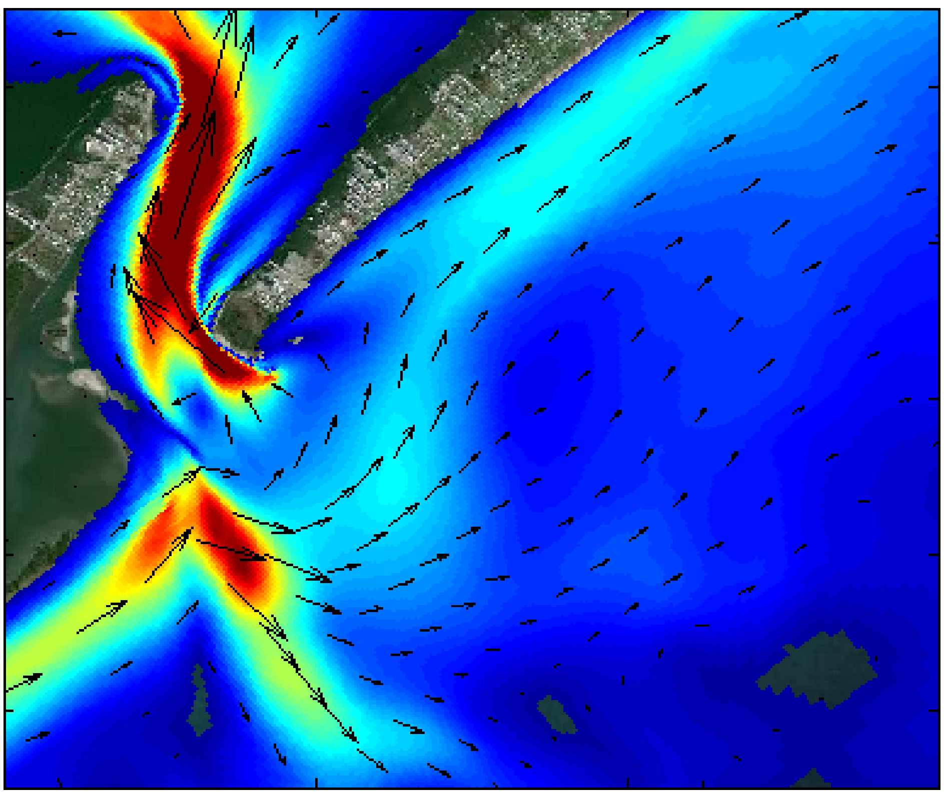

Figure 27. Sediment transport vector field over Caminada Pass ebb shoal.

The Delft3D model is known to not resolve the very shallow nearshore and therefore shoreline position with accuracy due to model limitations. Therefore, results from Delft3D are evaluated in terms of changes to the beach face, rather than the shoreline. The net total sediment transport or mean total transport rates were quantified using nearshore cross-shore transects. The mean total transport rates were calculated by integrating the incremental sediment transport compounded by the associated MORFAC value for all sediment classes (see section 3.1.4) and averaging over the morphological time. The results are shown in Figure 28.

The Deft3D model results agree with the shoreline change analysis, wave modeling, and sediment transport results by Mott MacDonald (2017). The nearshore cross-shore sediment transport analysis indicates the net longhore sediment transport is directed to the northeast at the Caminada Headlands and Grand Isle shorelines with one exception. A distance of 0 mi to 0.25 mi from the jetty (see Figure 28) the sediment transport is directed to the southwest indicative of an erosional hotspot; such results are in line with the Mott MacDonald, 2017 results summarized in section 1.3 and Figure 7. The erosional area extends between 0 mi to approximately 0.6 mi from the jetty where the Grand Isle shoreline becomes stable. It has been noted the 0.6 mi location matches the eastern end of the 2017 revetment at station 51+00 (see Figure 6). Figure 29 illustrates the realtionship between the GI-01A project stationing and cross-shore transects along the Grand Isle shoreline.

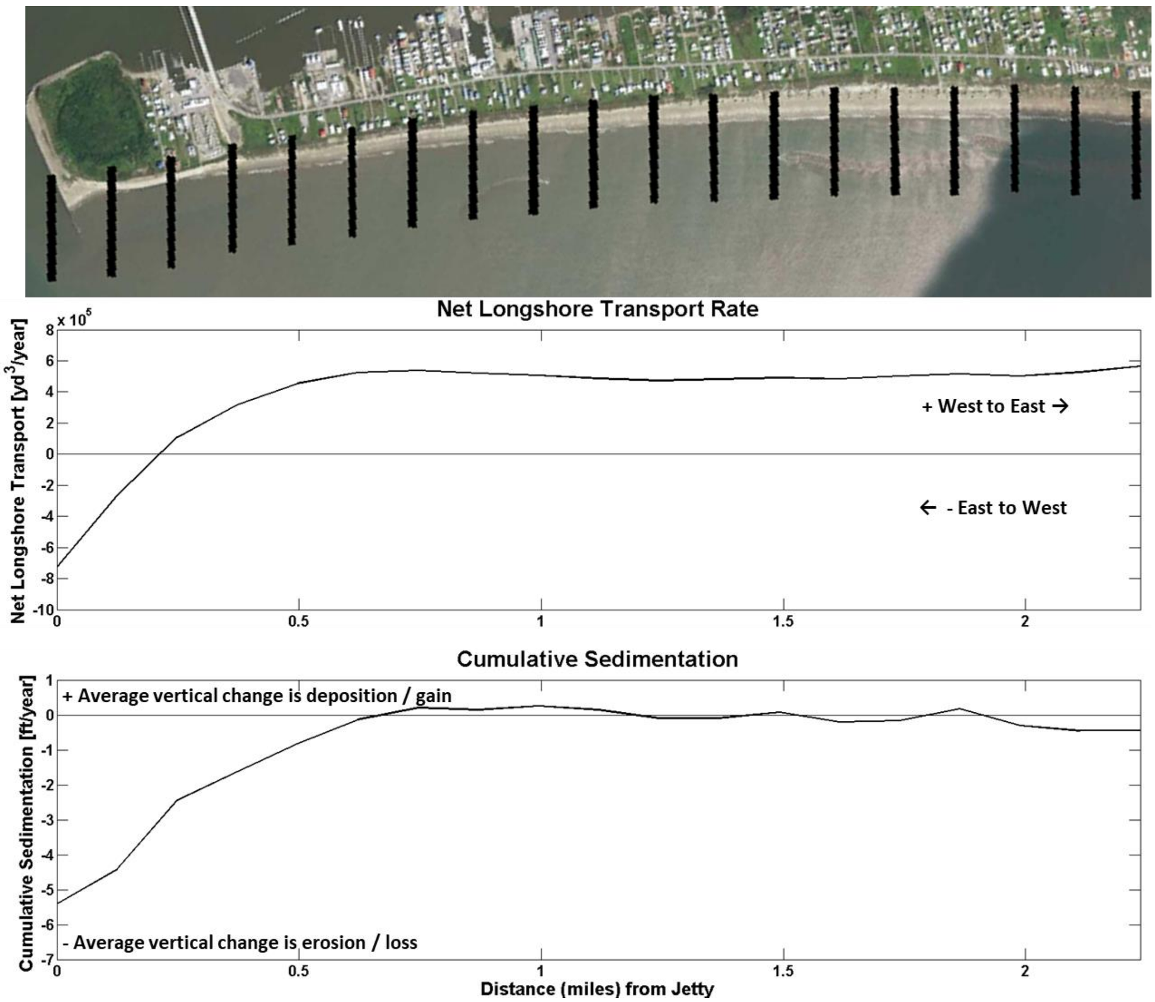

Figure 28. Top: cross-shore transects; middle: longshore transport (positive represents transport to the northeast/right and negative represent transport to the southwest/right); bottom: cumulative sedimentation (positive represents accretion and negative represents erosion).

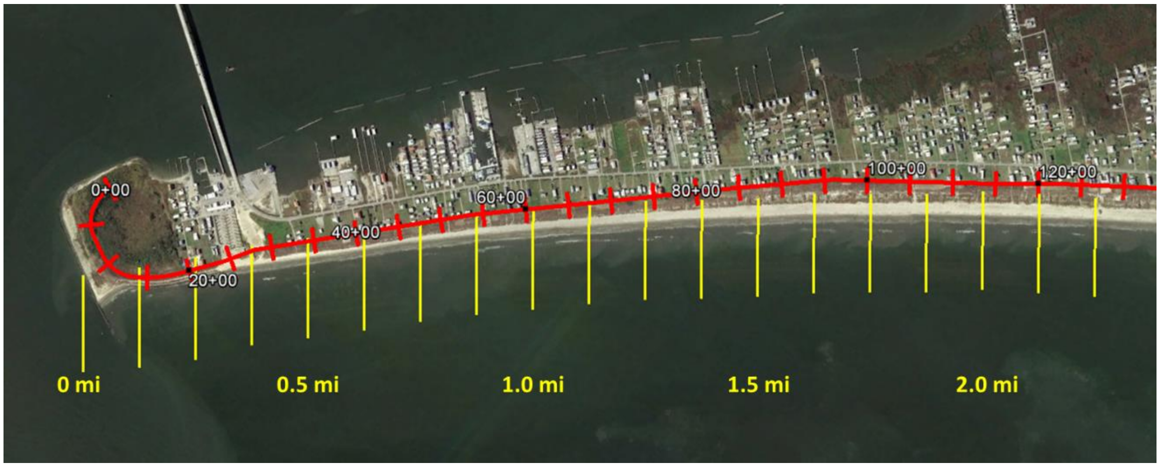

Figure 29. GI-01A project stationing shown in red and cross-shore transects shown in yellow with distance from the jetty on miles along the Grand Ilse shoreline.

