COASTAL ENGINEERING APPENDIX A
FOLLY BEACH, SOUTH CAROLINA

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Sub-Appendix

1 Introduction

The U.S. Army Corps of Engineers (USACE) Wilmington District is evaluating continued Federal interest of the Folly Beach, SC coastal storm risk management (CSRM) project. The project extends 30,890 feet along beachfront of the City of Folly Beach, see Figure 1-1. The ultimate goal of the study is to calculate the benefits for proposed project beachfill template for use in calculating the project benefit to cost ratio for a proposed 50 year extension to the authorization.

Figure 1-1. Folly Beach Project Area

The Beach-fx software was utilized to analyze the physical performance of the proposed template for the storm damage reduction project in the Folly Beach study area as well as the economic benefits and costs. Beach-fx is an event-based, Monte Carlo life cycle simulation tool capable of estimating storm damage along coastal zones caused by erosion, flooding, and wave impact. The software also calculates the economic benefits and costs associated with alternatives. The purpose of this appendix is to describe the Coastal Engineering input driving the Beach-fx software for the Folly Beach study area. This includes developing the representative reaches for the study area, a historical storm suite, historic shoreline change conditions, and profile response to the array of storm events using SBEACH.

2 Project Background

2.1 Current Authorized Project

The Folly Beach Shore Protection Project was authorized by Section 501 of the Water Resources Development Act of 1986, Public Law 99-662, as amended, and modified by the Energy and Water Development Appropriations Act of 1992, PL 102-104. The purpose of the project is to reduce damage to structures and shorefront property related to erosion and storms. The 1991 General Design Memorandum included a protective berm 15 ft wide at elevation 8.0 ft NAVD88 with a foreshore slope of 1V:10H to the mean high water (MHW) line then offshore at 1V:30H out to the existing bottom. The initial project length was 28,220 ft and the project included nine rehabilitated steel sheet pile groins. The initial and following beach fills included advanced nourishment of varying volume. The project was modified in 2005 with 670 feet added to the northeast end of the project for a total length of 28,890 ft.

2.2 Previous Nourishment Projects

Initial construction for the currently authorized project was completed in 1993 and involved the placement of approximately 2.7 million cubic yards (mcy) of sand on the beach with 3.1 mcy dredged from the Folly River. The shoreline was nourished in 2005 with approximately 2.3 mcy of sand from offshore. A partial nourishment occurred in 2007 with approximately 0.49 mcy of sand being placed on the beach. Borrow area locations are discussed in greater detail in Section 6 including a location map. A summary of past nourishment projects is provided in Table 2-1 and in Figure 2-1 with information available from the link below. Volume is placed on the beach is not the same as excavated from the borrow site. The Folly River has been used in previous nourishment projects of Folly Beach including 1993, 2013 and 2018. From 1979 to 2000 material dredged during maintenance of the navigation channel in the Folly River and Stono Inlet were placed on the southwest end of the island at the Folly Beach County Park. The Folly River is regularly recharged with material eroded from Folly Beach and beach nourishments sourced from offshore borrow sites adds to the littoral sand budget.
https://gis.dhec.sc.gov/renourishment/

2.3 Charleston Harbor Jetties

The 1987 the USACE report "Evaluation of the Impacts of Charleston Harbor Jetties on Folly Island, South Carolina" addressed the Section 111 issue of shoreline damage attributable to a federal navigation project (USACE, 1987). A sediment budget analysis was used to determine the impact of the jetties on the sub-aerial beach at Folly Island. The report states that approximately 57% of the sub-aerial beach volume loss can be attributed to the jetties. The report states that littoral sediment transport from the north has been blocked by the jetties causing a decreased sediment supply to Folly Island and to offshore areas. Morris Island is to the north of Folly and is also impacted by loss of sediment. The reduced sediment to the ebb-tide shoal and the steeping offshore profile has increased the wave energy along Folly Island and resulted in the landward migration of the ebb-tide shoals at Lighthouse Inlet.

2.4 Datums

All elevations provided in this report and used in the modeling efforts are in feet, NAVD88 vertical datum. The conversion from NGVD29 to NAVD88 is (NGVD29 – 0.98 ft) = NAVD88, the rounded value of - 1.0 ft has traditionally been used for the Folly Beach project. For NOAA nautical chart conversions the mean lower low water is at elevation -3.14 ft NAVD88.

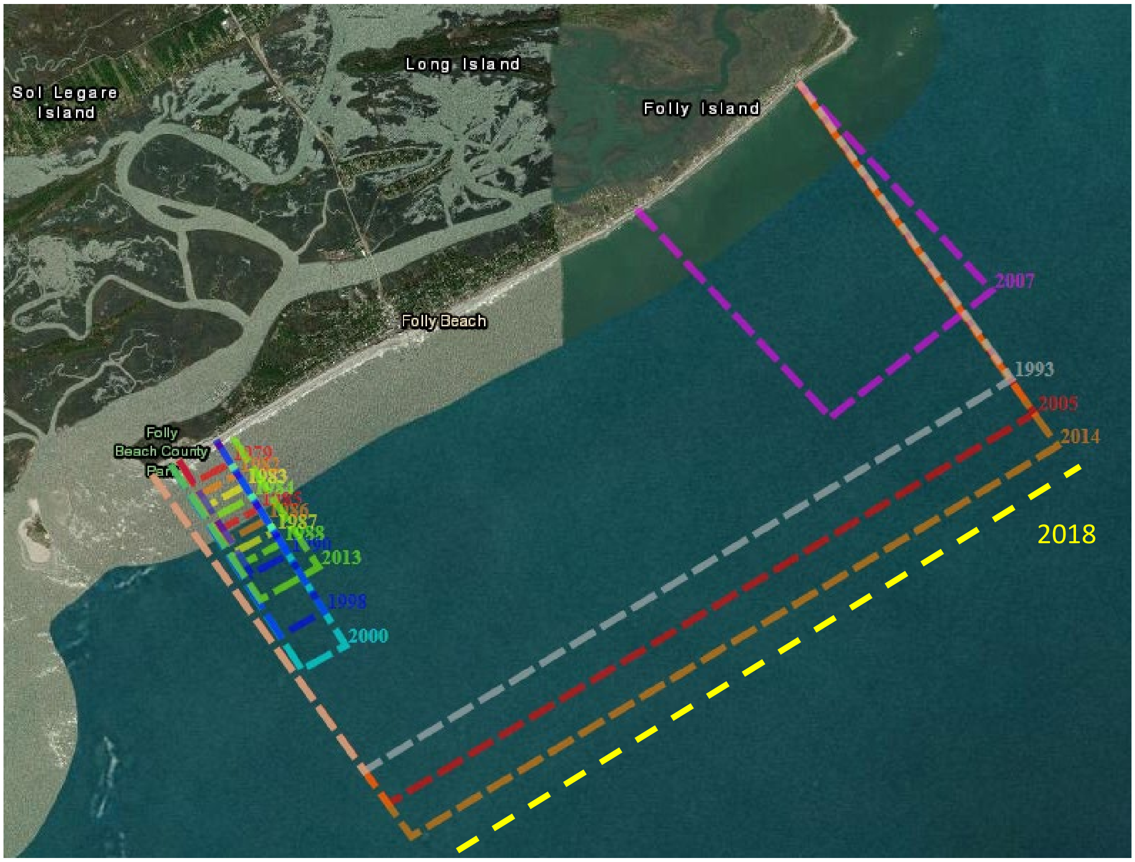


Figure 2-1. Previous Renourishment Events (Source: SCDHEC-OCRM)

Table 2-1. Previous Nourishment Projects

Year	Location or Station Volume Placed		Length (miles)	Total Placed (CY)	Source
1979	Southwest End		~0.5	20,000	Nav Channel
1982	Southwest End		~0.5	43,700	Nav Channel
1983	Southwest End		~0.5	43,700	Nav Channel
1984	Southwest End		~0.5	43,700	Nav Channel
1985	Southwest End		~0.5	43,700	Nav Channel
1986	Southwest End		~0.5	43,700	Nav Channel
1987	Southwest End		~0.5	43,700	Nav Channel
1988	Southwest End		~0.5	43,700	Nav Channel
1990	Southwest End		1.00	240,000	Nav Channel
1993	0+00	282+20	5.33	2,700,000	Folly River
1998	0+00	30+00	0.43	55,000	Folly River
2000	Southwest End		~0.5	101,500	Nav Channel
2005	0+00	288+90	5.34	2,395,000	Offshore
2007	188+00	288+90	1.91	490,000	Offshore (B)
2013	10+00	029+00	0.53	415,000	Folly River
2014	28+48	288+90	4.93	1,400,000	Offshore (A, B, C & D)
2018	28+48	288+90	4.93	1,200,000	Folly River

2.5 General Features

Folly Beach is located on Folly Island about eight miles south of Charleston, SC. The island is about six miles long and has a maximum width of 2,800 ft near the center of the island. The island narrows to 200 ft wide on the northeast end at a location known as the "Washout". The island is bounded by Lighthouse Inlet on the northeast and by Stono Inlet to the southwest. The tidally influenced Folly River is located behind the southeast end of the island and flows into Stono Inlet. Elevations on the island range from a low of 5 ft NAVD88 to over 14 ft NAVD88 along a remnant dune system that runs intermittently along the center of the island. The entire length of Folly Beach is experiencing shoreline recession with higher rates at the ends of the island and lower rates along the middle. The predominate longshore drift is toward the southwest. The mean grain diameter of the native beach is 0.17 mm (USACE, 2017).

2.6 Groin Structures

Groins are structures built perpendicular to a shoreline designed to trap and hold sand as it moves along the shore with the longshore drift. There are 50 groins along Folly Beach that were constructed by various local, state and federal agencies between the 1940's and 2013 (Folly Beach, 2015). An estimated 28 groins are non-functioning remnant timber and riprap structures and 22 are functioning. A GIS database was created to locate and catalog the structures and are shown in Figure 2-2.

There five timber sheet pile and riprap groins on the northeast end of the island originally constructed by the US Coast Guard in the 1970's. This 2,000 ft section is now referred to as the Lighthouse Inlet Heritage Preserve and is not included withing the currently authorized project. Three of the groins are in poor condition but are able to currently trap sand.

Nine groins were rehabilitated as part of the 1993 USACE beach nourishment project. The groins are located between Stations 109+00 and 158+00. The groins are steel sheet pile with a concrete cap and riprap placed along the base. The length of the nine groins varies between 100 and 200 ft with a crest elevation of approximately 6.5 ft NAVD88. The City of Folly Beach maintains the groins and they are currently functional.

In June 2013 a 745 ft long steel sheet pile groin with armor stone toe protection was constructed at the Folly Beach County Park on the southwest end of the island near Station 10+00. The groin was constructed in three sections to match the elevation of the berm, beach face and low-tide terrace (Kaczkowski, et al 2015). The project included 415,000 CY of material dredged from the Folly River to fill updrift reach of the groin. The groin was constructed to protect the Park's recreational beach and infrastructure.

Nine groins were rehabilitated in 2018 by the City of Folly Beach between Stations 164+30 to 210+60. Rehabilitation included removal of damaged timber sheet piles and rebuilding with armor stone and grout. Lengths of the groins varied between 242 ft and 336 ft to match the existing structure footprint. The general design included a crest elevation of 6.0 ft NAVD88 extending from the OCRM jurisdictional line to past the existing MHW contour then sloping downward to terminate with a crest elevation of approximately 2.0 ft NAVD88.

2.7 Revetments and Bulkheads

The Folly Beach shoreline is protected by numerous concrete and timber sheet pile bulkheads, stone revetments, concrete rubble revetments and bulkheads with armor stone at the base. The structures are of various length, elevation, design, age and construction quality. The location and elevations of exposed structures were surveyed in February 2019 by the Wilmington District. Information on buried armor structures was obtained from the 2015 Folly Beach Management Plan (Folly Beach, 2015). The location of structures included in the Beach-fx model can be reviewed in Figure 2-3. Construction of new or improved armoring is expected to continue in the future.

The Central Business District (CBD) includes the Tides Hotel and condominiums that are protected by a engineered concrete sheet pile seawall 1,528 ft in length and is in good condition, Station 96+57 to 111+85. This section was entirely contained within Beach-fx model Reach 8 (described Section 4.1 in this report). In the early 1990’s the South Carolina Department of Transportation (SCDOT) built a rock revetment to protect a 2,750 ft section of East Ashley Avenue at the “Washout” and is good condition. This revetment is between Station 210+00 to 237+50. The 2019 survey data was used to define the intermittent armoring at private beachfront lots within the Beach-fx model. New or replaced armor structures at single residential lots in Beach-fx were assumed to follow design guidelines in the Folly Beach Code of Ordinances. Locations with non-engineered small riprap or concrete rubble revetments were not included in the Beach-fx analysis.

The Beach-fx failure threshold for the armor structures utilized recommendations from the Egmont Key Feasibility Study and St Johns County SPP Beach-fx modeling efforts. The flooding armor failure threshold was assumed to occur when the structure was overtopped by 1.0 ft of flooding. Erosion failure for the timber bulkheads at residential properties was assumed to occur when ½ of the bulkhead height was exposed by erosion. Erosion failure of the concrete seawall in the CBD was assumed to fail when ¾ of the seawall was exposed. Erosion failure of stone revetments was assumed to occur when erosion reaches the base of the revetment. It was assumed that the seawalls and bulkheads are more likely to fail due to erosion before wave damage failure. A wave height of 10 ft was used as the wave damage armor failure threshold for seawalls and bulkheads to cover extreme storm events. For wave damage failure of revetments the USACE program ACES was used to determine the wave height until a damage level of 8 was reached in the rubble mound design revetment module.