The results presented in this section represent the existing conditions (without project conditions) and serve as the basis for the subsequent analysis. The alternative analyses and recommendations described in Sections 4, 5, and 6 are based in a comparison between the existing conditions shown in Figure 27 and Figure 28 and future with project conditions.

4 Project Impacts on Caminada Pass Inlet Processes

Previous analysis conducted by Mott MacDonald in 2017 indicate the Caminada Pass as highly dynamic system. The appropriate location and extent of the breakwater field is dependent on the sediment transport patterns across the Caminada Pass ebb shoal. The breakwater field should not block the natural sand bypassing. Also, the potential dredge pits on Caminada Pass ebb shoal should not cause a negative impact on the natural sand bypassing or the Grand Isle shoreline.

The numerical model described in Section 3.1 was employed to evaluate the impacts the proposed project alternatives have on the sand bypassing and the Grand Isle shoreline. The existing condition (or without project conditions) results illustrated in Section 3.2 are the basis of the comparative analysis. The breakwaters and dredge borrow pits impacts have been evaluated based on a relative comparison of existing conditions versus alternative project conditions. The intent of the model is to understand the impacts the proposed alternatives have on the coastal dynamics. Throughout the work presented in this report, the numerical analysis and its results are based on the 2015 bathymetric surface for consistency across the model (see Sections 2.2, 3.1.1, and 3.2 for details).

4.1 Breakwater Field

Two different breakwater fields were considered as alternatives: a five (5) and ten (10) breakwater fields. The two alternatives were implemented in the nested model domain described in 3.1. Five and ten breakwaters were implemented as thin dams in the nested FLOW domain and as obstacles in the nested WAVE domain; the breakwater implementation is shown in Figure 30. Each breakwater spans at 4 cells of the nested model grid in the alongshore direction.

Figure 31 shows the sediment transport vector fields and bypassing over Caminada Pass ebb shoal for the existing conditions, the 5-breakwater alternative, and the 10-breakwater alternative. In general, the model shows the breakwater fields do not have a negative impact on sediment bypassing. The 5-breakwater alternative does not reach the location where the sediment bypassing attaches onto the Grand Isle shoreline. On the other hand, the 10-breakwater alternative extends to the location where the sediment bypassing reaches the island which reduces the amount of sediment available to the nearshore.

Both breakwaters perform well in stabilizing the shoreline immediately in their lee. The breakwaters modify the nearshore longshore transport in their lee, resulting in erosion downdrift (east) of the breakwaters, similar to the pattern described by Bosboom and Stive (2013) as expected to result from any breakwater field. The downdrift erosion of the 5 and 10 breakwater alternatives was evaluated by computing the change in erosion with respect to existing conditions as shown in Figure 32 and Figure 33.

Both alternatives lead to localized downdrift erosion. However, the 5-breakwater alternative performed better since a larger erosive extent is observed for the 10-breakwater alternative than for the 5-breakwater alternative. Cross-shore mean total transport analysis was conducted on the 5 and 10 breakwater alternatives as shown on Figure 34. Both alternatives show improvements by reducing erosion from 0 mi to approximately 0.6 mi from the jetty. From approximately 1.2 mi to 2.0 mi from the jetty the 5 breakwater alternative leads to less downdrift erosion than the 10-breakwater alternative. These results are further quantified using a one-line model in Section 4.1.1.

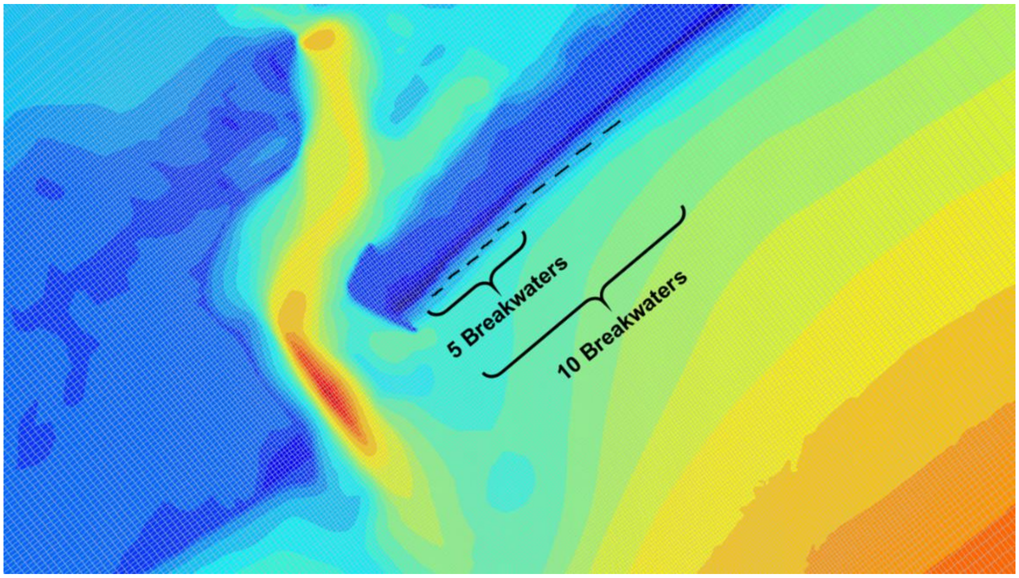

Figure 30. Five and ten and breakwater alternative implementation in nested FLOW domain.

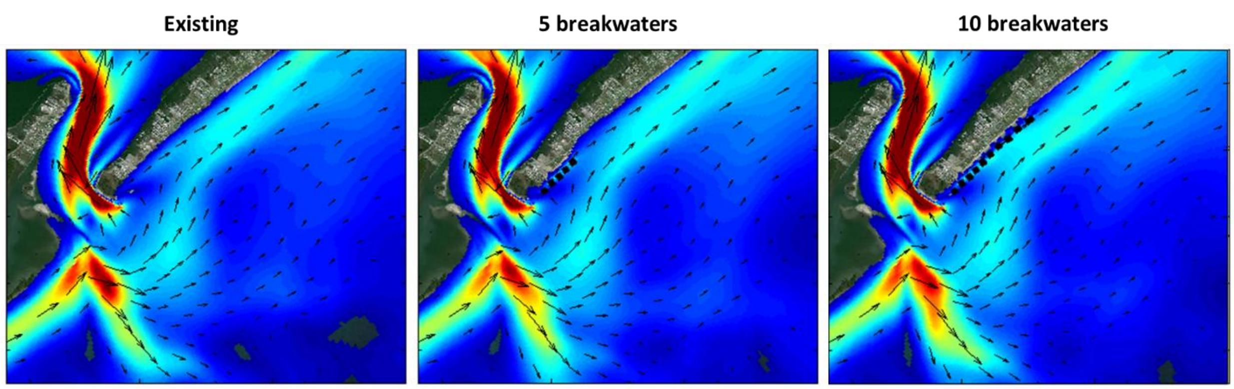

Figure 31. Computed sediment transport vector field over Caminada Pass ebb shoal for existing conditions (left), 5 breakwaters alternative (middle), and 10 breakwaters alternative (right).

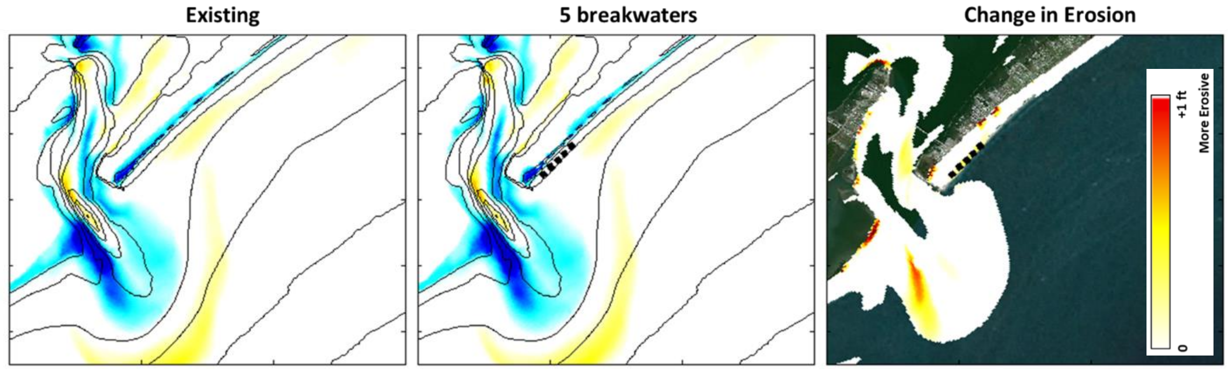

Figure 32. Sedimentation and erosion patterns for existing conditions (left), 5 breakwaters alternative (middle), and change in erosion between 5 breakwater alternative and existing conditions (right). Breakwaters shown in black rectangles.

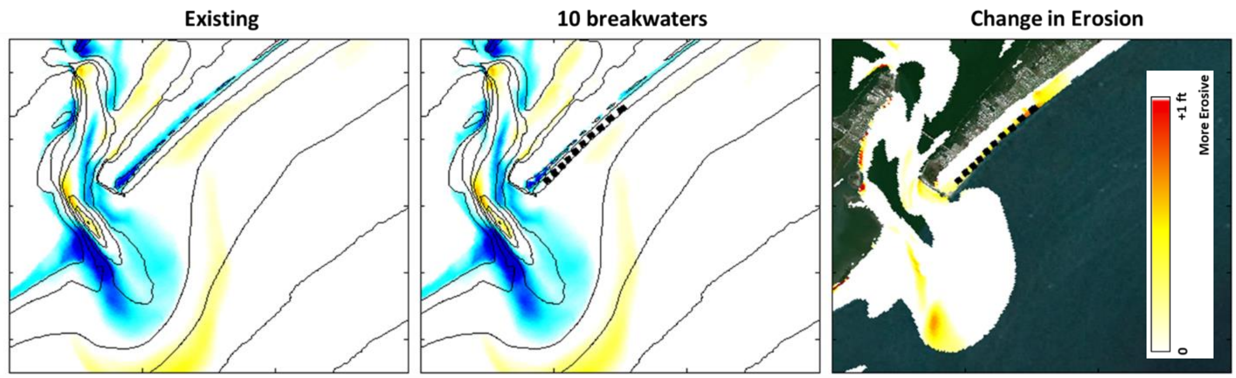

Figure 33. Sedimentation and erosion patterns for existing conditions (left), 5 breakwaters alternative (middle), and change in erosion between 10 breakwater alternative and existing conditions (right). Breakwaters shown in black rectangles.

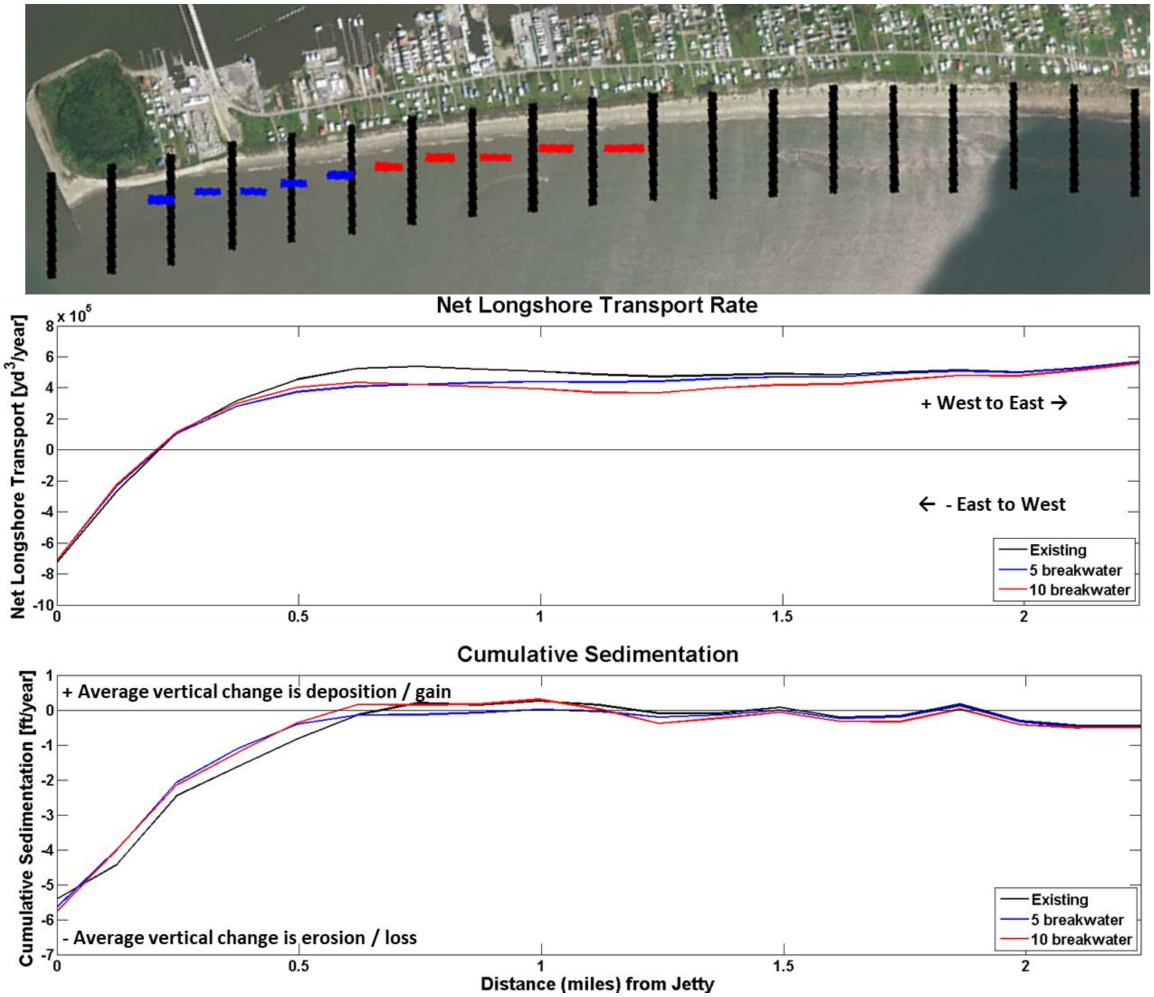

Figure 34. Cross-shore transects (top) with associated net longshore transport (middle) and cumulative sedimentation (bottom), for the 2015 bathymetric surface and reduced time series from June 1, 2015 to June 1, 2018, for existing conditions, 5-breakwater alternative, and 10-breakwater alternative.

4.1.1 Breakwater performance evaluation

The breakwater performance in terms of retaining beach fill and downdrift impacts were evaluated for the 5- and 10-breakwater field using the Gencade model developed in the Mott MacDonald (2017) work. The identical model setup was utilized. Bypassing was shown to be more or less unaffected by the breakwater field and therefore was unchanged in the model setup. As a revetment was constructed in 2017, the model was updated to include the extents of this structure to Station 51+00. In addition, the model includes a beach nourishment geometry identical to the 2017 proposed beach nourishment (Mott MacDonald, 2017) spanning the extents of the 5-breakwater field for both breakwater field alternatives; the beach nourishment is shown in Figure 8 and Figure 9.

Shoreline response results computed by the model are shown after 5 years (Figure 35) of morphology and after 10 years (Figure 36) of morphology for both the 5- and 10-breakwater field. The Gencade analysis showed that 5-breakwater field had no negative impact when compared to the future without project (FWOP) condition because no increase in erosion was observed and shoreline position for the entire western end of Grand Isle is seaward of the future without project shoreline for the 5-breakwater project. On the other hand, the 10-breakwater field resulted in significant downdrift erosion with respect to the future without project condition extending beyond 2 mi from the jetty and impacting the toe of the dune.

At year 5, the beach is at or seaward of the initial shoreline position for the area of interest for the 5-breakwater field. However, the 10-breakwater field shows nearly 75 ft of erosion downdrift of the end of the breakwater field, with downdrift erosion extending for nearly three quarters of a mile. After 10 years, both the 5 and 10-breakwater alternatives still retain beach fill seaward of the initial shoreline for much of the breakwater field, but the downdrift effects of the 10-breakwater field increase erosion by 35 ft whereas the 5-breakwater field remain seaward of the future without project throughout the Grand Isle shoreline.

Overall, the 5-breakwater field does not show downdrift erosion when compared to the future without project condition for the 5 and the 10 years simulation period unlike the 10-breakwater field.

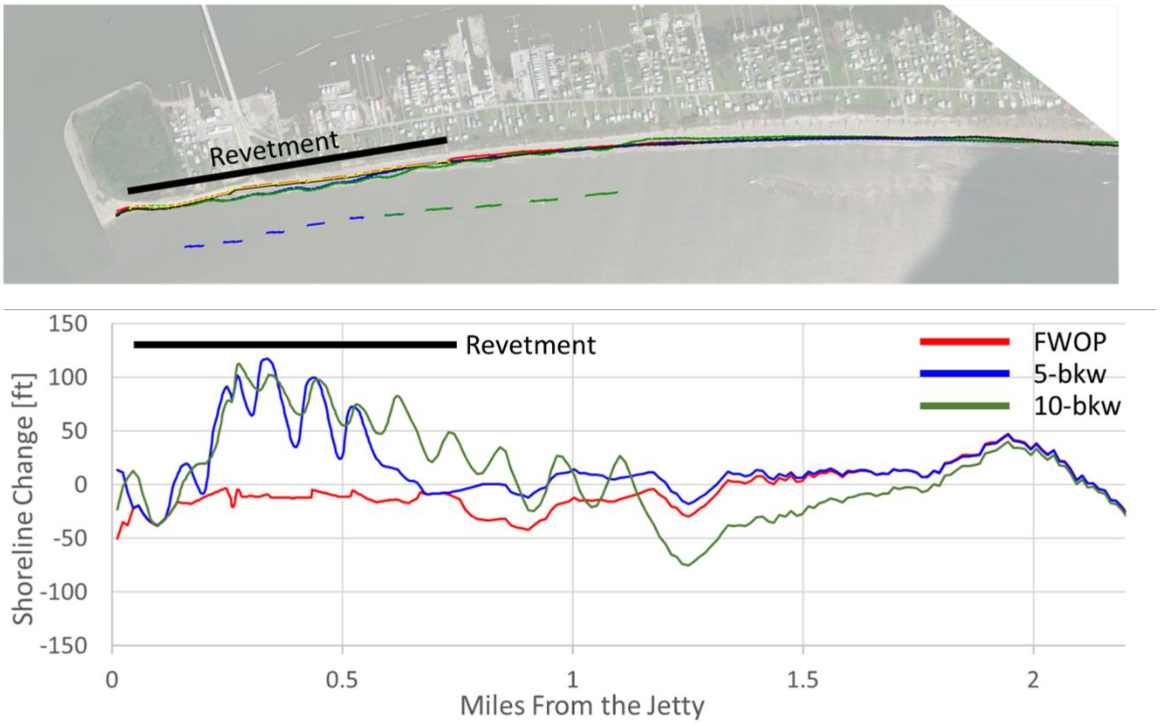

Figure 35. Shoreline change computed by Gencade model at year 5 after construction for future without project FWOP (red line), 5-breakwater (blue line) and 10-breakwater (green line).