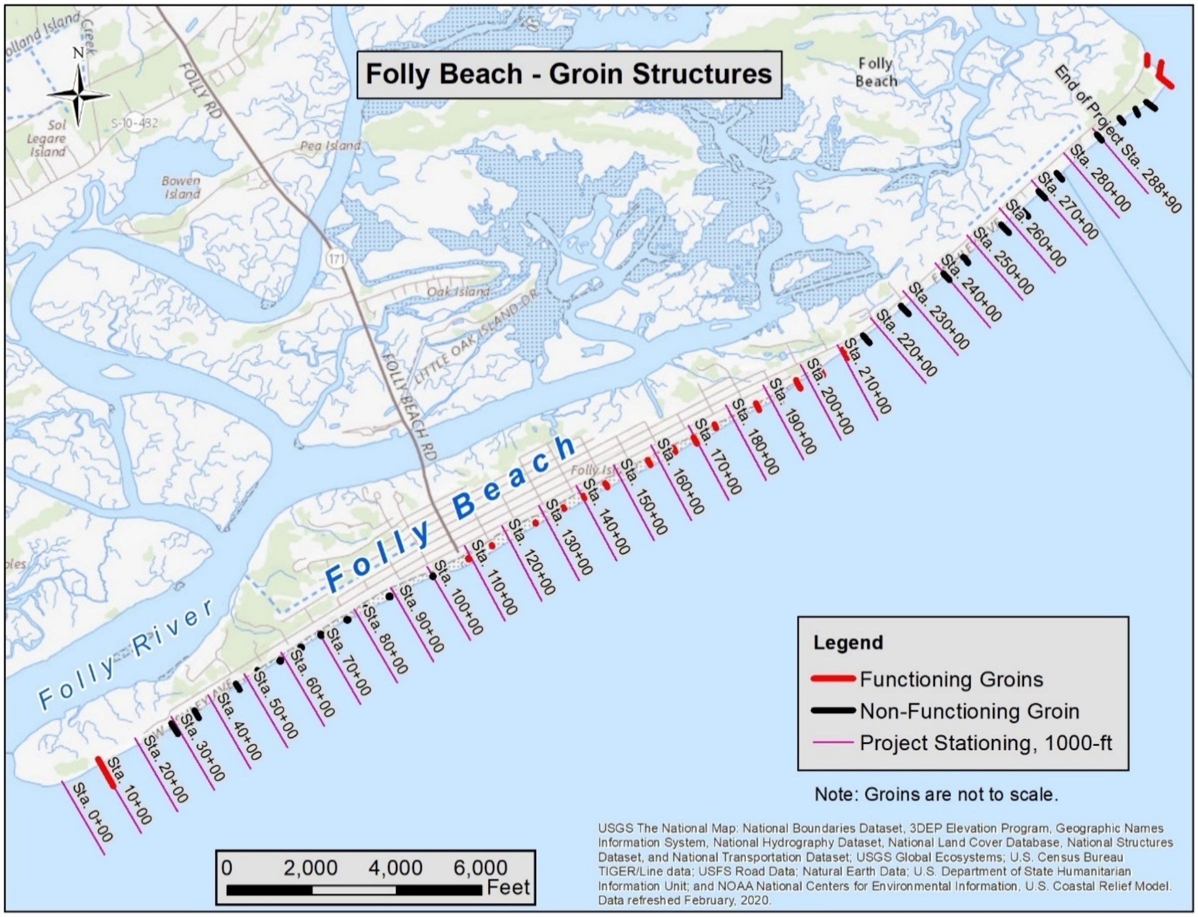


Figure 2-2. Existing Groin Structure Location

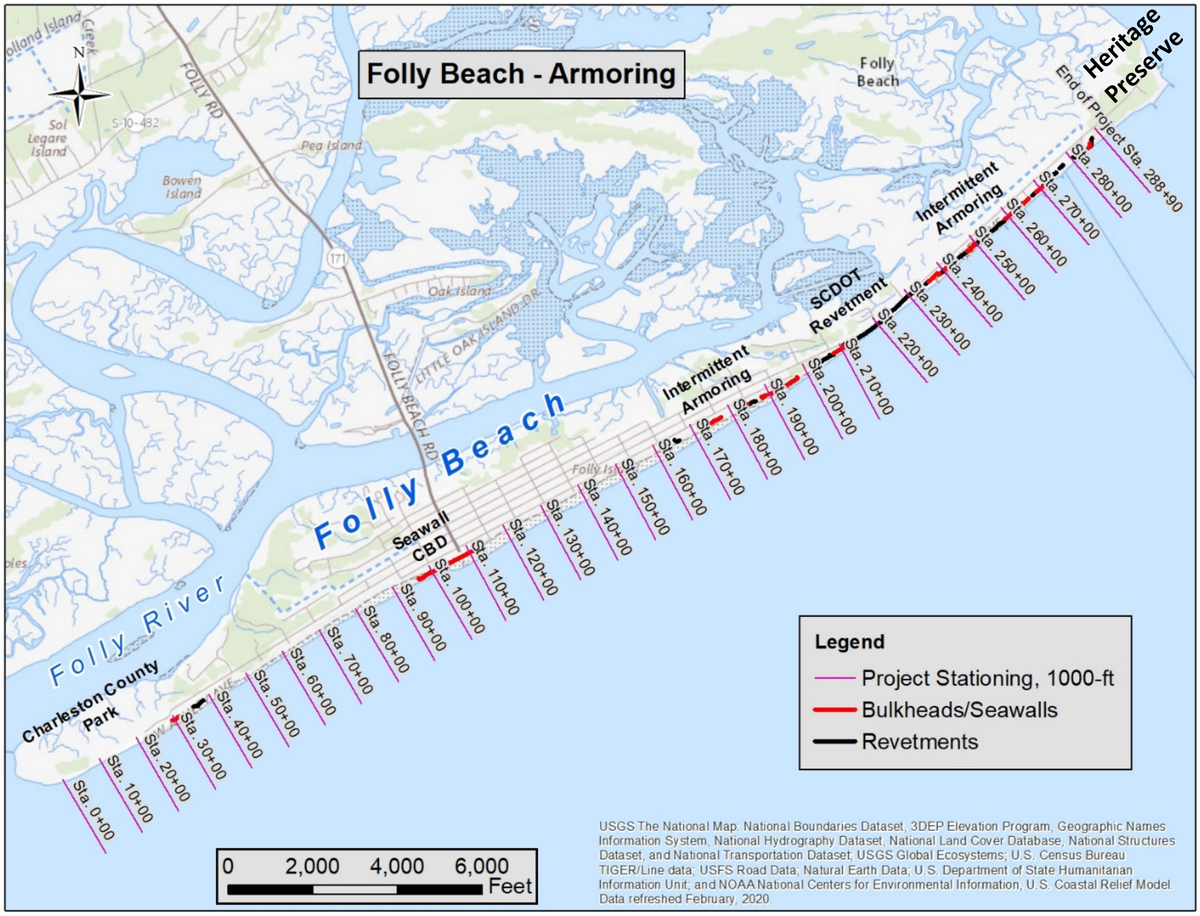


Figure 2-3. Existing Bulkhead, Seawalls and Revetment Sections

3 Natural Forces

3.1 Winds

Local winds are the primary means of generating the small-amplitude, short period waves that are an important mechanism of sand transport along the South Carolina shoreline. Winds in the project vicinity vary seasonally with prevailing winds ranging from the northeast though the southwest (in clockwise direction). The greatest velocities originate from the northeast quadrant in fall and winter months and from the southwest quadrant in the spring and summer.

Wind data offshore of the project area is available from the USACE Wave Information Study (WIS) Program. WIS hindcast data are generated using the numerical hindcast model WISWAVE (Hubertz, 1992). WISWAVE is driven by wind fields overlaying a bathymetric grid. Model output includes significant wave height, peak and mean wave period, peak and mean wave direction, wind speed, and wind direction. In the Atlantic, the available WIS hindcast database covers a 35-year period of record extending from 1980 to 2014.

WIS Station 63348 is representative of offshore deep water wind and wave conditions for the project area. Table 3-1 provides a summary of wind data from WIS Station 63348, located at latitude 32.58° N, longitude -79.67° W (about 17 miles east of Folly Beach; Figure 3-1). This table contains a summary of average wind speeds and frequency of occurrence broken down into eight 45 degree angle-bands. This table indicates that winds are predominantly from the southwest and northeast. The wind rose presented in Figure 3-2 provides a further breakdown of winds in the project area.

Table 3-1. Average Wind Conditions

Wind Direction (from)	WIS Station #63348 (1980 – 2014)	
	Percentage Occurrence (%)	Average Wind Speed (mph)
North	10.5	16.6
Northeast	17.8	16.6
East	9.8	12.2
Southeast	8.1	11.3
South	12.8	12.4
Southwest	21.1	14.1
West	10.8	15.7
Northwest	9.2	16.7

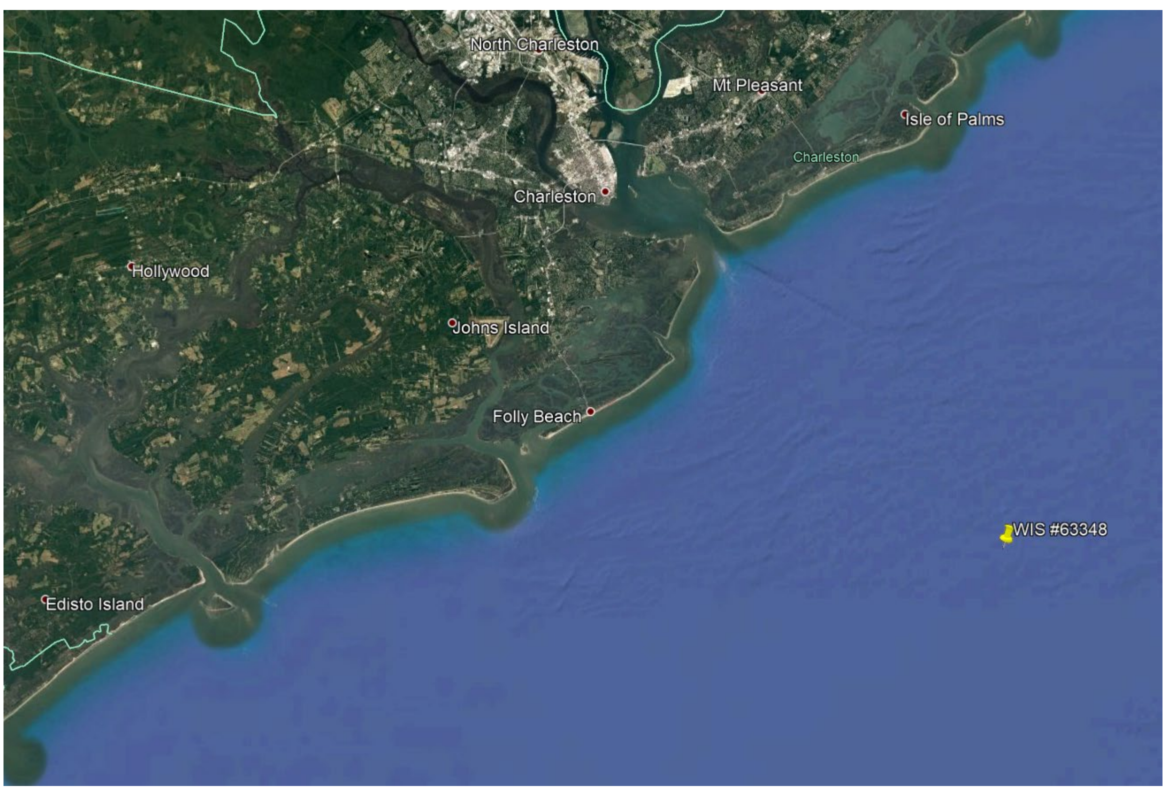


Figure 3-1. Location of WIS Station #63348 Relative to Project (Google Earth)

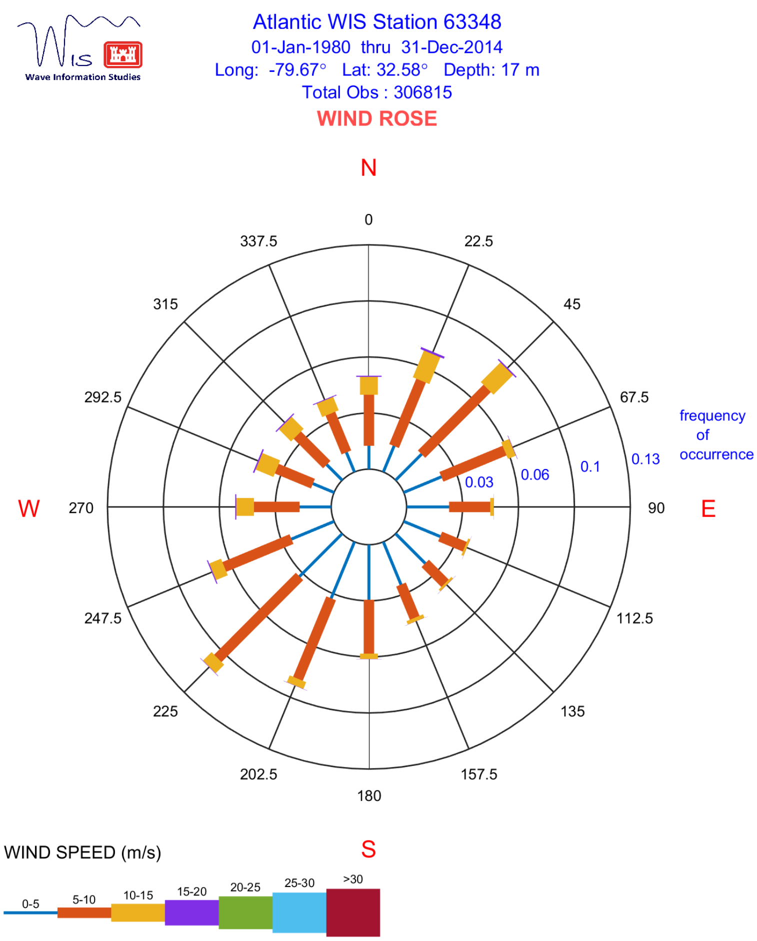


Figure 3-2. Wind Rose – WIS Station 63348 (USACE-ERDC)

Wind conditions in Coastal South Carolina are seasonal. A further breakdown of the wind data provides a summary of the seasonal conditions in Table 3-2. Between October and February, frontal weather patterns driven by cold Arctic air masses can extend into South Carolina. These fronts typically generate northeast winds before the frontal passage and northwest winds behind the front. Along much of the Atlantic coast "Northeaster" behavior is responsible for the increased intensity of wind speed in the northeast sector during the fall and winter months.

The summer months are characterized by southwest winds and tropical weather systems traveling west to northwest in the lower latitudes. Additionally, daily breezes onshore and offshore result from differential heating of land and water masses.

During the summer and fall months, tropical waves may develop into tropical storms and hurricanes, which can generate devastating winds, waves, and storm surge when they impact the project area. These storms contribute to the overall longshore and cross-shore sediment transport at Folly Beach. These intense seasonal events have an approximate recurrence interval of once every five years and will be discussed in greater detail under Section 3.4: Storm Effects.

Table 3-2. Seasonal Wind Conditions

Month	WIS Station #63348 (1980 – 2014)	
	Average Wind Speed (mph)	Predominant Direction (from)
January	17.4	NW
February	17.0	N
March	16.3	W
April	14.5	SW
May	12.8	SW
June	12.1	SW
July	12.1	SW
August	11.4	SW
September	13.2	NE
October	14.8	NE
November	16.3	NE
December	16.8	NE

3.2 Waves

The energy dissipation that occurs as waves enter the nearshore zone and break is an important component of sediment transport in the project area. Incident waves, in combination with tides and storm surge, are important factors influencing the behavior of the shoreline. The Folly Beach study area is exposed to both short period wind-waves and longer period open-ocean swells originating predominantly from the southeast.

Damage to the Folly Beach shoreline and upland development is attributable to large storm waves produced primarily by tropical disturbances, including hurricanes, during the summer and fall months, and by Northeasters during the late fall and winter months.

Wave data for this report were obtained from the long-term USACE WIS hindcast database for the U.S. Atlantic coast. This 35-year record extends from 1980 through 2014 and consists of a time-series of wave events at 3-hour intervals for stations located along the east coast. Similar to wind conditions, wave conditions in coastal South Carolina experience seasonal variability. The seasonal breakdown of wave heights is shown in Table 3-3.

Table 3-4 summarizes the percentage of occurrence and average wave height of the WIS waves by direction. It can be seen that the dominant wave direction is from the southeast, 83% of the waves are from between 90° and 180°. This can be seen in greater detail in the wave rose presented in Figure 3-3. The total wave climate reflects both the open-ocean swell and more locally generated wind-waves. Waves from the southwest quadrant are refracted by Stono Inlet ebb shoal and Kiawah Island.

Table 3-3. Seasonal Wave Conditions

Month	WIS Station #63348 (1980-2014)		
	Average Wave Height	Predominant Direction	Mean Period
	(ft)	(from)	(sec)
January	3.9	SE	8.6
February	3.9	SE	8.6
March	3.9	SE	8.1
April	3.6	SE	7.9
May	3.3	SE	8.1
June	3.0	SE	8.2
July	3.0	SE	8.3
August	3.0	SE	8.4
September	3.9	E	8.9
October	3.9	E	8.3
November	3.9	E	8.6
December	3.9	E	8.6

Table 3-4. Average Wave Heights (1980 to 2014)

Wave Direction (from)		WIS Station #63348 (1980-2014)	
		Percentage Occurrence	Average Significant Wave Height
	(degrees)	(%)	(ft)
N	0	0.5%	3.0
NNE	22.5	0.7%	3.3
NE	45	1.7%	3.6
ENE	67.5	6.4%	4.3
E	90	16.8%	3.9
ESE	112.5	29.8%	3.3
SE	135	22.6%	3.6
SSE	157.5	8.8%	3.6
S	180	5.3%	3.9
SSW	202.5	2.8%	3.9
SW	225	1.7%	3.9
WSW	247.5	0.9%	3.6
W	270	0.6%	3.6
WNW	292.5	0.5%	3.3
NW	315	0.5%	3.3
WNW	337.5	0.4%	3.0

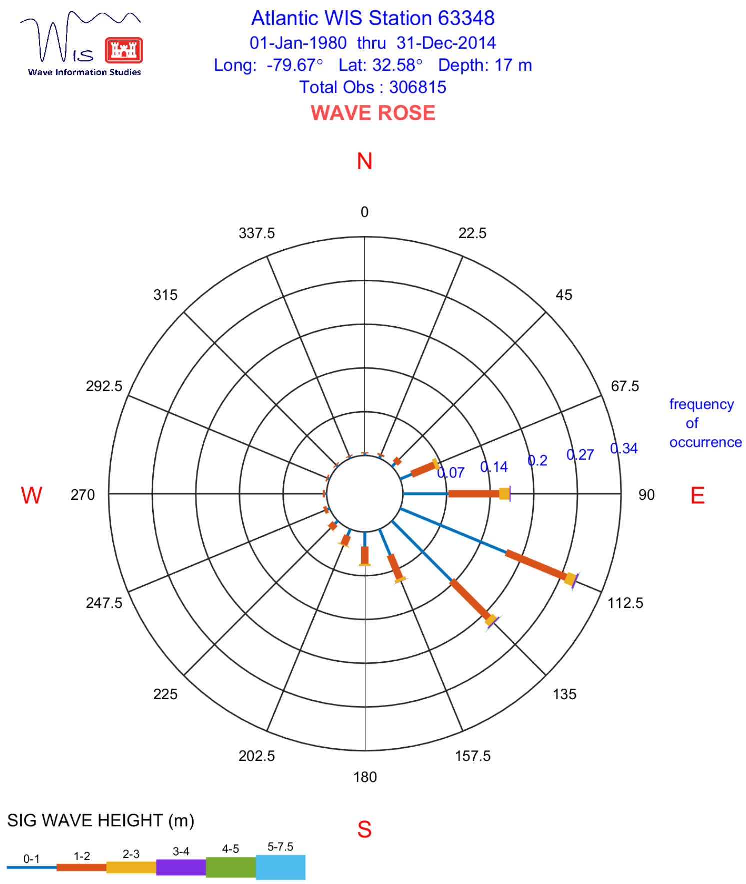


Figure 3-3. Wave Rose – WIS Station 63348 (USACE-ERDC)

3.3 Tides

Astronomical tides are created by the gravitational pull of the moon and sun and are predictable in magnitude and timing. The National Oceanic and Atmospheric Administration (NOAA) publishes tide tables for selected locations along the coastlines of the Unites States and locations around the world. These tables provide times of high and low tides, as well as predicted tidal amplitudes.