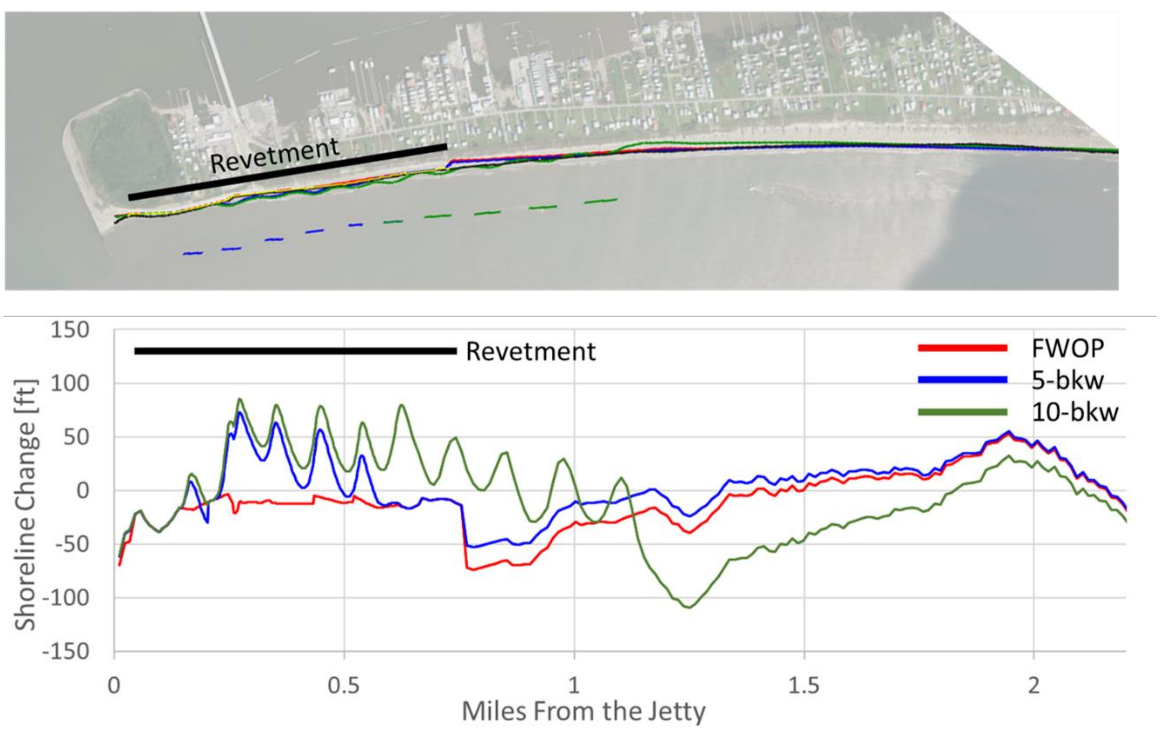

Figure 36. Shoreline change computed by Gencade model at year 10 after construction for future without project FWOP (red line), 5-breakwater (blue line) and 10-breakwater (green line).

4.2 Caminada Pass Ebb Shoal Dredging

The goal of this section is to define three (3) potential borrow sites at Caminada Pass and analyzing their impacts on the Caminada inlet processes. This section references the Borrow Source Technical Note by Mott MacDonald (2019) and the OSI Caminada Pass Borrow Source Desktop Study (OSI, 2018).

4.2.1 Borrow Source Conceptual Design

Following the results of OSI Caminada Pass Borrow Source Desktop Study (OSI, 2018), the 2015 bathymetric surface constructed from BICM survey data, and the Phase 1 Coastal Processes Analysis Report for Grand Isle Levee Dune and Beach Stabilization Project (Mott MacDonald, 2017), Mott MacDonald defined three conceptual level dredge pits in Caminada Pass.

The volume of fill associated with the preferred alternative described in Section 1.3, Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill (see Figure 8), from the Phase 1 study, based on the 2016 bathymetric surface was 310,000 cy (Mott MacDonald, 2017). However, it is necessary to increase the volume of material to account for the changes in bathymetry as well as the cut to fill dredging ratio. It is known the western shoreline of Grand Isle has severely eroded from 2016 to 2017 leading to the armoring of the dune during summer of 2017. The cut to fill dredging ratios are estimated conservatively on the order of 30%. In addition, additional volume is recommended to be included in the borrow pit planning to account for localized variation in material quality or other unexpected restrictions on extractable material such as cultural resources not yet identified. As a result, the 2019 necessary volume of each dredge pit has been estimated to be 700,000 cy to provide contingency for the project.

The “Caminada Sand Body” mapped in 2001 as part of USGS/UNO/USACE Barataria investigation Kindinger, et al. (2001) provides the most comprehensive existing data currently available for designing the conceptual level dredge pits. The USGS/UNO/USACE investigated potential sand sources by means of vibratory coring, other sampling, and subbottom geophysical investigations. The resulting isopachs from USGS/UNO/USACE (shown in Figure 37) were used in this study for designing the conceptual level dredge pits.

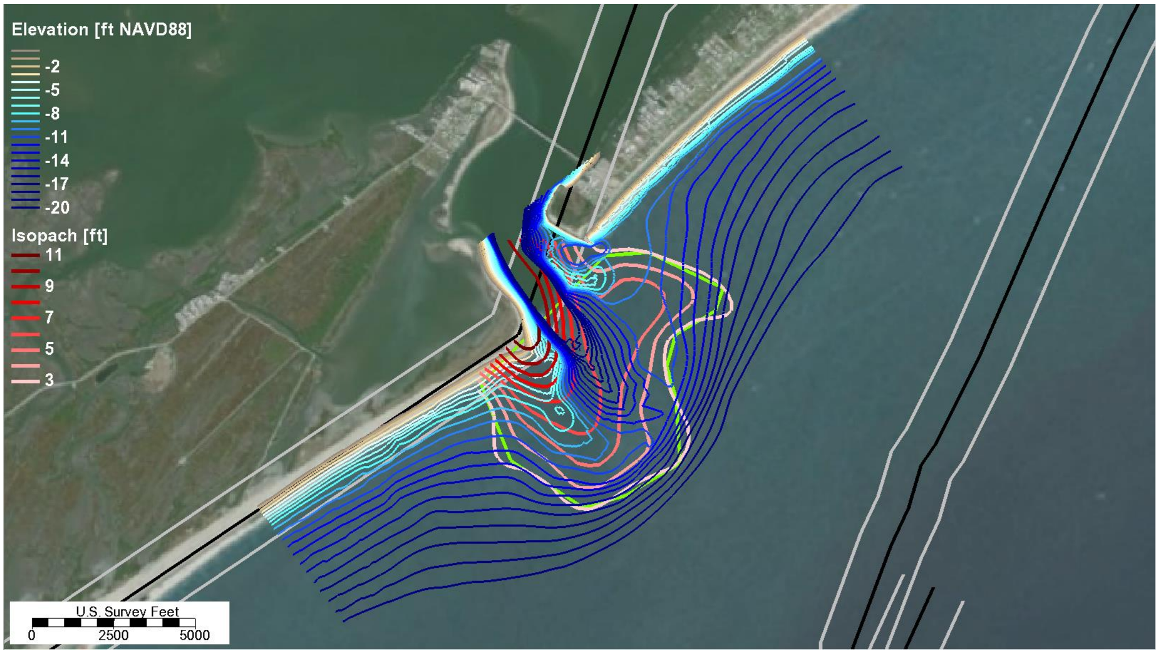

Figure 37. "Caminada Sand Body" as mapped by Kindinger, et al. (2001) shown in green, with isopachs, 2015 bed bottom elevation contours, and known existing pipelines in black with corresponding buffers in grey.

The volumes and bottom of cut elevations for each dredge pit are shown in Figure 38, Figure 39, and Figure 40. The conceptual level dredge pit data using the 2015 bathymetric surface are shown in Table 6. The following guidelines were used in the conceptual level design:

- The conceptual design dredge pits were based on data from Kindinger, et al. (2001).
- It is assumed the Kindirger, et al. (2001) isopachs hold valid under exisitng conditions.
- All dredge pits have a uniform (flat) bottom of cut elevation.
- The bottom elevation of the dredge pits were taken as the top of the upper most sand layer of all the isopachs within the perimeter of the pit.
- No overdredge and buffer were considered for the conceptual design. Overdredge and buffer should be accounted for in the final design.
- 4H:1V ratio was used in designing the dredge pits sides slopes.

Table 6: Conceptual level dredge pit design data based on 2015 bathymetric surface

Dredge Pit ID	Volume [cy]	Bottom Elevation [ft NAVD88]	Description
A	745,000	-15	Deepest possible cut west of Caminada Pass
B	755,000	-14.5	Shallowest possible cut west of Caminada Pass
C	711,000	-14	Cut east of Caminada Pass

The impacts of the potential borrow pits discussed in the following Section 4.2.2 were analyzed using the 2015 bathymetric surface for consistency across the modeling efforts. The numerical model was used to show changes resulting from dredging, not to exactly quantify impacts. For reasons similar to the breakwater analysis, the borrow pit analysis used the 2015 bathymetry as the initial condition for all simulations.

To determine the feasibility of the previously identified potential sand resources, detailed multi-sensor geophysical surveys integrated with a geotechnical sampling program is highly recommended. Such investigations will allow for more complete mapping of the sand resource, determination of suitability and identification of potential obstructions to project activities (OSI, 2018). Using the same dredge pit templates from Table 6, dredge pits volumes were calculated using the 2018 bathymetric surface for comparison purposes; the values are shown on Table 7.