Tidal datums for the Folly Beach project site were obtained from NOAA tide station #8665530 Charleston, SC. Tidal ranges and datums are summarized in Figure 3-4 and Table 3-5. Mean high water (MHW) is at +2.26 ft NAVD88 and mean low water (MLW) is at -2.96 ft NAVD88 for a mean tide range of 5.22 ft in the project area. The record high water level was 9.38 ft NAVD88 during Hurricane Hugo on 22Sep1989. A temporary NOAA tide gage (Station #8666467) was available on the Folly River from 01Feb1977 to 31Jan1978. The mean tide range for the Folly River gage for that period was 5.38 ft.

Station Homepage: https://tidesandcurrents.noaa.gov/stationhome.html?id=8665530

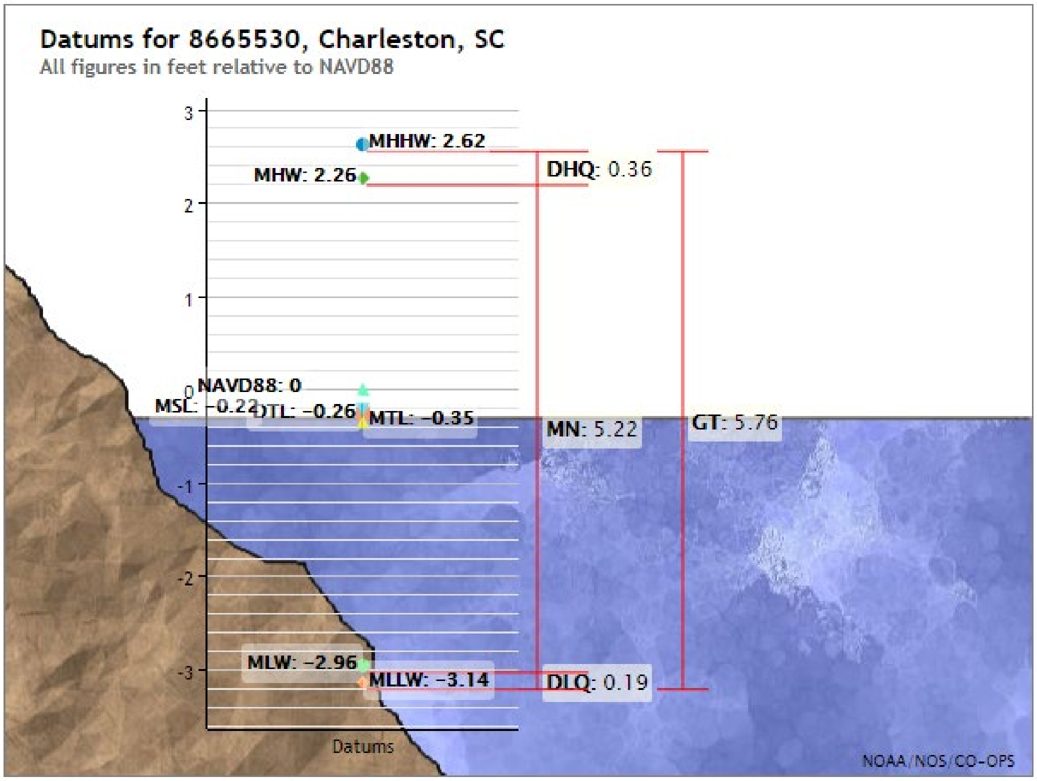


Figure 3-4. Tidal Range for NOAA Charleston Gage

Table 3-5. Tidal Datums

Tidal Datum	Elevation Relative to NAVD88 (feet)
Mean Higher High Water (MHHW)	2.62
Mean High Water (MHW)	2.26
North American Vertical Datum (NAVD88)	0.00
Mean Tide Level (MTL)	-0.35
Mean Low Water (MLW)	-2.96
Mean Lower Low Water (MLLW)	-3.14

3.4 Storm Effects

The shoreline of Folly Beach is influenced predominantly by tropical systems which occur during the summer and fall and Northeasters during the late fall and winter. Although hurricanes typically generate larger waves and storm surge, northeasters also impact the shoreline because of their longer duration and higher frequency of occurrence.

During intense storm activity, the shoreline is expected to naturally modify its beach profile. Storms erode and transport sediment from the beach into the active zone of storm waves. Once caught in the waves, this sediment is carried along the shore and re-deposited farther down the beach or is carried offshore and stored temporarily in submerged sand bars. Hurricanes and coastal storms, with high energy breaking waves and elevated water levels, can change the width and elevation of beaches and accelerate erosion. After storms pass, lower energy waves usually return sediment from the sand bars to the beach, which is restored gradually to its natural shape. While the beach profile typically recovers from storm energy as described, extreme storm events may cause sediment to leave the beach system entirely, sweeping it into inlets or far offshore into deep water where waves cannot return it to the beach. Therefore, a portion of shoreline recession due to intense storms may never fully recover.

Folly Beach is located in an area of significant hurricane activity. Figure 3-5 shows historic tracks of hurricanes and tropical storms from 1853 to 2019, as recorded by the National Hurricane Center (NHC) and is available from NOAA Office for Coastal Management. The dashed circle in the center of this figure indicates a 50-nautical mile radius from Folly Beach. Based on NHC records 67 hurricanes and tropical storms have passed within this 50-mile radius over the 166-year period of record. The 50-mile radius was chosen because any tropical disturbance passing within this distance would be likely to produce some damage along the shoreline. Stronger storms are capable of producing shoreline damage from greater distances.

Hurricane Hugo made landfall north of Charleston on September 22, 1989 as a Category 4 and was the costliest storm event in South Carolina history. Folly Beach experienced sustained winds of 85 mph and gust of 107 mph (FEMA, 2004). Another storm of interest is Hurricane Gracie which made landfall south of Folly Beach in September 1959 as a Category 4. In recent years, a number of named storms have significantly impacted the project area, including Florence (2018), Matthew (2016), Bonnie (2016), Ana and Joaquin (2015), and Beryl (2012). Damages from these storms, as well as from more distant storms causing indirect impacts, included substantial erosion and damage from winds, waves and elevated water levels. The storm suite did include storms greater than the historic event by the process of peaking the tide and phase of the historic event to produce the maximum storm surge possible. This peaking increased multiple storms above the 1% annual chance of exceedance.

There is concern that climate change is increasing the frequency and intensity of tropical storm events within the Atlantic Basin with the potential of increasing erosion along the eastern United States shorelines. The USACE Engineering Regulation (ER) No. 1100-2-8162 '*Incorporating Se Level Change in Civil Works Programs'* addresses the issue of future storm events and summarizes recent research. Four excerpts from the ER are provided below. The ER concludes that the science is inconclusive at this time as to if storms are increasing in frequency and intensity. The Folly Beach analyses did not include an increase future storm events. As with addressing relative sea level rise at Folly Beach, the potential for an increase in storm activity will be address in an adaptive management approach during the Planning, Engineering and Design phase of the study. Monitoring the impacts of climate change at Folly Beach will be coordinated with other regional CSRM projects including the Charleston Peninsula Study and the Edisto Island.

(1) Determining the effects of climate change on individual storms and on statistical descriptions of storm distributions is difficult because of the relatively small number of storms and the analytic problem of associating changes in measurements of storms with a few, very large-scale climate changes in basins around the world.

(2) At this time, no certain effects of climate change on tropical cyclone (TC) activity in terms of frequency, intensity, and rainfall across all global basins have been identified as changes to the variability of TC activity expected from natural causes (Knutson et al., 2010). As a result, the current science related to climate effects on TC activity relevant to the United States (U.S.) has

not reached the point of standard consensus necessary to inform a change in storm analysis baselines.

(3) In the Atlantic Basin including the Gulf of Mexico, the unadjusted record of raw hurricane counts in the best track Atlantic hurricane database (HURDAT) maintained by NOAA shows an increase since the early 1900s (Vecchi and Knutson, 2011). But when the record is adjusted for storms likely missed in the pre-satellite era before the mid-1970s, no significant increase can be seen since the late 1800s (Vecchi and Knutson, 2011). Also, the number of U.S. landfalling hurricanes since the late 1800s has not significantly changed (Vecchi and Knutson, 2011).

(4) Concerning the projected future effects of climate change on TC activity in all global basins (including the Eastern and Central North Pacific), the World Meteorological Organization (WMO) TC Expert Team (Knutson et al., 2010) concluded, based on atmospheric theory and high-resolution models, that by the late 21st century the number of tropical cyclones could remain at current levels or decrease by up to one-third; that average TC intensity could increase by up to 10%; and that near-storm (~50mi radius) rainfall rates could increase by ~20%.

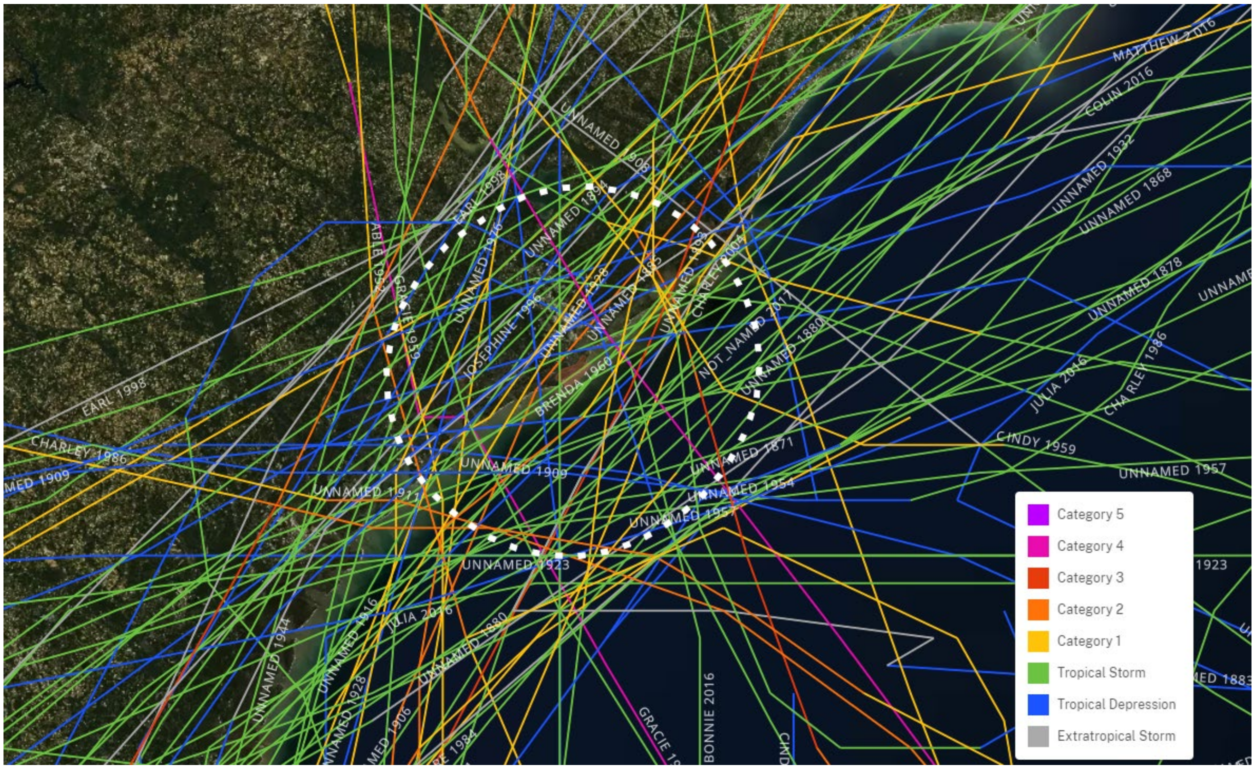


Figure 3-5. Hurricanes and Tropical Storm Tracks (1853 – 2019, 50 NM radius)

3.5 Storm Surge

Storm surge is defined as the rise of the ocean surface above its astronomical tide level due to storm forces. Surges occur primarily as a result of atmospheric pressure gradients and surface stresses created by wind blowing over a water surface. Strong onshore winds pile up water near the shoreline, resulting in super-elevated water levels along the coastal region and inland waterways. In addition, the lower atmospheric pressure which accompanies storms also contributes to a rise in water surface elevation. Extremely high wind velocities coupled with low barometric pressures (such as those experienced in tropical storms, hurricanes, and very strong Northeasters) can produce high, damaging water levels. In addition to wind speed, direction and duration, storm surge is also influenced by water depth, length of fetch (distance over water), and frictional characteristics of the nearshore sea bottom. An estimate of storm surge is required for the design of dune crest elevations. An increase in water depth may increase the potential for coastal flooding and allow larger storm waves to attack the shore.

Due to sand management over the life of the project within the dune (sand fencing and planting) the existing condition dune system along the Folly Beach study area varies from no dune to dunes between elevations 9 and 18 feet NAVD88 and is susceptible to overtopping from extreme storm surges. This can be seen from 3-6 which provides total storm surge levels vs storm frequency along Folly Beach and was obtained from the Charleston County Federal Emergency Management Agency (FEMA) Preliminary Flood Insurance Study (FIS) (FEMA, 2016). The total storm tide includes storm surge, wave setup and astronomical high tide. The record combined surge and peak wave height of 13 to 14 ft NAVD88 occurred at Folly Beach during Hurricane Hugo in 1989 (FEMA, 2004).

Table 3-6. Storm Tide and Frequency along Folly Beach (FEMA, 2016)

Annual Chance (%)	Return Period (Years)	Storm Surge Elevation (Feet, NAVD88)
10	10	5.5
2	50	7.5
1	100	10.0
0.2	500	13.5

3.6 Depth of Closure

The seaward limit of changes in depth over long-time periods due to movement of sediment is referred to as the “closure depth” and this depth is used for several calculations in the coastal analysis. The depth of closure along the Folly Beach shoreline varies from -9.0 ft NAVD88 at the ends of the island and -11.5 ft NAVD88 along the center of the island (Ebersole et al, 1996). The 2001 monitoring report of the 1993 nourishment project used a closure depth of -10.2 feet NAVD88 for the entire project length in calculating volume changes (CSE, 2001) and was used for this study.

3.7 Sea Level Change

Sea level change (SLC) at Folly Beach was evaluated following the guidelines presented in USACE Engineer Pamphlet EP 1100-2-1 “Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation” (30Jun2019). The purpose of the EP was to provide instructional and procedural guidance to analyze and adapt to the direct and indirect physical and ecological effect of projected sea level change on USACE projects and systems of projects needed to implement Engineer Regulation (ER) 1100-2-8162.

ER 1100-2-8162 “Incorporating Sea Level Change in Civil Works Programs” (31Dec2013) provides both a methodology and a procedure for determining a range of SLC estimates based on global sea level change rates, the local historic sea level change rate, the construction (base) year of the project, and the design life of the project. Three estimates are required by the guidance, a Low (Baseline) estimate representing the minimum expected SLC, an Intermediate estimate, and a High estimate representing the maximum expected SLC. The guidance will be used to evaluate the future sea levels, the impacts to the Folly Beach project during a 50-Year period and to assess the risk associated with the SLC estimates.

The first step in evaluating sea level change at Folly Beach was to identify a near-by NOAA water level gage with a sufficiently long data record. The analysis was based on the NOAA tide gauge located in Charleston, South Carolina (Station #8665530), approximately 8 miles north of Folly Beach. The gage is compliant and active with a historic record of 1901 to present, there was a data gap from 1905 to 1924. From Figure 3-6 the linear relative sea level trend for this gauge is 3.26 mm/year (0.01070 ft/year) with a 95% confidence interval of +/- 0.19 mm/year (0.00062 feet/year) based on monthly mean sea level data. For the 50-year analysis of 2024 to 2074 this is equivalent to an increase of 0.54 ft in sea level. For stations with sufficient historical data the linear relative sea level trends were calculated by NOAA in overlapping 50-year increments. The variation on each 50-year trend is provided in Figure 3-7. The variation of each 50-year trend, with 95% confidence interval, is plotted against the mid-year of each 50-year period. The solid horizontal line represents the linear relative sea level trend using the entire period of record. Cyclical trends in the sea level data can be noted in both figures.

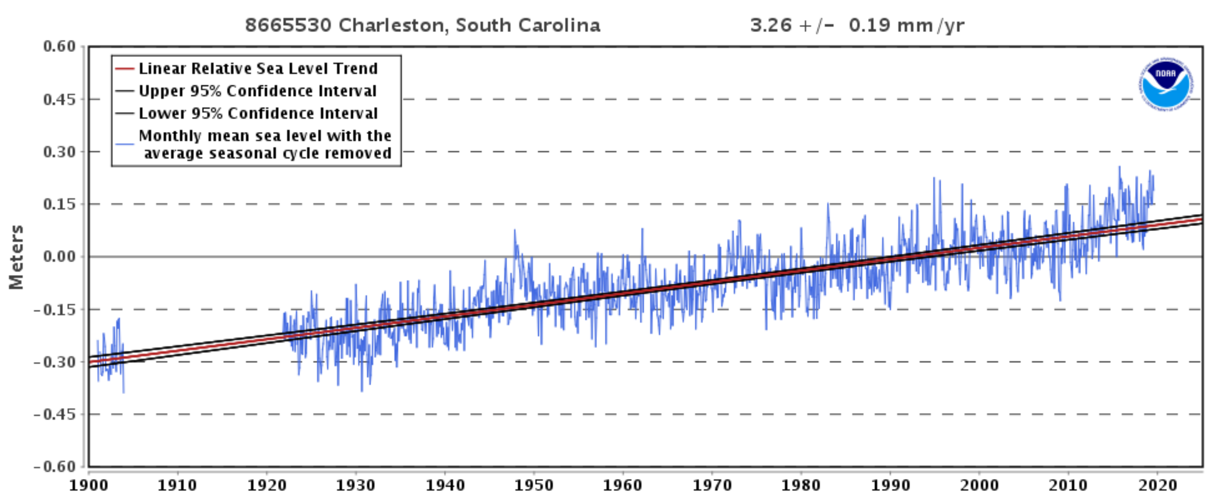


Figure 3-6. Relative Sea Level Trend, NOAA Gauge 8665530

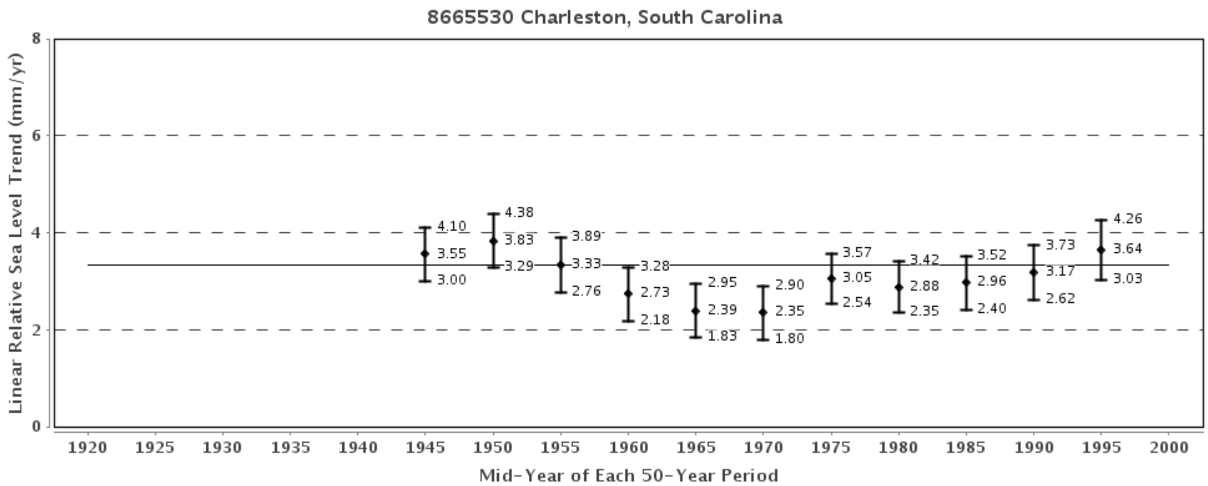


Figure 3-7. Variation of 50-Year Relative Sea Level Trend, NOAA Gauge 8665530

The second step in evaluating SLC at Folly Beach was to assess future trends, mainly will the rate of sea level rise accelerate in the future. Any future increase or decrease in this long-term trend along with land subsidence and glacial rebound needs to be addressed throughout the 50-year period.

The USACE online tool Sea Level Tracker was used to determine the current rate of SLC observed and the projected future trends in the rate of SLC, a link to the tool is provided below. Extreme water levels (EWL) incorporated into the tool are based on statistical probabilities using recorded historic monthly extreme water level values. The Sea Level Tracker is used to compare actual mean sea level (MSL) values and trends for specific NOAA tide gauges with the USACE SLC scenarios as described in ER 1100-2-8162 and Engineer Pamphlet (EP) 1100-2-1. The Sea Level Tracker tool calculates the USACE Low, Intermediate and High sea level change scenarios based on global and local change effects. Historical MSL is represented by either 19-year or 5-year midpoint moving averages. Guidance in using the Sea Level Tracker and technical background is provided in USACE "Sea Level Tracker User Guide", Version 1.0, December, 2018.

https://climate.sec.usace.army.mil/slr_app/

The Sea Level Tracker tool was used to evaluate the NOAA Charleston tide gauge data. The regionally corrected rate of 0.00965 ft/yr was used as the rate of SLC and was sourced from Technical Report NOS CO-OPS 065 (NOAA, 2013) and accounts for vertical land motion. This regional rate is also the Low USACE estimated SLC rate. Based on the regional rate only, the sea level increase was 0.48 ft during the 50-year period of 2024 to 2074. Figure 3-8 presents the results of the Tracker tool focused on trends between 1990 to 2020. The light blue line represents the 5-year moving average and the heavy dark blue line represents the 19-year moving average. The 19-year average is useful in that this represents the moon's metonic cycle and the tidal datum epoch. These estimates are referenced to the midpoint of the latest National Tidal Datum epoch, 1992. The reader is referred to ER 1100-2-8162 for a detailed explanation of the procedure, equations employed and variables included to account for the eustatic change as well as site specific uplift or subsidence to develop corrected rates. The red line is the High SLC prediction, the green is the Intermediate and the blue is the Low rate prediction. From Figure 3-8 it can be noted that the 19-year moving average tracks well with the intermediate rate. The 5-year rate is tracking upwards but is cyclical and does not match the tidal epoch period of 19-years.

The future USACE sea level predictions for the Folly Beach project based on the Charleston gauge are provided in Figure 3-9. For the 2024 to 2074 period the predicted Low rate sea level rise (regional rate) is 0.52 ft, the Intermediate SLC increase was 1.03 ft and the High SLC increase was 2.63 ft. Table 3-7 includes a summary of the USACE SLC estimates and for comparison the regionalized NOAA estimates (NOAA et al, 2012) are also provided. For the 100-year period of 2024 to 2074 the USACE sea level rise for the Low rate was 1.03 ft, Intermediate was 2.49 ft and the High was 7.11 ft.

Table 3-7. USACE and NOAA 50-Year Sea Level Change Estimates

Project Year	Year	USACE			NOAA			
		Low	Int	High	Low	Int-Low	Int-High	High
Base	2019	0.06	0.12	0.33	0.04	0.11	0.25	0.41
Start	2024	0.11	0.20	0.49	0.09	0.18	0.38	0.61
	2034	0.21	0.37	0.87	0.19	0.34	0.69	1.09
	2044	0.32	0.56	1.32	0.28	0.52	1.05	1.66
	2054	0.42	0.76	1.85	0.38	0.72	1.48	2.34
	2064	0.52	0.99	2.45	0.47	0.94	1.96	3.12
End	2074	0.63	1.23	3.12	0.57	1.17	2.49	4.01
	2084	0.73	1.48	3.87	0.73	1.48	3.15	5.05
	2094	0.83	1.76	4.69	0.83	1.76	3.81	6.15
	2104	0.94	2.05	5.59	0.94	2.05	4.52	7.35
	2114	1.04	2.36	6.56	1.04	2.36	5.29	8.64
	2124	1.14	2.69	7.60	1.14	2.69	6.12	10.05
50-Year Increase =		0.52	1.03	2.63	0.48	0.99	2.11	3.40
100-Year Increase =		1.03	2.49	7.11	1.05	2.51	5.74	9.44

To compare the predicted Charleston USACE SLC trends with near-by NOAA gauges, the tide gauges at Springmaid Pier (#8661070) in Myrtle Beach, SC and the Ft. Pulaski (#8670870) near Tybee Island, GA were reviewed. The 1990 to 2020 SLC trends with the 19-year and 5-year moving averages are provided in Figures 3-10 and 3-11. Both gages are active and compliant with over 40-years of data. The Ft. Pulaski gauge shows the same trends as the Charleston gauge with the 19-year moving average tracking well with the Intermediate rate and the 5-year average rising. For the Springmaid gauge the 19-year and 5-year moving averages are below the Low SCL curve but both are sloping upwards.

The USACE Intermediate SLC scenario was selected for the Folly Beach project because it tracked well with the 19-year moving average in Figure 3-8. The USACE predicted Intermediate rate was also selected for the Charleston Peninsula Coastal Storm Risk Management Feasibility Study. Similar SLC trends were noted at regional tide gauges. The Intermediate rate was also selected in coordination with the USACE Climate Preparedness and Resilience Community of Practice.

Figure 3-9 also includes design details of the recommended plan as presented in Section 5 of this report. The initial construction of the beachfill is planned for 2024 along with the three proposed renourishments. The plan includes a combination berm and dune with the dune crest elevation set at elevation 15.0 ft NAVD88. The dune crest elevation was based on the storm suite used in the Monte Carlo Beach-fx analysis using the Intermediate SLC. The storm surge of the largest storm events used in the storm suite approximates the 2% annual exceedance probability elevation as provided in Table 3-6. The project design loading for the dune will be exceeded for the Intermediate SLC in year 2074. Based on the High SLC rate the project design loading will be exceeded earlier in approximately year 2048. Reference elevations for the roadways and existing dune line in also provided. The causeway Hwy 171 connecting Folly Beach to the mainland has a varying elevations and is at elevation 7.0 ft NAVD88 at Folly Beach Island.

The FEMA Base Flood Elevation (BFE), defined as the 1% Annual Exceedance Probability (AEP) Flood, is the regulatory requirement for the elevation or floodproofing of structures and are referenced to FEMA panels and transects, see Figure 3-12. BFE at Folly Beach varies along the shoreline and averages about elevation 10 ft NAVD88 (Section 3.5: Storm Surge). The BFE plotted relative to relative sea level change, see Figure 3-13. As a reference the tidal datums and extreme water levels (including the BFE) for the Charleston Gauge #8665530 are shown in Figure 3-14.

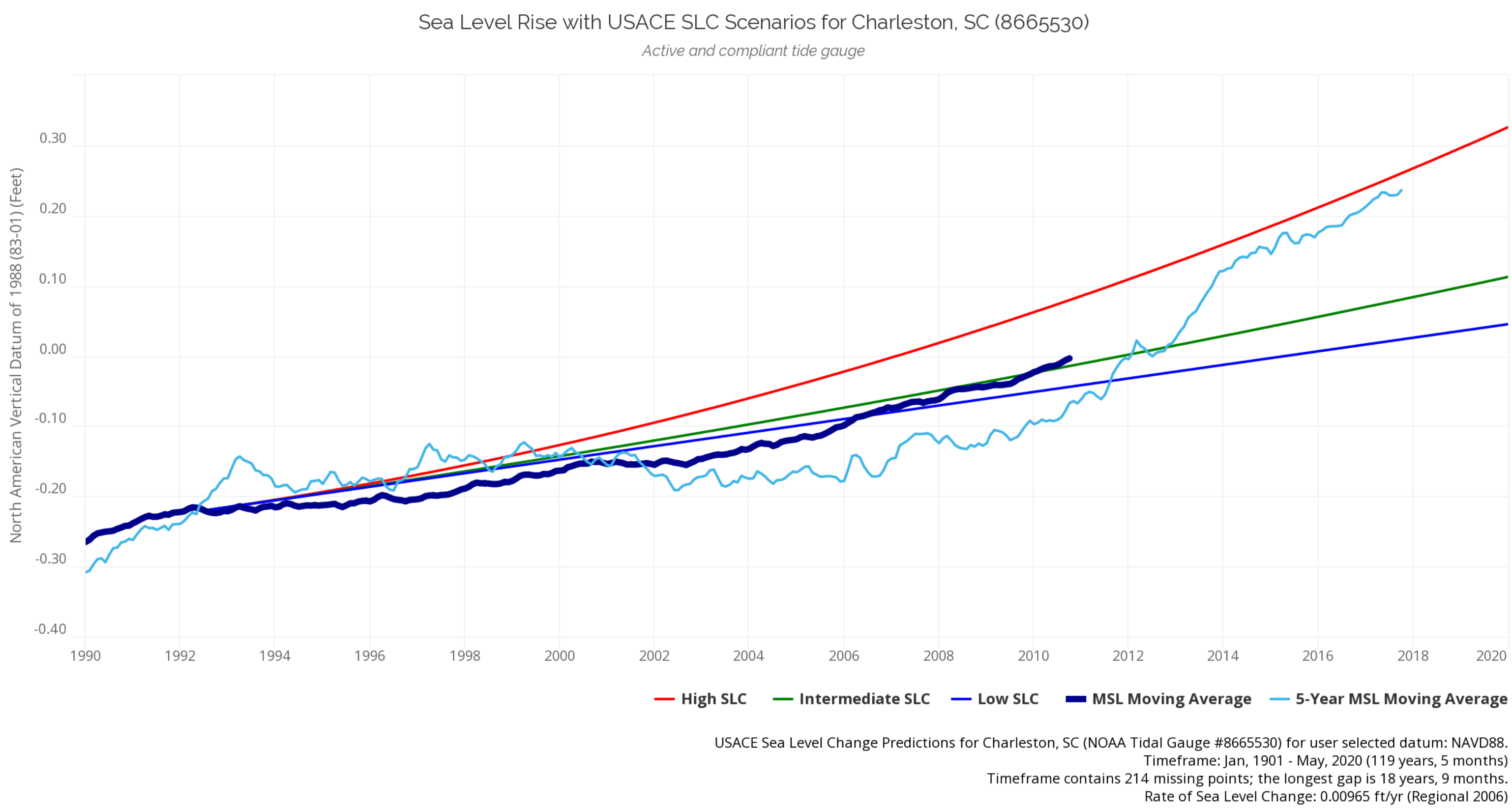
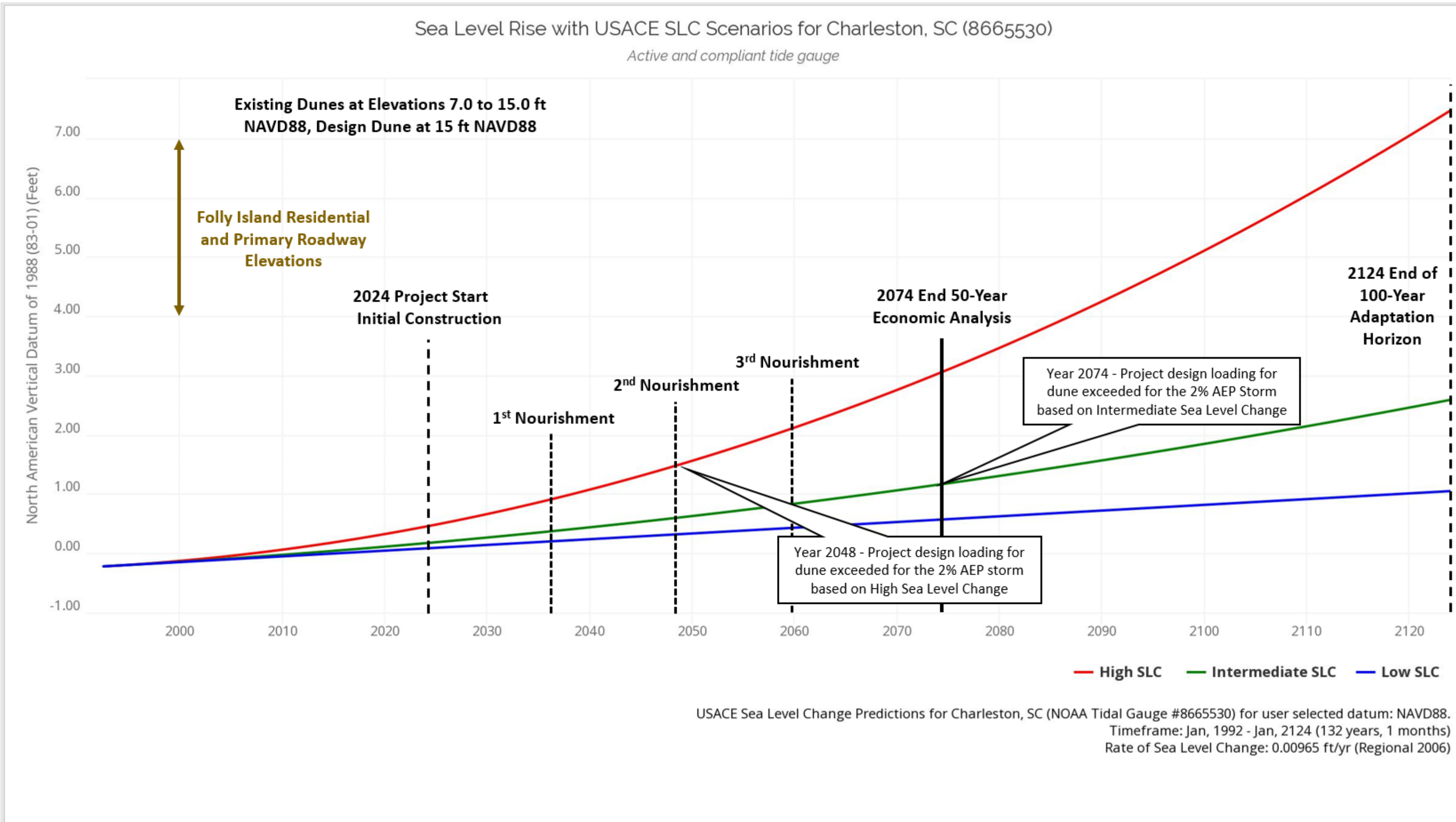