Table 7: Conceptual level dredge pit design data based on 2018 bathymetric surface

Dredge Pit ID	Volume [cy]	Bottom Elevation [ft NAVD88]	Description
A	591,000	-15	Deepest possible cut west of Caminada Pass
B	613,000	-14.5	Shallowest possible cut west of Caminada Pass
C	823,000	-14	Cut east of Caminada Pass

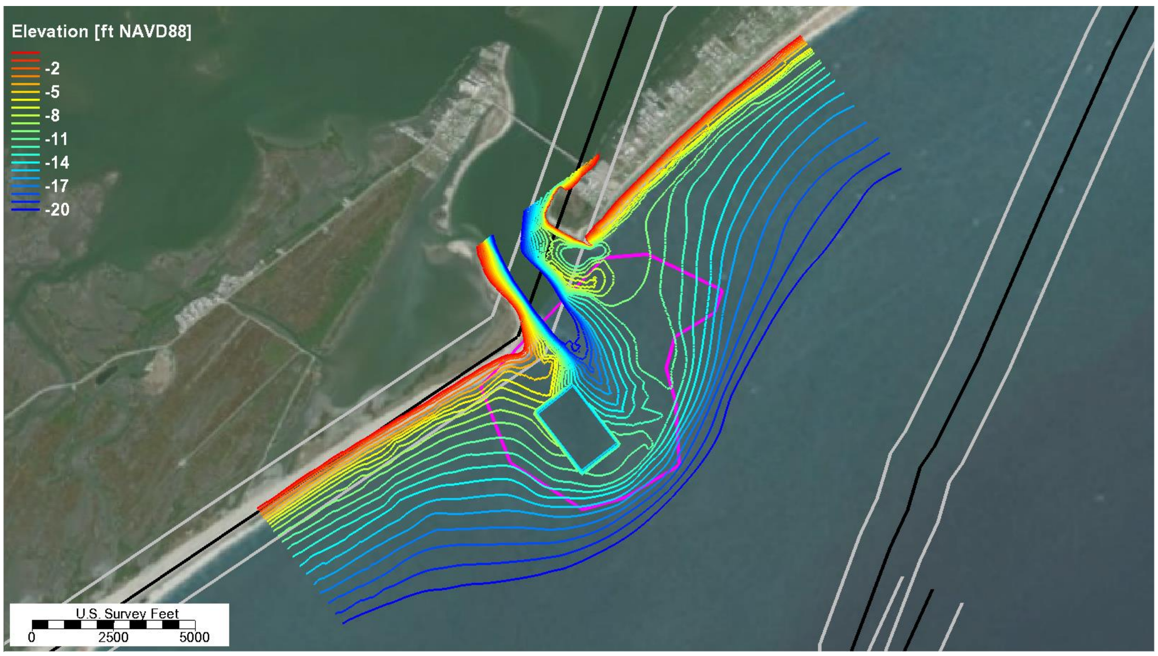

Figure 38. Plan view of Dredge Pit A bed bottom elevation contours with "Caminada Sand Body" in pink and known existing pipelines in black with corresponding buffers in grey.

4.2.2 Borrow Source Impacts on Caminada Pass Inlet Processes

The bathymetric surfaces with dredge pits (or borrow sources) A, B, and C described in section 4.2.1 were incorporated into the initial conditions used in the nested FLOW and nested WAVE model grids described in 3.1. The dredge pits were implemented in the model without including the 5- of the 10-breakwater field alternatives. Thus, the results presented in this section represent the independent impacts of the dredge pits on the Caminada Pass dynamics and the Grand Isle shoreline.

Figure 41 shows the sediment transport vector fields and bypassing over Caminada Pass ebb shoal for the existing conditions and dredge pits A and B. Overall, the dredge pits do not hinder sediment bypassing, but they do have impacts on Grand Isle and Elmer's Island shorelines as illustrated in Figure 42. The effects of the borrow areas are not confined to Gulf-front beaches. In particular, Pit C appears to increase scour with the interior of Caminada Pass and erosion along the interior shorelines of the inlet (see red areas in Figure 42, right graphic). Dredge Pit C on the other had decreases sediment bypassing by 17%.

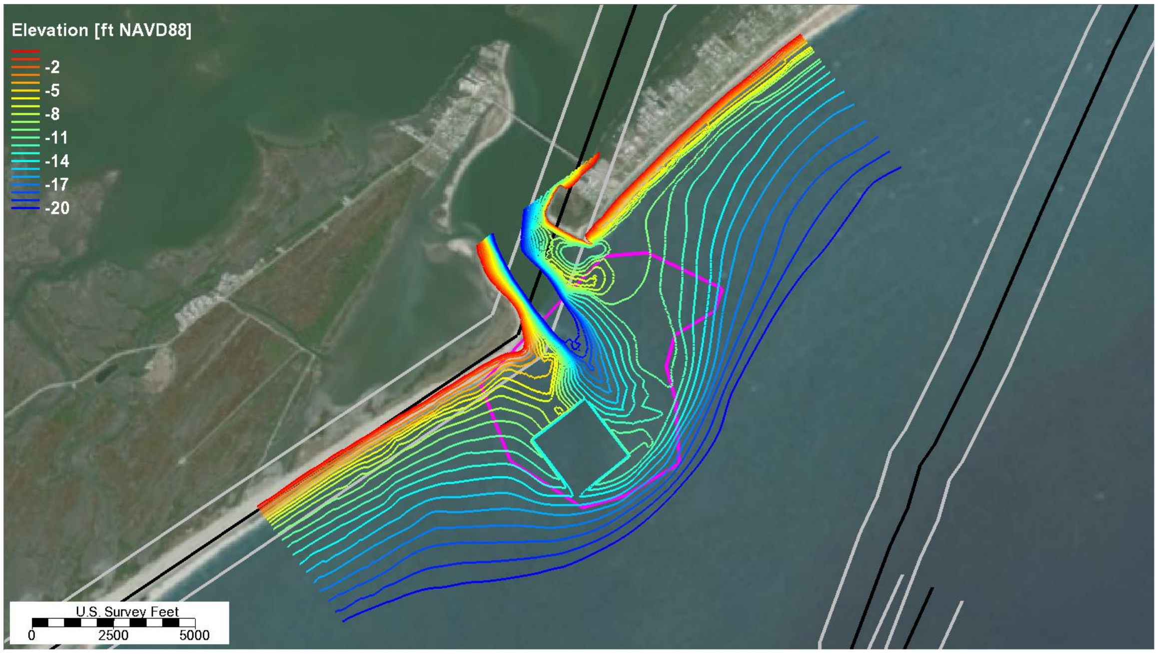

Figure 39. Plan view of Dredge Pit B bed bottom elevation contours with "Caminada Sand Body" in pink and known existing pipelines in black with corresponding buffers in grey.

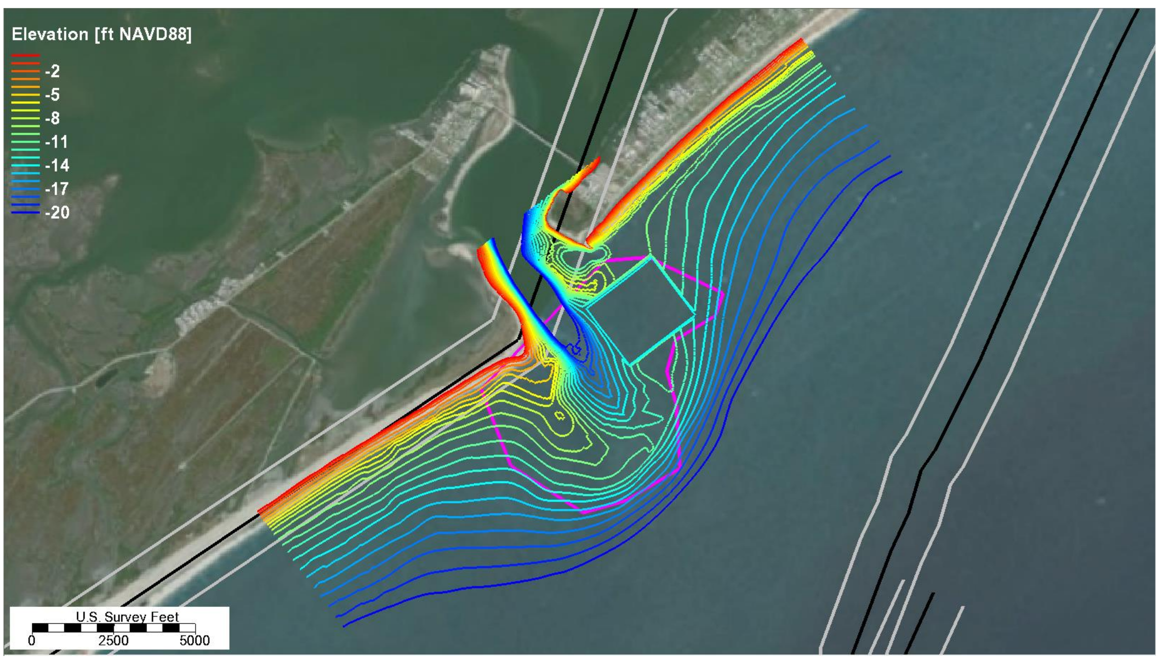

Figure 40. Plan view of Dredge Pit C bed bottom elevation contours with "Caminada Sand Body" in pink and known existing pipelines in black with corresponding buffers in grey.