Figure 3-8. Charleston NOAA Gauge #8665530 SLC with 19-Year and 5-Year Moving Average

Figure 3-9. USACE Sea Level Change Predictions, 2024 to 2124



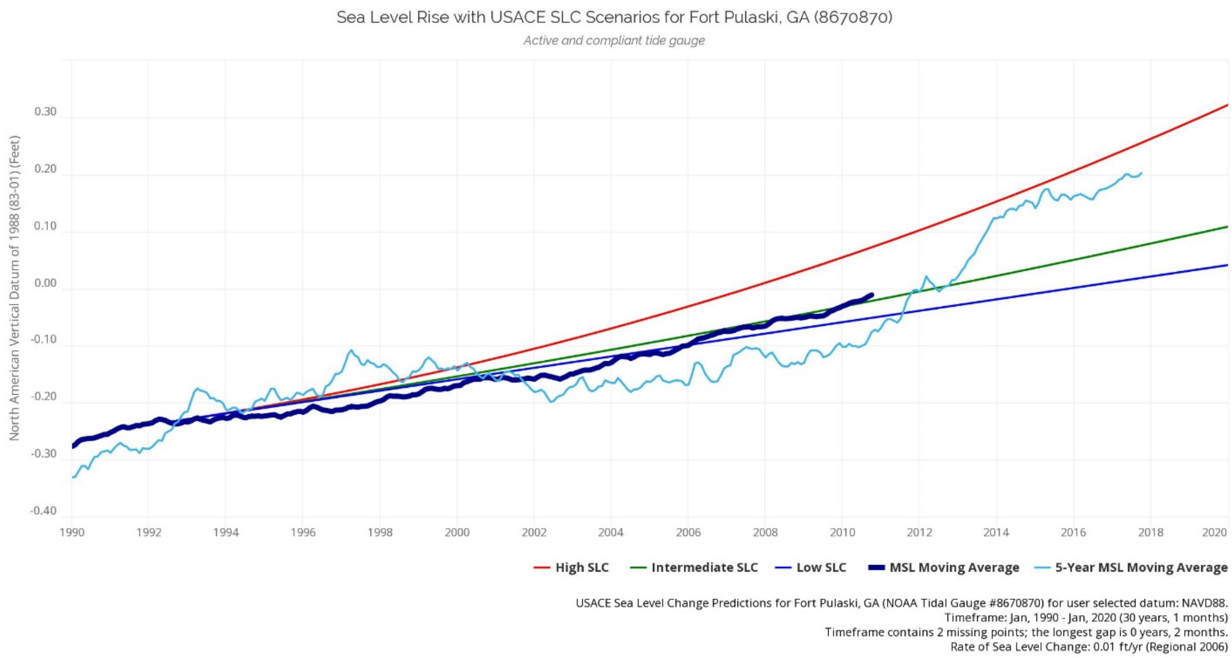


Figure 3-10. Ft. Pulaski, GA NOAA Gauge #8670870 SLC with 19-Year and 5-Year Moving Average

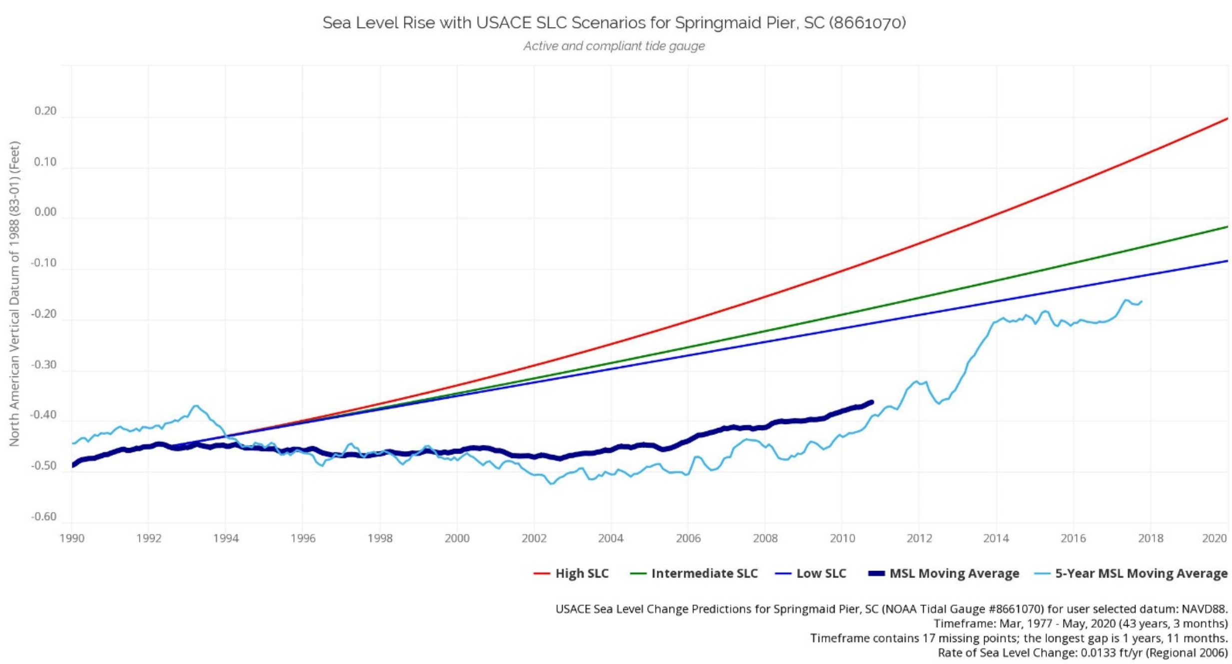


Figure 3-11. Springmaid Pier, SC NOAA Gauge #8661070 SLC with 19-Year and 5-Year Moving Average

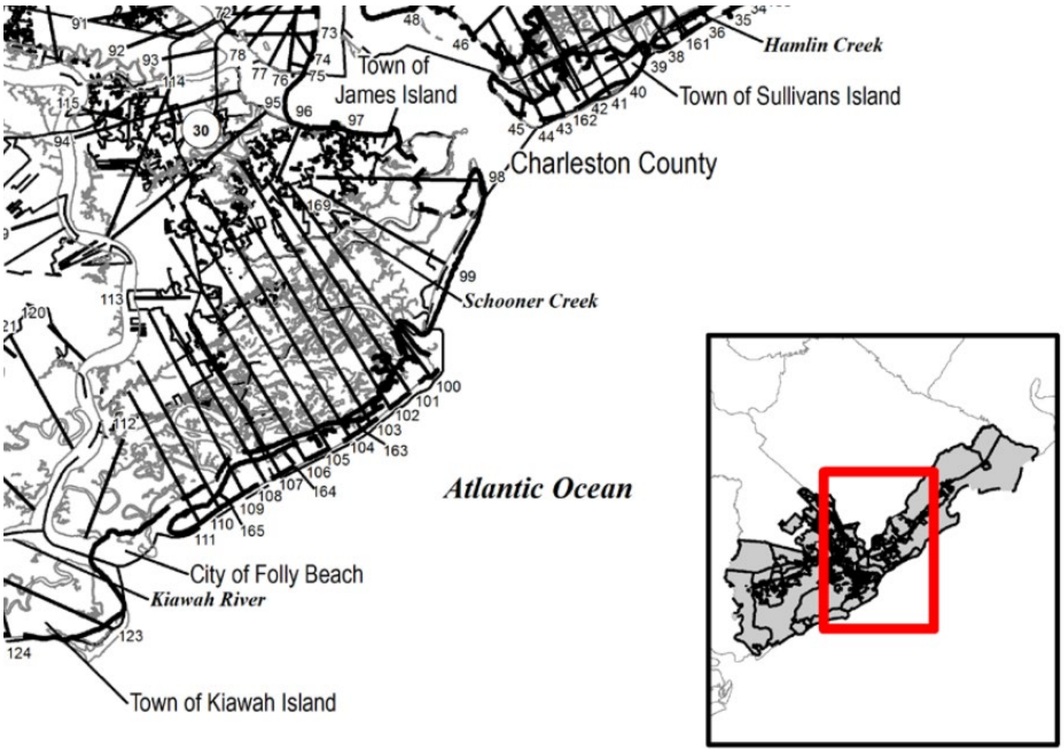


Figure 3-12. FEMA Flood Insurance Study Transect Location for Folly Beach

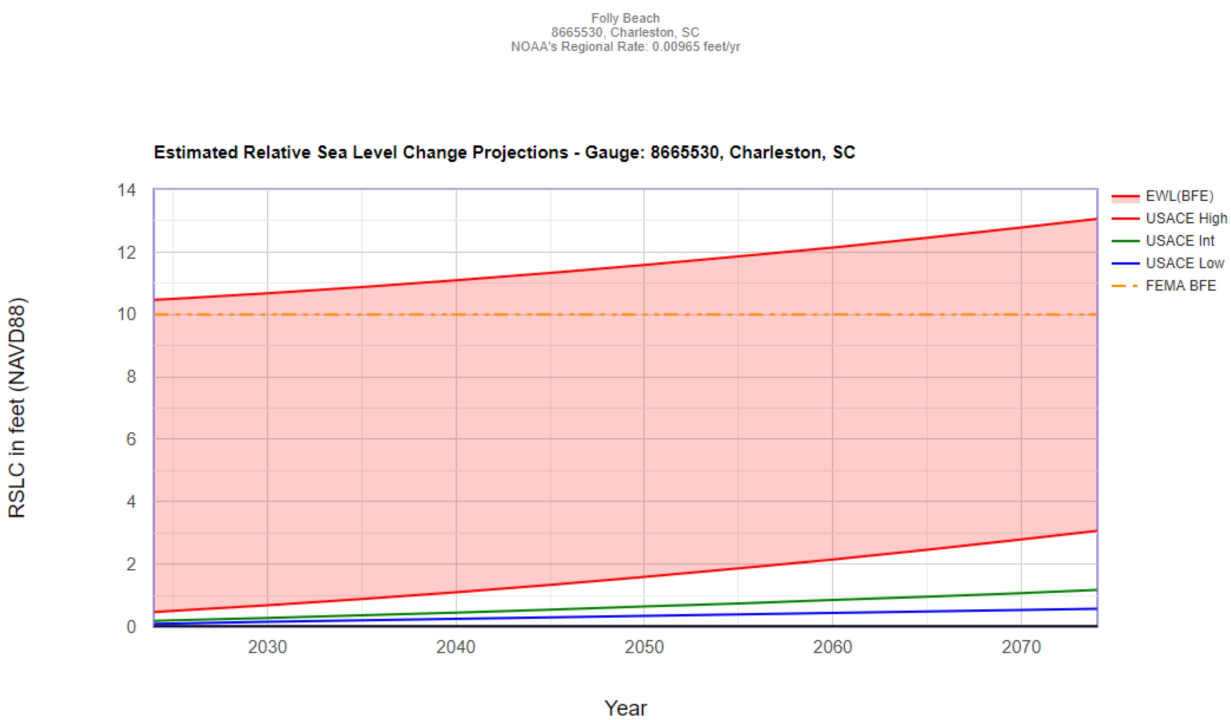


Figure 3-13. Estimated Relative Sea Level Change with BFE

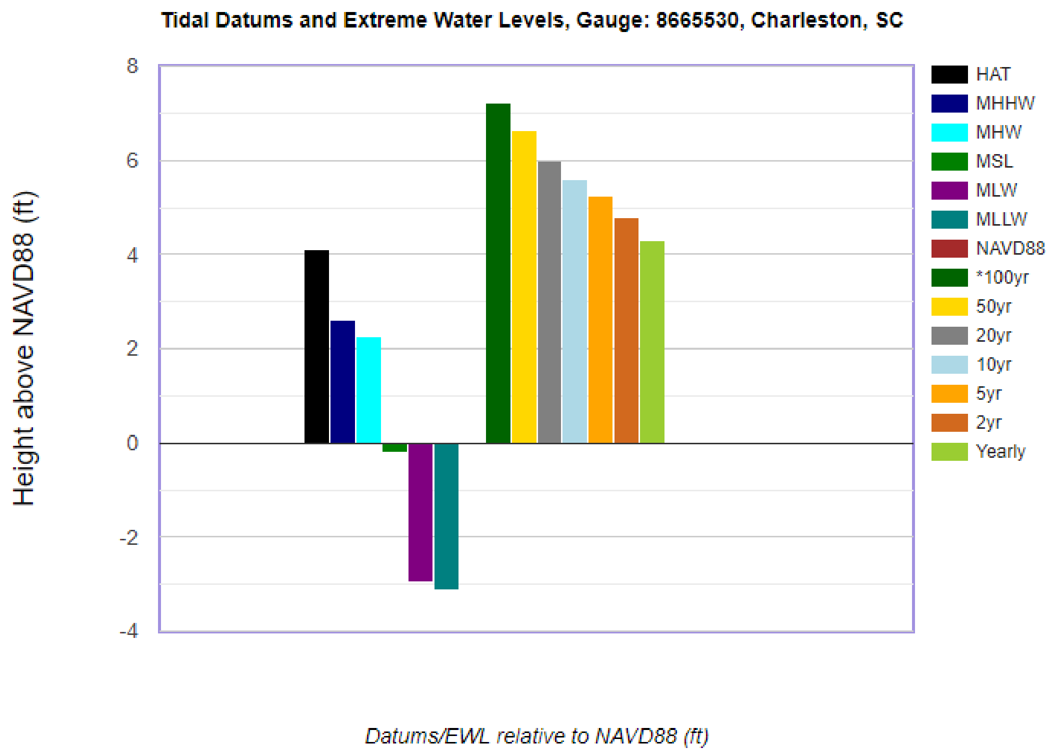


Figure 3-14. Tidal Datums and Extreme Water Levels

3.8 Storm Tide and Back Bay Flooding

Potential impacts of rising sea level on total water levels experienced at the site include overtopping of waterside structures, increased shoreline erosion, and flooding of low lying areas. Three cross-sections were drawn along the Folly Beach project site to determine elevations across the island, see Figure 3-15. Elevations at each transect are plotted with the BFE as well as the 2%, and 0.2% AEP water elevations for the High SLC scenario (+2.58 ft) at the end of the 50 years, see Figures 3-16, 3-17 and 3-18. The figures indicate that for existing conditions most of Folly Beach is currently susceptible to flooding during the 1% AEP and 0.2% AEP. The northeast end of the island is below elevation 10.0 ft NAVD88 and is susceptible for flooding during all flood events. A more detailed study of the vulnerability of Folly Island to potential sea level rise for the 50-year and 100-year time periods and critical thresholds is provided in Sub-Appendix A.