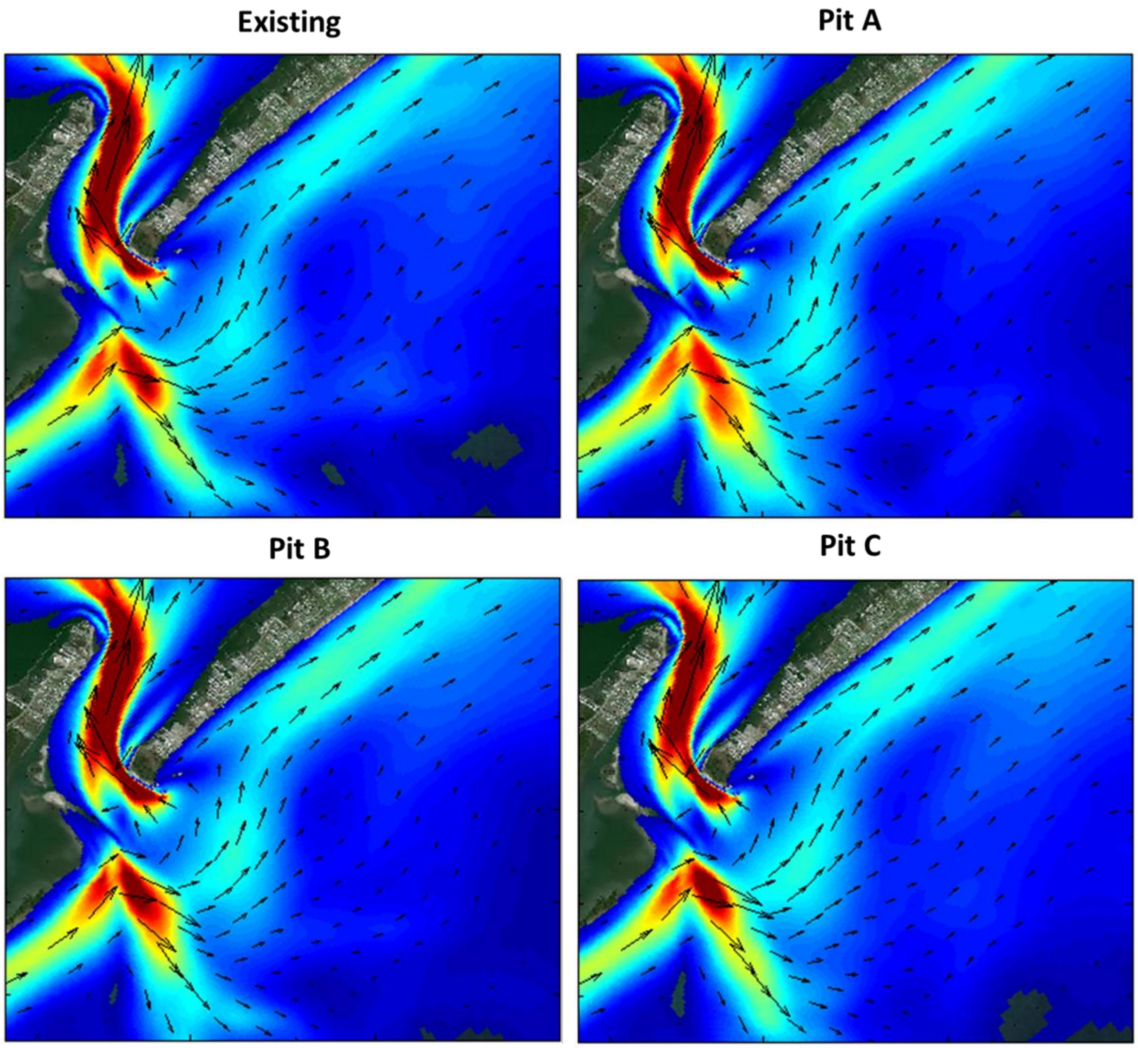

Figure 41. Sediment transport vector field over Caminada Pass ebb shoal for existing conditions (top left), Pit A (top right), Pit B (bottom left), and Pit C (bottom right).

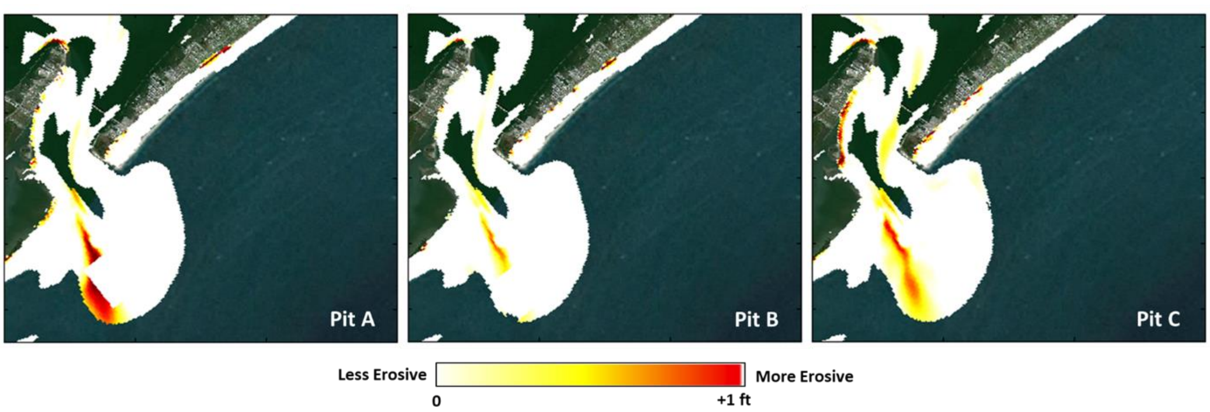

Figure 42. Erosion patterns for Pit A (top left), Pit B (middle), and Pit C (right).

Two sets of transects were used to analyze the nearshore and the ebb shoal impacts of the dredge pits; the nearshore and ebb shoal results are shown on Figure 43 and Figure 44, respectively. In the nearshore, all borrow pits result in an increase in erosion for the first 0.5 mi downdrift of the inlet (up to 1.1 ft/yr). However, along the west end of Grand Isle, Pits A and B show a greater increase in erosion than Pit C. Updrift (west) of the inlet, Pit B appears to result in the largest increases in erosion, followed by Pit C; Pit A has the smallest effect in the nearshore erosion rate along the east end of Elmer's Island. Further offshore the model results suggest that on both sides of the inlet, Pit A will result in the largest increases in erosion or decreases in accretion, followed by Pit B, with Pit C generally having the least impacts.

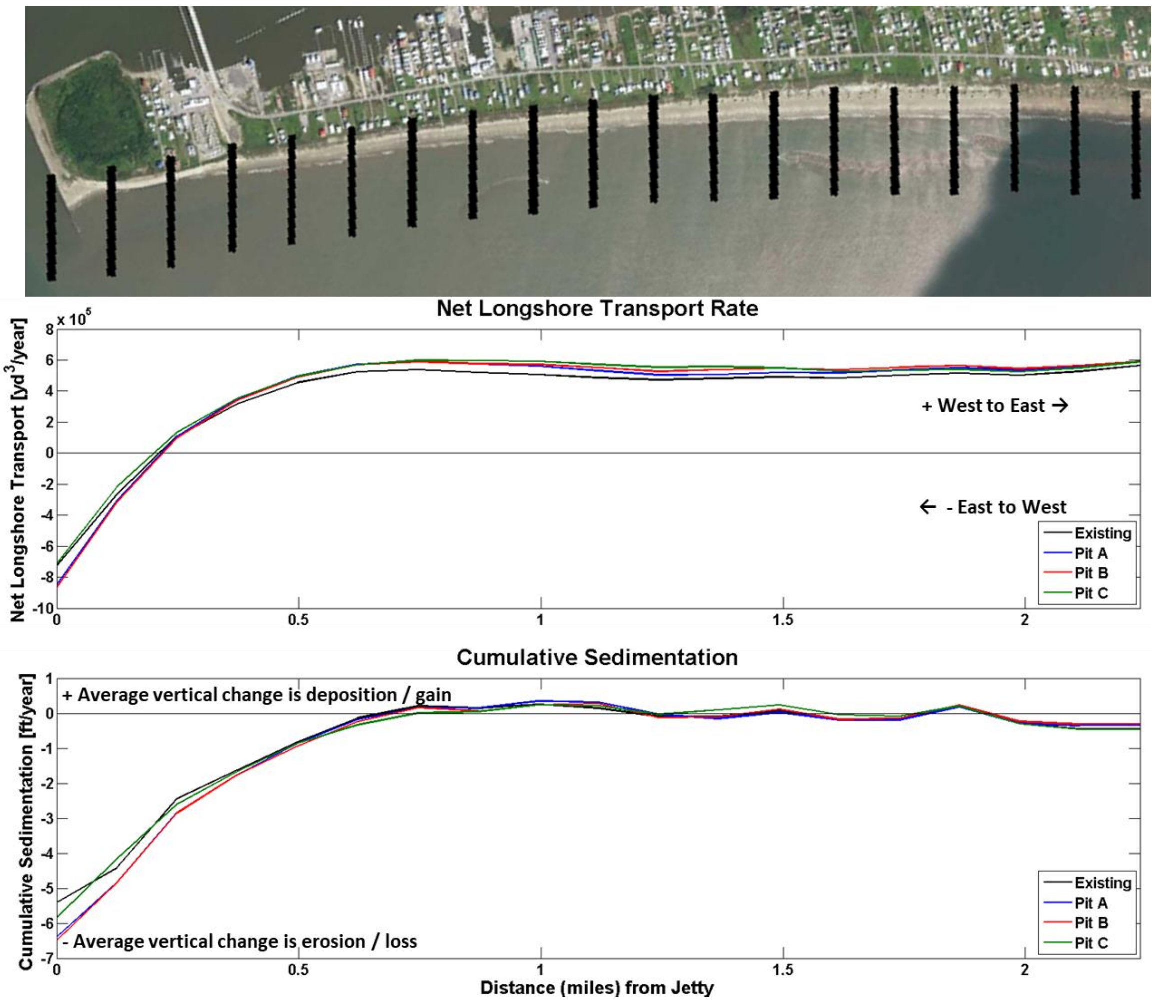

Figure 43. Nearshore cross-shore transects (top) with associated net longshore transport (middle) and cumulative sedimentation (bottom), for the 2015 bathymetric surface and reduced time series from June 1, 2015 to June 1, 2018, for existing conditions and dredge pits A, B, and C.

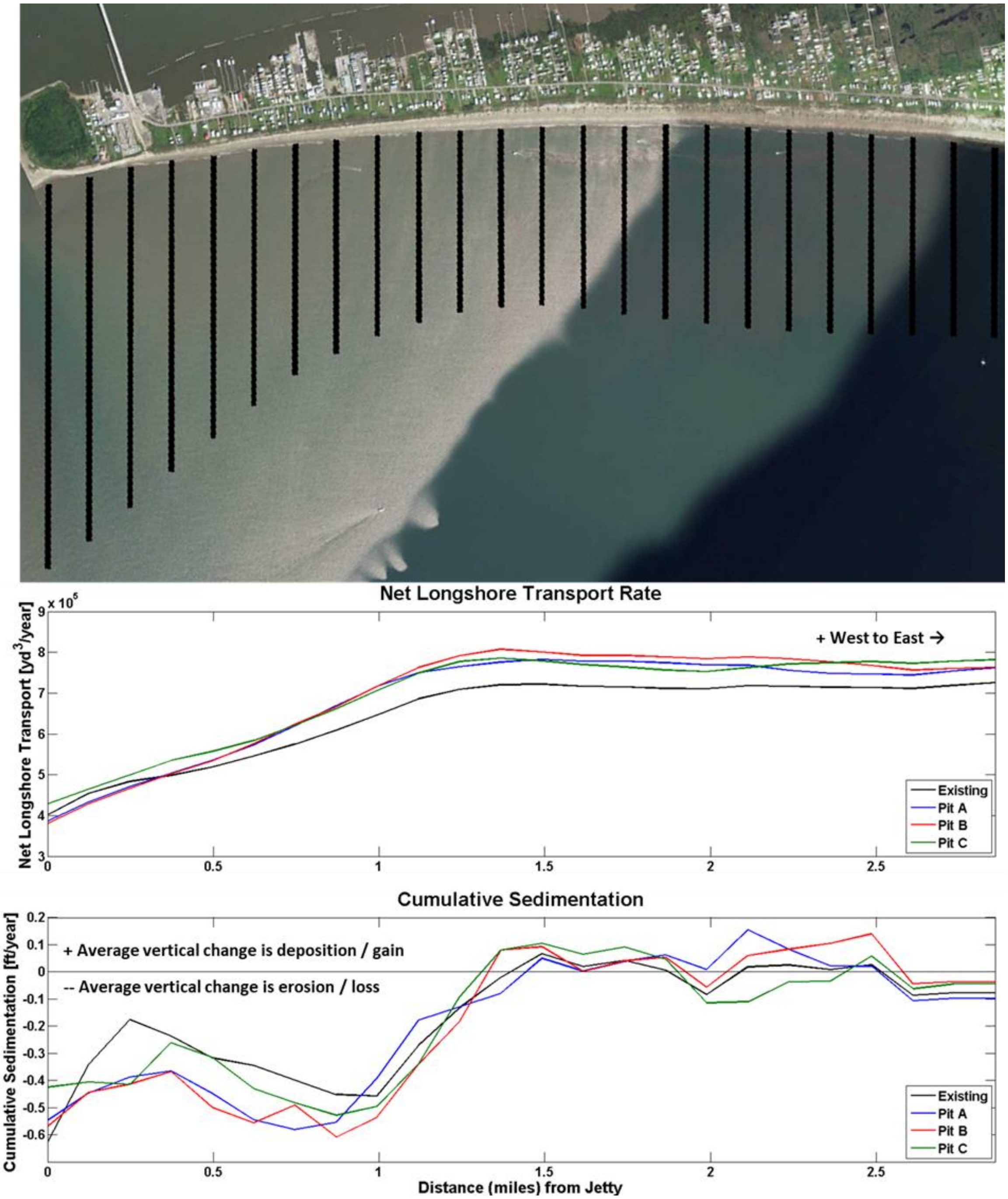

Figure 44. Ebb shoal cross-shore transects (top) with associated net longshore transport (middle) and cumulative sedimentation (bottom), for the 2015 bathymetric surface and reduced time series from June 1, 2015 to June 1, 2018, for existing conditions and dredge pits A, B, and C.

The impacts from the borrow pits on the bypassing was computed in alternative manner by calculating the flux of sand the is directed from the shoal to the nearshore area that is transported through the area shown in the box in Figure 45. Results are shown in Table 8. Both Pits A and B have minimal changes to sand bypassing to Grand Isle for the 3 year period simulated in the model. Pit C reduces the sand bypassing to Grand Isle by 18%.

Table 8: Change in sand bypassing arriving at Grand Isle from dredging Caminada Pass.

Simulation	% Change in sand bypassing from existing conditions
Pit A	1%
Pit B	2%
Pit C	-18%

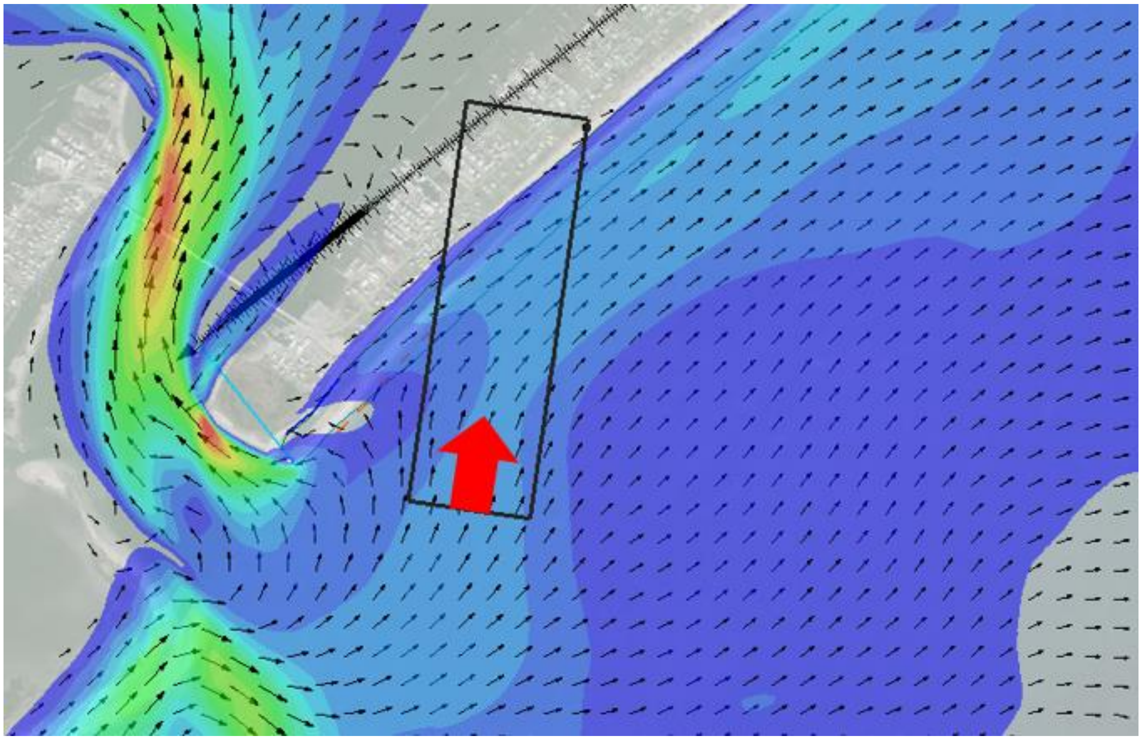

Figure 45. Mean total transport for existing conditions and box of bypassing flux calculation.

5 Breakwater Field Optimization

The optimization the breakwater field includes determining breakwater length, spacing between breakwaters, breakwater field distance from shoreline, and length of breakwater field (number of breakwaters). Both breakwater field alternatives show improvements by reducing erosion from 0 mi to approximately 0.6 mi from the jetty. However, the recommended length of the breakwater field has been based on the comparative analysis between the existing conditions and the 5- and 10-breakwater field using the numerical model results described in 4.1 and 4.1.1.

Analytical methods were employed for recommending the breakwater length, spacing between breakwaters, and breakwater field distance from shoreline. There is no one standard formula or methodology to compute shoreline response due to detached breakwaters. Therefore, five different empirical methods were used to evaluate the performance of the 5-breakwater, summarized in Table 9. They include methodologies defined by the Shore Protection Manual (USACE, 1984), Inman and Trautschy (1976), Gourlay (1981), the empirical approach presented by Dally and Pope (1986) and Ahrens and Cox (1990).

Table 9: Methods used in Empirical Breakwater Analysis.

Method	Minimum Response	Salient Formation	Tombolo Formation
Inman and Trautschy (1978)	Ls/Y > 0.17 to 0.33	-	-
Ahrens and Cox (1990)	Ls/Y < 0.27	Ls/Y < 0.8 to 1.5	Ls/Y > 2.5
Dally and Pope (1986)	Ls/Y <0.125	Ls/Y = 0.5 to 0.67	Ls/Y = 1.5
SPM (1984)	-	-	Ls/Y > 2.0
Gourlay (1981)	-	Ls/Y < 0.4 to 0.5	-

Where L_s = length of breakwater; Y = distance from the shoreline

Figure 46 shows the boundaries of the criteria defined by different methods. The ranges for the three criteria - minimal shoreline response, salient formation, and tombolo formation – are denoted with an arrow spanning the appropriate region. The work conducted during Grand Isle Shoreline Stabilization Study Part 1: Basis of Engineering (CHE, 2008) validated the general ranges of the empirical formulas on the Grand Isle shoreline with the existing breakwaters on the eastern end of the island.

For the breakwaters to be effective, some shoreline response is required. The stronger the shoreline response (more tombolo response), the more stable the shoreline. However, with tombolo response, the sediment transport behind the breakwaters is dramatically reduced, if not eliminated, which results in large downdrift impacts. Salient response reduces the longshore transport which is desirable but does not completely eliminate it. Generally, the desired shoreline response when downdrift impacts are detrimental is a salient response.