Relative vulnerability to back-bay flooding during extreme events is consistent between both with and without project conditions. The Beach-fx model incorporated back bay flooding by using the peak surge levels present on the oceanside along the rear of the economic reaches to ensure there was no double counting of structure or damage element cost or benefits.

The Folly Beach CSRM report does not address or reduce the risk of flooding caused by the expected gradual increase in sea level over the next 50-year and 100-year periods. Flooding of low-lying areas along the oceanside and back bay of Folly Island will increase over time during non-storm event conditions, particularly during monthly spring high tides or king tides conditions. The analysis for the Recommended Plan presented in this report does incorporate the Intermediate Sea Level Change with the Storm Suite over the 50-years of model simulations.

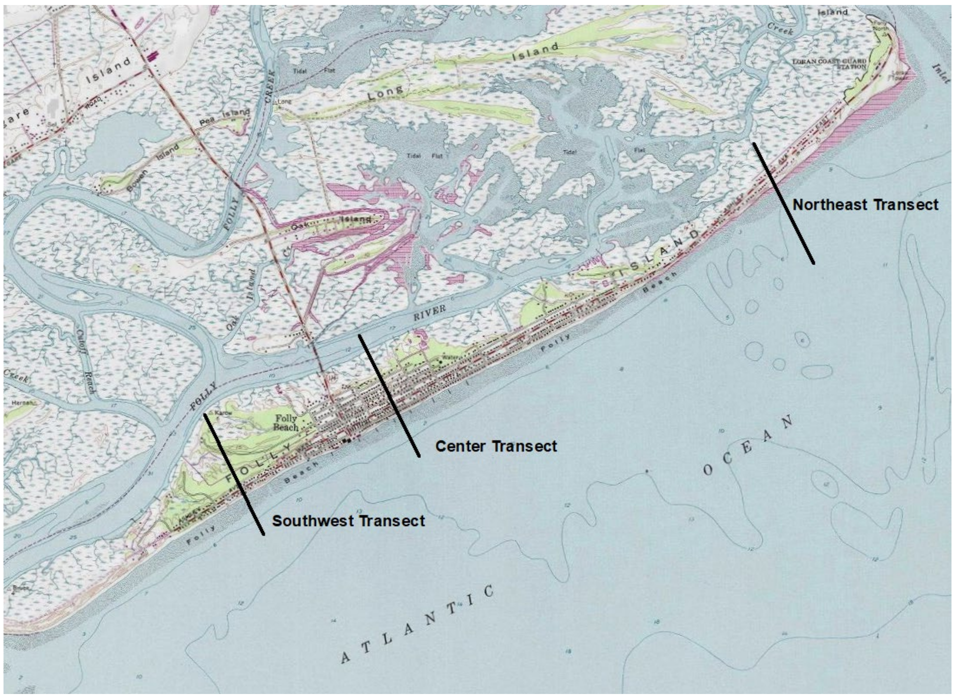


Figure 3-15. Folly Beach Elevation Transects

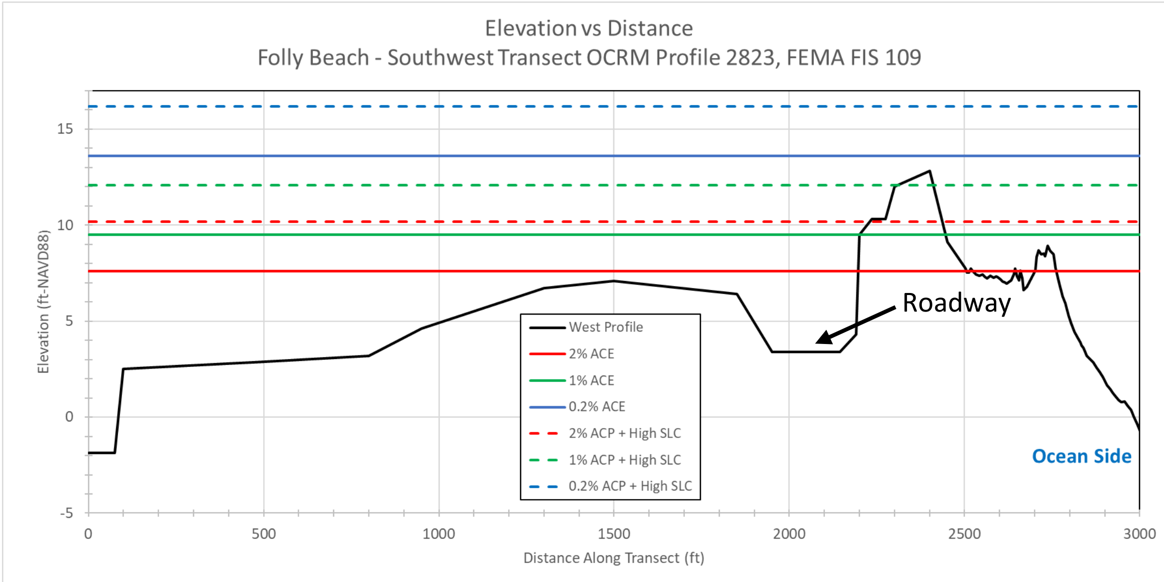


Figure 3-16. Land and AEP Elevations – Southwest Transect

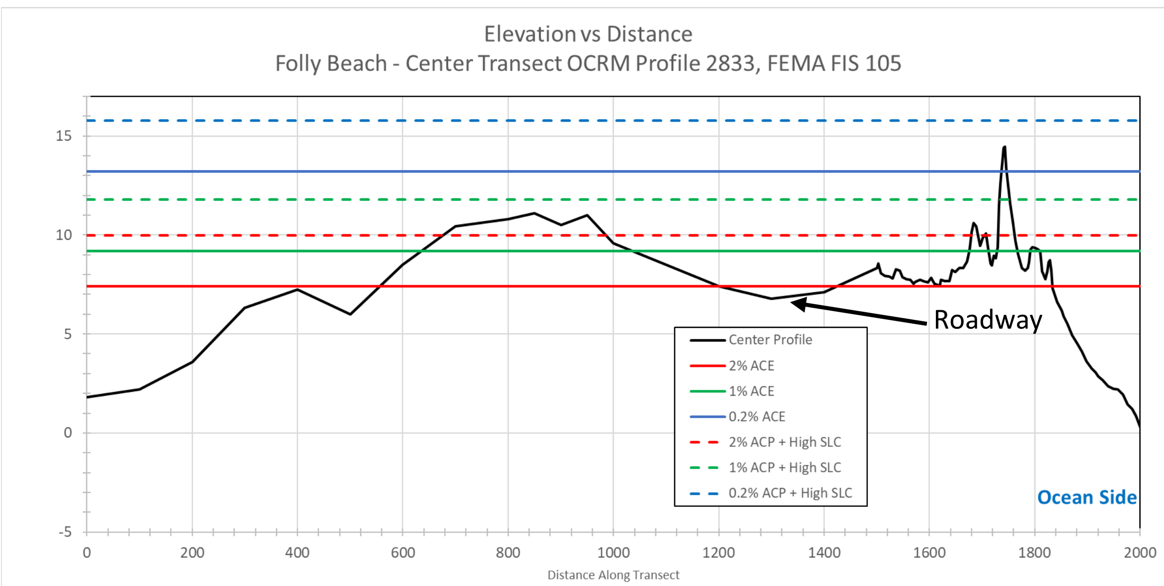


Figure 3-17. Land and AEP Elevations – Center Transect

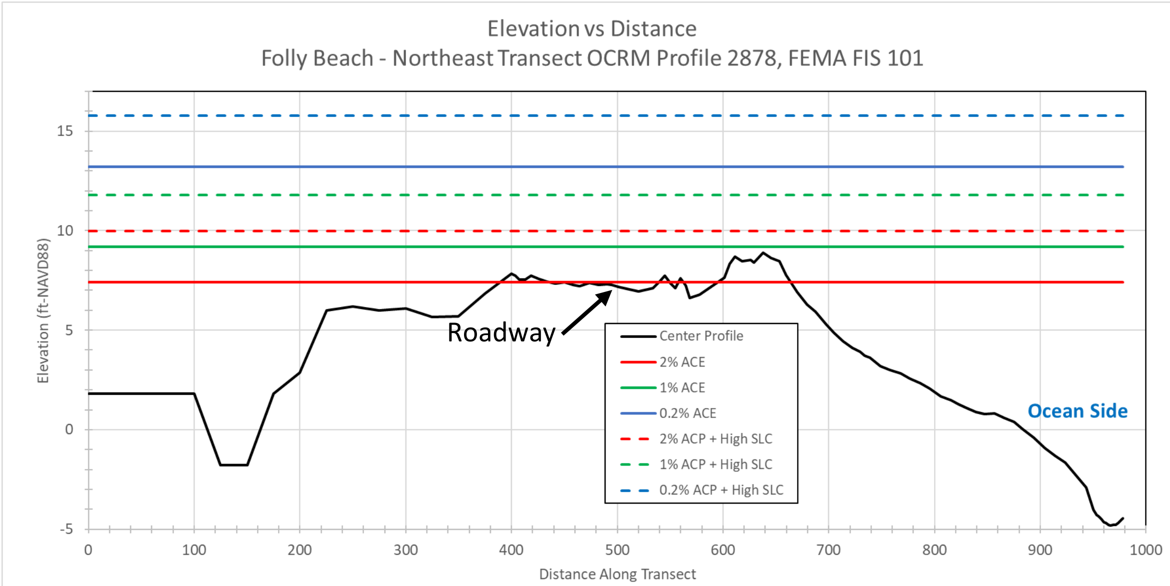


Figure 3-18. Land and AEP Elevations – Northeast Transect

4 Beach-fx Life-Cycle Shore Protection Project Evolution Model

Federal participation in projects is based on a favorable economic justification in which the benefits of the project outweigh the costs. Determining the Benefit to Cost Ratio (BCR) requires both engineering analysis (project cost, performance, and evolution) and economic analyses (plan formulation, plan selection, and quantification of project benefits). The interdependence of these functions has led to the development of the life-cycle simulation model Beach-fx. Beach-fx combines the evaluation of physical performance and economic benefits and costs of shore protection projects (Gravens et. al., 2007), particularly beach nourishment, for justification of Federal participation.

4.1 Background & Theory

Beach-fx is an event-driven life-cycle model. USACE guidance (USACE, 2006) requires that flood damage reduction studies include risk and uncertainty. The Beach-fx model satisfies this requirement by incorporating risk and uncertainty throughout the modeling process. Over the analysis cycle, typically 50 years for new studies, the model estimates shoreline response to a series of historically based storm events applied for each of three USACE sea level change scenarios as required by USACE Engineering Regulation, ER 110-2-8162 (USACE, 2013) and Engineer Pamphlet EP 1100-2-1 as described in Section 3.7. These plausible storms, the driving events, are randomly generated using a Monte Carlo simulation. The corresponding shoreline evolution includes not only erosion due to the storms, but also allows for storm recovery, post-storm emergency dune and/or shore construction, and planned nourishment events throughout the life of the project. Risk based damages to structures are estimated based on the shoreline response in combination with pre-determined damage functions for all structure types. Uncertainty is incorporated within the input data (storm occurrence and intensity, structural parameters, structure and contents valuations, and damage functions) and in the applied methodologies (probabilistic seasonal storm generation and multiple iteration, life cycle analysis). Results from the multiple iterations of the life cycle are averaged over a range of possible values.

The project site itself is represented by divisions of the shoreline referred to as “Reaches”. Because this term may also be used to describe segments of the shoreline to which project alternatives are applied (SBEACH reaches), Beach-fx reaches will be referred to in this appendix as “economic reaches”. Economic reaches are contiguous, morphologically homogenous areas that contain groupings of structures (residences, businesses, walkovers, roads, etc…), all of which are represented by Damage Elements (DEs). DEs are grouped within divisions referred to as Lots. Figure 4-1 shows a conceptual representation of the model setup. A single SBEACH Reach may be composed of several economic reaches. Economic reaches capture the diversity of shoreline dimension and erosion potential that can occur over a single economic reach.

Within the model, each economic reach is associated with a representative beach profile that approximates the cross-shore profile and beach composition of the reach. Multiple economic reaches may share the same representative beach profile while groupings of economic reaches may represent a single design reach. For Folly Beach, the project area was separated into 9 SBEACH reaches and 26 economic model reaches. Table 4-1 provides Folly Beach SBEACH and economic reaches with a map shown in Figure 4-2.

Implementation of the Beach-fx model relies on a combination of meteorology, coastal engineering, and economic analyses and is comprised of four basic elements:

- Meteorological driving forces
- Coastal morphology
- Economic evaluation
- Management measures

The subsequent discussion in this section addresses the basic aspects of implementing the Beach-*fx* model. For a more detailed description of theory, assumptions, data input/output, and model implementation, refer to Gravens et al. 2007; Males et al., 2007, and USACE 2009.

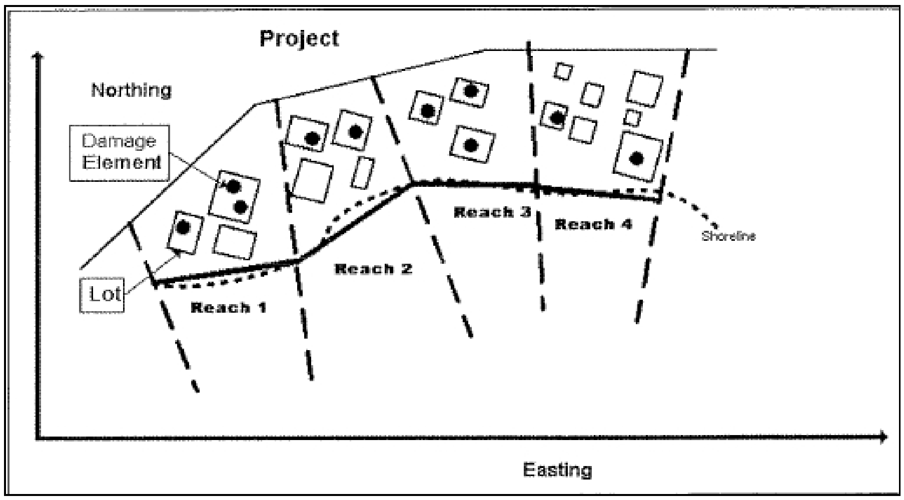


Figure 4-1. Beach-fx Model Setup Representation

Table 4-1. Folly Beach Economic and SBEACH Reaches

SBEACH Reach	Economic Reach
FB 01	R01
FB 02	R02
FB 03	R04 – R07
FB 04	R08
FB 05	R09 – R13
FB 06	R14 – R17
FB 07	R18 – R20
FB 08	R21 – R24
FB 09	R25- R26