The methods described in Table 9 were used to evaluate the performance of the breakwater field. To setting the distance offshore, we first consider constructability. Construction in depths shallower than 5 to 6 ft of water will reduce construction efficiency and increase prices due to draft limitations. Setting the breakwaters at this range of depths for the 5-breakwater field alternative puts the longest offshore distance is on the eastern side of the breakwater filed at 350 ft. After incorporating a beach nourishment by extending the shoreline in the cross-shore direction by 160 ft, the shortest offshore distance becomes 190 ft. Thus, the resulting range of offshore distance spans 190 to 350 ft which is plotted as green points connected with a line in Figure 46. Using a breakwater length of 250 ft with this distance offshore will yield salient formation. The breakwater spacing should not exceed 1 to 1.5 time the breakwater length otherwise, shoreline erosion may occur (Bricio, et al 2008). Based on the 2018 bathymetric data, the associated bottom elevations satisfying the salient criteria are between -6 ft on the west and -5.5 ft on the east side of the breakwater field. Salient formation is expected under the existing shoreline condition or under beach nourishment conditions.

Figure 47 illustrates the breakwater field optimization results including number of breakwaters, distance from offshore, breakwater length, breakwater spacing and bottom elevation.

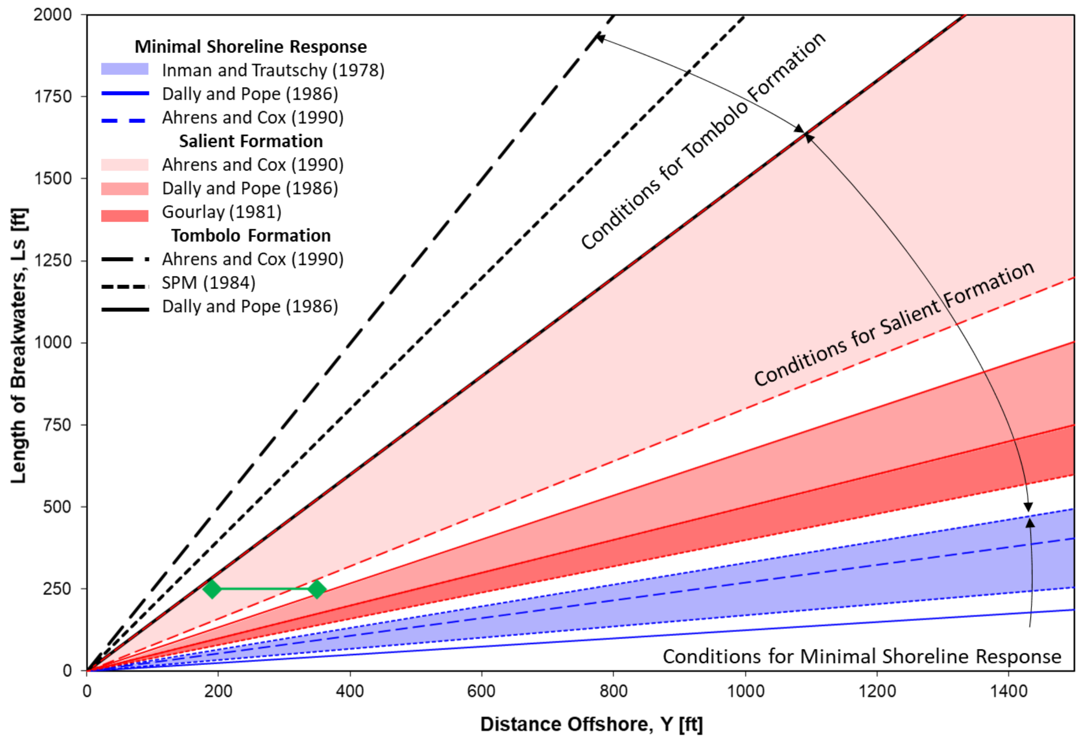

Figure 46. Evaluation of breakwater performance for existing eastern breakwaters. Optimum distance offshore and length of breakwater shown in green

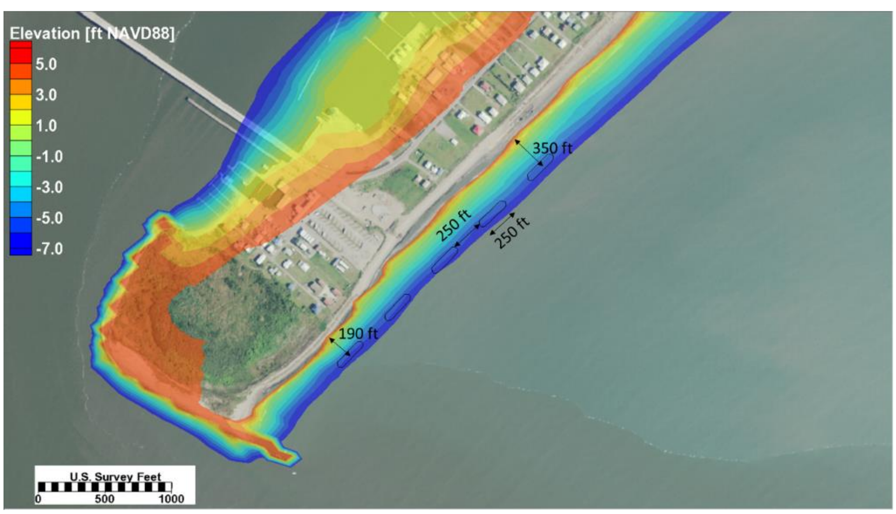

Figure 47. Distances and 2018 bottom elevations associated with the optimization of the breakwater field.

6 Recommendations

The recommendations listed in this section pertain to the preferred alternative, Segmented Offshore Breakwaters + Mitigation Dune + Beach Fill shown in Figure 8, which includes the optimization of the breakwater field and the feasibility of Caminada Pass ebb shoal as a potential borrow source area for beach nourishment.

6.1 Breakwater Geometry

The 5-breakwater field is recommended over the 10-breakwater field. The Delft3D analysis showed that the extent of the 5-breakwater field does not reach the location where the sediment bypassing attaches onto the Grand Isle shoreline while the extent of the 10-breakwater field does reach it. The Delft3D analysis showed that the 5-breakwater field alternative is likely to have less downdrift erosion on the Grand Isle shoreline than the 10-breakwater field.

The Gencade analysis showed that for the 5-breakwater field, the shoreline position for the entire western end of Grand Isle is seaward of the future without project shoreline and the alternative has no negative impact when compared to the future without project condition; no increase in downdrift erosion was observed. On the other hand, the 10-breakwater field resulted in significant downdrift erosion with respect to the future without project condition extending beyond 2 mi from the jetty and impacting the toe of the dune.

The analytical and empirical methods used for recommending the breakwater length, spacing between breakwaters, breakwater field distance from shoreline, and spacing between breakwaters (USACE, 1984; Inman and Trautschy,1976; Gourlay,1981; Dally and Pope, 1986; Ahrens and Cox, 1990; and Bricio, et al 2008) were employed to optimize the breakwater field.

For the breakwaters to be effective, some shoreline response is required. Salient response reduces the longshore transport which is desirable but does not completely eliminate it. Generally, the desired shoreline response when downdrift impacts are detrimental is a salient response. Salient formations are expected under the existing shoreline condition or under beach nourishment conditions. The length of the breakwater as well as the spacing between the breakwaters is recommended to be 250 ft. The offshore distance varies across the breakwater field with the shortest distance of 190 ft at -6 ft NAVD88 and the longest distance of 350 ft at -5 ft NAVD88. Shallower depths and therefore a shorter offshore distance may reduce construction efficiency.

6.2 Caminada Pass Ebb Shoal Dredging

Three different borrow area alternatives were examined using the Delft3D model – two alternatives that would be located on the western lobe of the Caminada Pass ebb shoal (Pits A and B) and a third located eastern lobe (Pit C). The model results suggest that Pit C located on the eastern lobe of the ebb shoal will result in the greatest increases in scour and shoreline erosion in the interior of the inlet but the smallest increases in nearshore erosion on the Gulf-front shorelines on either side of the inlet, however, it will also reduce the sand transported to the Gulf Shoreline through bypassing by nearly 18%, while Pits A and B have negligible changes to overall bypassing. These results appear to be in conflict but are speculated to reflect the complex morphology of the system. Pit C likely changes the hydrodynamics in a way that minimize impacts to the shoreline in the 3 year time period modeled compared to Pits A and B. However, the change in sediment flux to the shoreline is likely to take a longer time period to appear as shoreline impacts than was modeled.

Generally, the modeling indicates that the impacts of dredging the ebb shoal may be mild to moderate and the Caminada Pass may be a feasible borrow source that should be further considered. We recommend that a more detailed geotechnical investigation be performed to develop a better understanding of the sand body geometry, and then develop a more precise borrow site for further evaluation.

Based on the studies of a large borrow area located along the Florida Gulf Coast by Dabees and Kraus (2012), additional modeling with an assessment of refilling rates is recommended to confirm that the borrow area will not lead to long-term, negative, downdrift impacts.

6.3 Next Steps

Further evaluation of the project geometry may be warranted based on expected project budget for construction and cost estimates for the breakwater construction and beach nourishment after more refined design. Some optimization may be required if project budget is limiting. In addition, we recommend additional effort to minimize downdrift impact and optimization of the time when downdrift impacts start to occur. This may include variations of the beach nourishment template based on available funds.

In addition, downdrift erosion at the end of breakwater field and existing revetment is expected for any number of the breakwaters. We recommend consideration of additional sand placed as beach and/or dune nourishment to further reduce downdrift erosion at the transition between the end of the breakwater field and the end of the existing revetment.

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