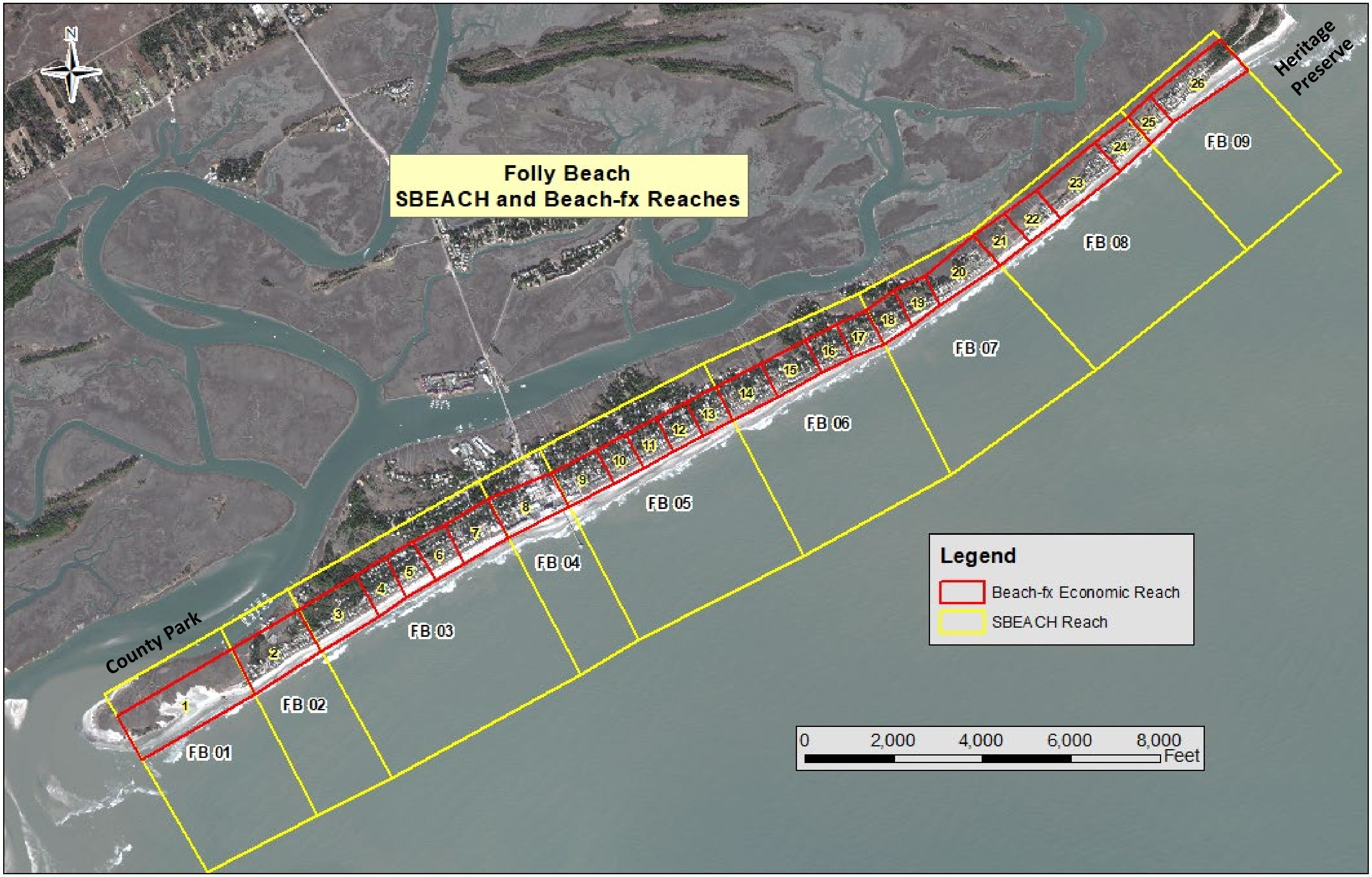


Figure 4-2. Folly Beach SBEACH and Beach-fx Economic Reaches

4.2 Engineering Parameters

4.2.1 Meteorological Driving Forces

The predominant driving force for coastal morphology and associated damages within the Beach-fx model is the historically based set of storms that is applied to the life-cycle simulation. Because the coast of South Carolina is subject to seasonal storms, tropical storms (hurricanes) in the summer months and extra-tropical storms (Northeasters) in the winter and fall months, the “plausible storms” dataset for Folly Beach is made up of both types. These storms were derived from hindcast data obtained from Oceanweather Incorporated (see Sub-Appendix B: Storm Suite Development – Folly Beach). The Folly Beach plausible storm set contains 21 tropical storms and 16 extra-tropical storms, see Table 4-2.

Because storm events may to be of limited duration, passing over a given site within a single portion of the tide cycle, it is assumed that any of the historical storms could have occurred during any combination of tidal phase and tidal range. Therefore, each of the plausible storm surge hydrograph was combined with possible variations in the astronomical tide. This was achieved by combining the peak of each storm surge hydrograph with the astronomical tide at high tide, mean tide falling, low tide, and mean tide rising for each of three tidal ranges corresponding to the lower quartile, mean, and upper quartile tidal ranges. This resulted in 12 combinations for each historically based storm and a total of 252 tropical storm conditions and 192 extra-tropical storms in the plausible storm dataset.

In addition to the plausible storm dataset, the seasonality of the storms must be specified. Storm seasons are based on the season in which the original historical storm occurred. Storm probability is defined through the Probability Parameter which is determined for each season and storm type by dividing the number of storms by the total number of years in the storm record (extra-tropical or tropical). Two storm seasons and two dormant periods were specified for Folly Beach, see Table 4-3.

The combination of the plausible storm dataset and the specified storm season allows the Beach-fx model to randomly select from storms of the type that fall within the season currently being processed. For each storm selected, a random time within the season is chosen and assigned as the storm date. The timing of the entire sequence of storms is governed by a pre-specified minimum storm arrival time. To allow for the possible frequency of Northeaster events in this area, a minimum arrival time of 10 days was specified for Folly Beach. Based on this interval, the model attempts to place subsequent storm events outside of a 20 day window surrounding the date of the previous storm (i.e. a minimum of 10 days prior to the storm event and a minimum of 10 days following the storm event). However, due to the probabilistic nature of the model the minimum arrival time may be overridden as warranted during the course of the life cycle analysis.

Table 4-2. Storm Selection

Tropical Storms	Extratropical Storms
30-Nov-1925	2-Feb-1961
25-Jul-1926	3-Mar-1962
15-Sep-1928	1-Feb-1973
9-Aug-1940	9-Feb-1973
18-Oct-1944	10-Feb-1981
14-Sep-1945	10-Feb-1985
16-Oct-1950	2-Jan-1992
28-Aug-1952	10-Feb-1993
13-Oct-1954	12-Mar-1993
23-Sep-1959	26-Jan-1998
7-Sep-1964	3-Feb-1998
1-Sep-1979	1-Jan-1999
20-Sep-1989	20-Mar-2001
14-Nov-1994	15-Feb-2003
7-Oct-1996	6-Feb-2013
13-Sep-1999	30-Apr-2013
23-Oct-2005	
3-Sep-2008	
1-Oct-2015	
5-Oct-2016	
9-Sep-2017	

Table 4-3. Folly Beach Beach-*fx* Storm Seasons

Storm Season	Start Date	End Date	Probability Parameter Extra-Tropical Storm	Probability Parameter Tropical Storm
Extra-tropical	Jan 1	Apr 30	0.48	0.00
Dormant	May 1	Jun 30	---	---
Tropical	Jul 1	Nov 30	0.00	0.33
Dormant	Dec 1	Dec 31	---	---

4.2.2 Coastal Morphology

The Beach-*fx* model estimates changes in coastal morphology through four primary mechanisms:

- Shoreline storm response
- Applied shoreline change
- Project-induced shoreline change
- Post-storm berm recovery

Combined, these mechanisms allow for the prediction of shoreline morphology for both with and without project conditions.

4.2.3 Shoreline Storm Response

Shoreline storm response is determined by applying the plausible storm set to simplified beach profiles that represent the shoreline features of the project site. For this study, application of the storm set to the idealized profiles was accomplished with the SBEACH coastal processes response model (Larson and Kraus 1989). SBEACH is a numerical model which simulates storm-induced beach change based on storm conditions, initial profiles, and shoreline characteristics such as beach slope and grain size. Output consists of post-storm beach profiles, maximum wave height and wave period information, and total water elevation including wave setup. Pre- and post-storm profiles, wave data, and water levels can be extracted from SBEACH and imported into the Beach-fx Shore Response Database (SRD). The SRD is a relational database used by the Beach-fx model to pre-store results of SBEACH simulations of all plausible storms impacting a pre-defined range of anticipated beach profile configurations.

4.2.4 Idealized Representative Profiles

In order to develop the idealized SBEACH profiles from which the SRD was derived, it was necessary to first develop representative profiles for the project shoreline. The number of representative profiles developed for any give project depends on the natural variability of the shoreline itself. Typically, profiles taken along the project shoreline are compared, aligned and averaged into composite profiles representative of dimensionally consistent segments of the shoreline. A representative profile may define one or more economic model reach. For Folly Beach nine representative profiles define the 26 economic reaches. This is necessary as each of the 26 economic reaches have either a unique background erosion rate or upland width. Folly Beach also included reaches where the majority of the shoreline reach length is armored. Representative profiles are developed according to the similarity between the following seven dimensions:

- Upland elevation
- Dune slope
- Dune height
- Dune width
- Berm height
- Berm width
- Foreshore slope

The start year of the current Beach-fx analysis is 2019 and the base year is 2024. The last nourishment prior to the start year was completed in early 2018. The 2024 shoreline would represent 4 years of erosion applied to the template. In order to estimate a 2024 shoreline, representative profile dimensions for the initial shoreline condition were derived from the late December 2018 and early January 2019 OCRM survey. Because the 2018/2019 OCRM survey did not capture the full upland extent of the dune system, additional upland information was obtained from a LiDAR elevation survey conducted by the USACE Charleston District in 2016.

Idealized profiles were calculated from the 2018/2019 shoreline survey, supplemented by the 2016 inland LiDAR survey, using the Composite Dune Methodology. Table 4-4 provides the dimensions of the idealized future without project representative profiles and the economic reaches they define.

Table 4-4. Dimensions of Idealized Without Project Representative Profiles

SBEACH Reach	Economic Reach	Upland Elevation	Dune Elevation	Dune Width	Dune Slope	Berm Elevation	Berm Width	Foreshore Slope
		(ft- NAVD88)	(ft- NAVD88)	(ft)	(H:1V)	(ft- NAVD88)	(ft)	(H:1V)
FB 01	R01	10	10	0	0.333	8	125	0.033
FB 02	R02	11	11	0	0.333	8	50	0.033
FB 03	R04 – R07	11	14	25	0.333	8	25	0.033
FB 04	R08	12	12	35	0.333	8	125	0.033
FB 05	R09 – R13	10	12	45	0.333	8	50	0.033
FB 06	R14 – R17	10	10	0	0.333	8	25	0.033
FB 07	R18 – R20	10	10	0	0.333	8	0	0.033
FB 08	R21 – R24	9	9	0	0.333	8	0	0.033
FB 09	R25- R26	9	9	0	0.333	8	0	0.033

4.2.5 SBEACH

SBEACH simulates beach profile changes that result from varying storm waves and water levels. These beach profile changes include the formation and movement of major morphological features such as longshore bars, troughs, and berms. SBEACH is a one-dimensional model that considers only cross-shore sediment transport. Longshore wave, current, and sediment transport processes are not included in SBEACH and are computed externally when required.

SBEACH is an empirically based numerical model, which was formulated using both field data and the results of large-scale physical model tests. Input data required by SBEACH describes the storm being simulated and the beach of interest. Basic requirements include time histories of wave height, wave period, water elevation, beach profile surveys and median sediment grain size. Beach-fx is designed to import and process output files exported directly from the SBEACH model.

SBEACH simulations are based on six basic assumptions:

- Waves and water levels are the major causes of sand transport and profile change
- Cross-shore sand transport takes place primarily in the surf zone
- The amount of material eroded must equal the amount deposited (conservation of mass)
- Relatively uniform sediment grain size throughout the profile
- The shoreline is straight and longshore effects are negligible
- Linear wave theory is applicable everywhere along the profile without shallow-water wave approximations

Once applied, SBEACH allows for variable cross shore grid spacing, wave refraction, randomization of input waves conditions, and water level setup due to wind. Output data consists of a final calculated profile at the end of the simulation, maximum wave heights, maximum total water elevations plus setup, maximum water depth, volume change, and a record of various coastal processes that may occur at any time-step during the simulation (accretion, erosion, over-wash, boundary-limited run-up, and/or inundation).

4.2.5.1 SBEACH Calibration

Traditionally, calibration and verification of the SBEACH model is performed as part of the study being undertaken. However, survey profile data at OCRM monuments beyond MLW were not available for calibration of the Folly Beach model immediately before and after significant storms. SBEACH parameters were determined from modelers experience with similar project shorelines and from studies on SBEACH model calibration for a given beach slope and sand size (Leadon, 2015; Leadon & Nguyen, 2011). The native mean grain size at Folly Beach is 0.17 mm and the inverse beach slope varies between 20 and 30 along the shoreline. Based on those conditions, the sediment transport rate coefficient (K) was estimated at 2.0 x 10^{-6} m^4/N and coefficient for slope-dependent term (ε) at 0.005 and the avalanching maximum slope at 40°. Time step was step to 1 minute to ensure stability of the model results. Sensitivity analysis indicated that model results were sensitive grid cell size and time step but not to the sand grain size.

4.2.5.2 SBEACH Simulations

Folly Beach SBEACH simulations were completed for each of the without project profiles and an array of incremental profiles covering a range of potential with-project conditions in combination with each of the tropical and extra-tropical storms in the plausible storm database. From these profiles, changes in the key profile dimensions were extracted and stored in the Folly Beach-fx SRD.

4.2.6 Applied Shoreline Change

The applied shoreline change rate (in feet per year) is a Beach-*fx* morphology parameter specified at each of the model reaches. It is a calibrated parameter that returns the historic background shoreline change rate for that location. Calibration is essential to insure that the morphology behavior is appropriate and representative of the study area.

The applied erosion rate used in Beach-fx is the expected rate of shoreline change in the absence of storm events (USACE, 2009). Beach-fx uses a suite of historic storm events over the 50-year period along with the background erosion rate. It should be noted that this shoreline change rate is averaged over the period of record and does not represent the initial high rates of change that occur immediately after a nourishment project or that might be cyclical or a recent change in the shoreline trends. The planform rates are used later in Beach-fx simulations to capture the higher rates after nourishments and are based on more recent profile data.

The South Carolina Department of Health and Environmental Control's (SCDHEC) Office of Ocean and Coastal Resource Management (OCRM) has established 31 permanent beach profile monuments along Folly Beach, see Figure 4-3. The surveyed profiles extend seaward from the perpetual easement line out to a distance of approximately 3,300 ft offshore and to of depth of -13.5 ft to -16.5 ft NAVD88. The profiles are typically surveyed once a year since 1988 and are provided from OCRM in the NAVD88 datum. The OCRM profile database was the primary source of data used in the coastal morphology analysis because of its consistency in capturing the dune, shore and submerged profile along the same azimuth each year. All available OCRM data was imported into the USACE Regional Morphology Analysis Package (RMAP) software program within CEDAS for processing and analysis of the beach profile data. RMAP Analysis tools were used to calculate the distance from the OCRM monument seaward to the MHW elevation 2.26 ft NAVD88 contour for each year in the dataset to develop the shoreline change rates. RMAP was also used to develop the representative profiles used in SBEACH and Beach-fx by averaging the OCRM profiles within the reach to create the one representative profile.

The results of the historic shoreline analysis at Folly Beach revealed recession and accretion rates that varied both in time and in location along the shoreline. There are numerous natural and man-made features that influence the shoreline change rates at Folly Beach when compared to other shoreline in the Southeast. A 1987 Section 111 report determined that the Federal Charleston Harbor navigation jetties were responsible for 57% of the shoreline retreat at Folly Beach (USACE, 1987). The Section 111 report calculated an averaged Folly Beach long-term shoreline erosion rate of -4.2 ft/yr for the years of 1857 to 1983. Folly Beach is bounded by two inlets with tidal shoals that are continually evolving over time. Terminal groins at ends of the island complicates the dynamics in those areas. Morris Island is located northeast of Folly Beach and has a history of high erosion also related to the navigation jetties. The retreat of Morris Island has likely influenced increasing rates of shoreline retreat on the on the northeast end of Folly Beach. Another issue noted in calculating shoreline change rates on the northeast end of the island was the influence of bulkhead and revetment armoring. After a nourishment event the mean high tide line quickly retreats landward but slows as armoring in encountered.

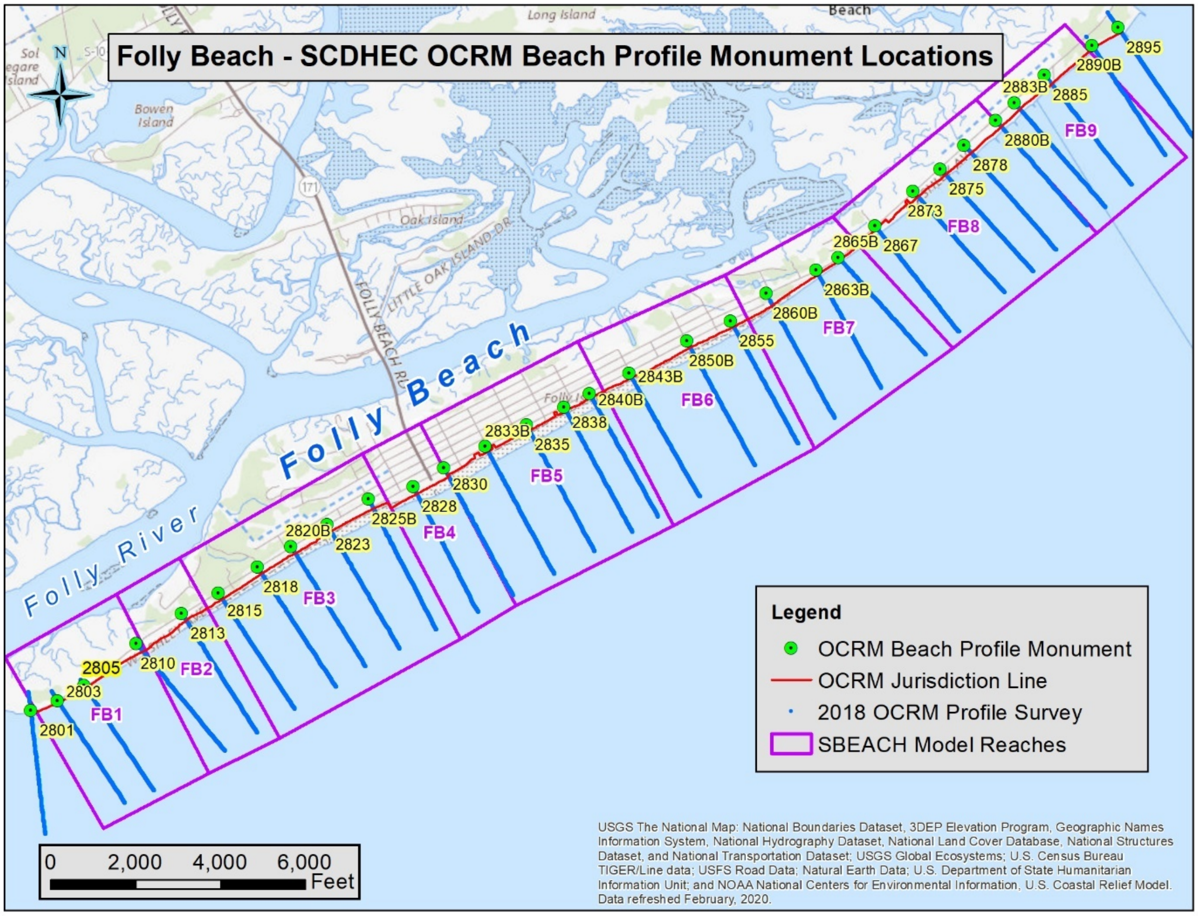


Figure 4-3. OCRM Beach Profile Survey Monuments

The OCRM profile data were used in calculating the historic shoreline change rates. The calculations looked at two periods between beach nourishment projects. Period 1 was between the July 1993 and May 2005 nourishment projects and Period 2 was between the June 2007 and June 2013 projects. The immediate post-nourishment survey was not used in the calculation to allow adjustment of the construction berm. Published long-term OCRM shoreline change rates were also reviewed but profiles on the southeast end were not used given the new terminal groin (SCDHEC, 2010).

The historic shoreline change rate calculations for Reaches FB1 and FB2 were impacted by the 2013 construction of the terminal groin and beach nourishment on the southwest end of the island. Historic rates at OCRM profiles 2805, 2810 and 2813 in FB1 and FB2 exceeded -20 ft/yr prior to 2014. Following adjustments to the new groin, the rates have varied with accretion and erosion in SBEACH Reaches FB1 and FB2. A rate of -2.0 ft/yr was used for those reaches. Shoreline rates for FB4 using OCRM monument 2828 was highly variable (accretion and erosion) and was likely influenced by the Folly Beach Fishing Pier, the interactions with the concrete seawall and by beach scraping. FB4 used an average of the rates from FB3 and FB5. The rates calculated for FB5 to FB7 are relatively high but were consistent through time. The shoreline change rates for FB8 and FB9 on the northeast end of the island have significantly increased since 2008. The loss of Morris Island is likely impacting sediment transport along with the changing dynamics of Lighthouse Inlet shoal on the northeast end. Planform erosion rates used with the nourishment project in-place were significantly higher than the historic applied erosion rates for FB8 and FB9. The groin fields along also play a role in the varying erosion rates along Folly Beach. As the beach profile lowers and retreats the groins become more effective in holding the sand in place with the MHW line becoming more stable within the groins fields.

The Beach-fx calibration followed the steps outlined in Section 7.2 of the Beach-fx Application Guide. The first step in calibrating the Beach-fx model was to determine the role of storm climatology and the post-storm recovery factor. The applied erosion rates were set to zero for each reach to determine the storm induced erosion. The berm width recovery factor was set at 90%. During Beach-fx calibration, applied erosion rates were adjusted for each model reach and the Beach-fx model was run for 100 iterations for the 50-year period. Calibration is achieved when the rate of shoreline change, averaged over hundreds of life cycle simulations, is equal to the background (target) shoreline change rate expected. Table 4-5 provides the historical background erosion rates and the calibrated Beach-fx applied erosion rates.

Table 4-5. Historic Background and Calibrated Beach-fx Applied Erosion Rates

Model Reach	Historic Background Rate	Calibrated Applied Erosion Rate
	(ft/yr)	(ft/yr)
FB1	-2.00	-1.31
FB2	-2.00	-1.49
FB3	-5.40	-5.30
FB4	-4.33	-3.80
FB5	-3.27	-2.82
FB6	-4.90	-4.46
FB7	-7.66	-7.38
FB8	-7.00	-6.30
FB9	-8.88	-8.21

4.2.7 Project Induced Shoreline Change

The project induced shoreline change rate accounts for the alongshore dispersion of placed beach nourishment material. Beach-fx requires the use of shoreline change rates in order to represent the planform diffusion of the beach fill alternatives after placement. The GenCade model was selected for the Folly Beach analysis and a combination of the Genesis and Cascade models developed by the USACE and is on the approved software list for this application. The Genesis model accounts for the interaction of the existing groin fields and terminal groins with beach nourishment. The Cascade model has the ability to simulate the impact of inlets and regional geomorphology.

4.2.7.1 GenCade Model

The development of the GenCade on-line shoreline evolution model of Folly Beach was performed under contract with the engineering firm Moffatt & Nichol. The contract required the compilation and analysis of historical beach profile data to develop and calibrate a planform evolution model and to conduct analytical calculations relative to post-nourishment shoreline change rates of Future with Project (FWP) scenarios in the study reaches along Folly Beach. Details of the Moffatt & Nichol analysis and results are provided in Sub-Appendix C.

4.2.7.2 Shoreline Change Rates

Using the calibrated GenCade model, the project induced shoreline change rates for the selected plan were calculated. Table 4-6 provides the calculated project induced shoreline change rates for the first 12-years after the initial beach nourishment in 2024. The rates reflect the high erosion rates of the beachfill during the first three years as the system adjust. Longshore current transported much of the sand eroded from northeast reaches (R18-R26) to the middle and southwest reaches which reflected accretion during the first years of the project. In the later years of the 12-year project, the change rates moderate and become more uniform as the beach profile is lowered and the groin fields become more effective in trapping sand and as beachfront armor is encountered. Higher rates were noted at transitions in the project alignment; the concrete seawall extends seaward in R8 and at R12 the profile transitions existing dune and berm lines. Table 4-7 includes the planform shoreline change rates for the four beach nourishments averaged over the 12-year period. The trends in Table 4-7 are similar but there are differences reflecting the different grain size from the borrow areas and different starting beach profiles at the time of the renourishment. Within the Beach-fx simulation, the applied erosion rates were subtracted from the planform erosion rates to ensure erosion was not double counted. Storm wave induced erosion continued within the Beach-fx simulation.

The GenCade model was also used to optimize the beachfill taper at the ends of the project by evaluating distances of 750 ft, 1,000 ft and 1,500 ft. Model results indicated the three distance were all viable alternatives and the 750 ft distance was selected on the ends and a 500 ft transition between the 35 ft and 50 ft berm widths between reach 21 and 22. The final recommended plan included placing a berm only at the project ends at the Heritage Preserve and at the County Park to fulfill the Section 111 requirements, see Section 2.3. The fill would extend to the existing terminal groins at each end of Folly Island and therefore no transition at the project ends will be needed if the Section 111 volumes are included.

The initial results of the Beach-fx model using the GenCade developed planform rates were used to refine the Tentatively Selected Plan (TSP) dune and berm beach fill design. The original TSP design included a 35 ft wide berm along the southwest segment of the project between Beach-fx reaches R2 and R21 and a 50 ft wide berm along the northeast segment between reaches 22 and 26. Because of the higher erosion rates along economic reaches R18 to R21, the 50 ft wide berm was extended south for a total length of 9,720 ft between reaches R18 to R26. The planform rates were updated to reflect the change in the berm widths and transition location in the final Beach-fx model simulations.

Table 4-6. Project Induced Planform Shoreline Change Rates, Years 2024 to 2035

Beach -fx Reach	Shoreline Change Rates (ft/yr)											
	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035
R#2	-3.0	-6.3	-7.5	-6.7	-4.1	-1.3	0.9	2.2	2.9	3.1	3.1	3.0
R#3	-1.2	1.2	2.0	1.8	1.2	0.9	1.0	1.2	1.5	1.7	1.9	1.9
R#4	8.4	8.6	7.7	6.0	4.4	3.5	2.7	2.1	1.8	1.6	1.5	1.5
R#5	12.5	13.7	9.6	6.9	5.2	3.7	2.8	2.1	1.7	1.4	1.2	1.1
R#6	30.5	12.9	7.2	4.8	3.6	2.7	2.1	1.6	1.3	1.0	0.8	0.7
R#7	2.8	-1.3	-1.1	-0.5	0.1	0.3	0.3	0.3	0.2	0.1	0.0	-0.1
R#8	-33.8	-14.3	-7.6	-4.7	-3.3	-2.5	-2.0	-1.7	-1.5	-1.4	-1.3	-1.3
R#9	18.5	2.7	-1.5	-2.8	-3.2	-3.2	-3.1	-2.9	-2.7	-2.6	-2.5	-2.4
R#10	18.3	6.2	0.3	-2.0	-3.0	-3.3	-3.3	-3.3	-3.1	-3.0	-2.9	-2.9
R#11	-4.4	-3.1	-2.8	-3.2	-3.4	-3.5	-3.5	-3.4	-3.4	-3.3	-3.2	-3.1
R#12	-29.6	-10.6	-6.2	-4.4	-3.6	-3.3	-3.2	-3.1	-3.1	-3.2	-3.2	-3.3
R#13	-11.0	-10.0	-6.2	-4.3	-3.4	-3.0	-2.9	-3.0	-3.1	-3.2	-3.3	-3.4
R#14	0.3	-1.9	-2.2	-2.2	-2.2	-2.4	-2.6	-2.8	-3.0	-3.2	-3.3	-3.4
R#15	8.5	4.8	1.6	-0.4	-1.7	-2.6	-3.2	-3.6	-3.9	-4.1	-4.3	-4.2
R#16	6.9	2.1	-0.6	-2.4	-3.5	-4.3	-4.8	-5.1	-5.3	-5.4	-5.5	-5.8
R#17	-3.6	-5.0	-5.6	-5.9	-6.2	-6.3	-6.4	-6.4	-6.4	-6.4	-6.4	-6.8
R#18	-18.6	-13.5	-11.8	-10.7	-9.9	-9.3	-8.9	-8.6	-8.5	-8.3	-8.2	-7.9
R#19	-19.1	-18.0	-15.5	-13.7	-12.6	-11.8	-11.3	-10.9	-10.5	-10.3	-10.1	-9.8
R#20	-21.2	-17.2	-15.9	-14.9	-14.2	-13.6	-13.1	-12.7	-12.4	-12.0	-11.7	-11.5
R#21	-3.5	-12.0	-13.2	-13.5	-13.6	-13.5	-13.4	-13.2	-13.0	-12.7	-12.4	-12.1
R#22	-12.6	-11.4	-12.1	-12.5	-12.7	-12.8	-12.8	-12.7	-12.5	-12.2	-12.0	-11.8
R#23	-13.4	-11.9	-11.6	-11.7	-11.8	-11.9	-11.8	-11.6	-11.4	-11.2	-11.0	-10.8
R#24	-9.3	-10.2	-11.5	-11.7	-11.5	-11.2	-11.0	-10.7	-10.6	-10.4	-10.2	-10.2
R#25	-4.8	-11.5	-12.4	-11.8	-11.1	-10.5	-10.1	-9.8	-9.7	-9.7	-9.8	-10.0
R#26	-21.7	-13.0	-9.3	-7.6	-6.9	-6.7	-6.6	-6.9	-7.2	-7.6	-8.1	-8.5

4.2.8 Post Storm Berm Recovery

Post storm recovery of eroded berm width after passage of a major storm is a recognized process. Within Beach-fx, post-storm recovery of the berm is represented in a procedure in which the user specifies the percentage of the estimated berm width loss during the storm that will be recovered over a given recovery interval. It is important to note that the percentage itself is not a “stand alone” parameter that is simply applied during the post storm morphology computations. The percentage of berm recovery is estimated prior to model calibration and becomes a tunable calibration parameter to ensure model convergence (when the model reproduces the target erosion rates as discussed in Section 4.2.6: Applied Shoreline Change). For Folly Beach calibration required a varying berm recovery factor of 90% over a recovery period of 21 days.

Table 4-7. Project Induced Planform Shoreline Change Rates, 12 Year Average

Beach-fx Reach	Average Shoreline Change Rates (ft/yr)			
	Jan2024 Fill	Jan2036 Fill	Jan2048 Fill	Jan2060 Fill
R#2	-1.1	0.1	0.2	-1.5
R#3	1.3	0.6	0.3	-0.4
R#4	4.1	0.7	0.3	-0.2
R#5	5.2	0.5	0.1	-0.2
R#6	5.8	0.4	0.2	-0.1
R#7	0.1	0.0	0.0	0.0
R#8	-6.3	-0.4	0.0	0.1
R#9	-0.5	0.2	0.9	0.9
R#10	-0.2	-1.2	-0.9	-0.9
R#11	-3.3	-3.0	-3.2	-3.3
R#12	-6.4	-6.9	-7.1	-7.0
R#13	-4.7	-5.3	-5.4	-5.2
R#14	-2.4	-2.7	-2.8	-2.4
R#15	-1.1	-1.2	-1.3	-0.9
R#16	-2.8	-2.8	-2.8	-2.3
R#17	-6.0	-5.9	-6.0	-5.5
R#18	-10.3	-10.4	-10.4	-10.3
R#19	-12.8	-12.9	-12.9	-13.0
R#20	-14.2	-14.3	-14.3	-14.5
R#21	-12.2	-12.2	-12.2	-12.5
R#22	-12.4	-12.4	-12.4	-12.6
R#23	-11.7	-11.7	-11.7	-11.8
R#24	-10.7	-10.8	-10.8	-10.9
R#25	-10.1	-10.2	-10.2	-10.5
R#26	-9.2	-9.2	-9.2	-9.2

4.2.9 Management Measures

Shoreline management measures that are provided for in the Beach-fx model are emergency nourishment and planned nourishment.

4.2.9.1 Emergency Nourishment

Emergency nourishments are generally limited beach fill projects conducted by local governments in response to storm damage. The Beach-fx model assumes emergency fill events have a single profile template, a consistent length of coverage, and occur when specific post-storm shoreline conditions are met. Folly Beach does not have a history of consistent emergency nourishment in response to storm related erosion. The lack of a history of consistent locally sponsored post-storm emergency events, makes assigning realistic emergency fill triggers and specifications within Beach-fx impossible. Therefore, this management measure was not included in the Folly Beach-fx analysis.

4.2.9.2 Planned Nourishment

Planned nourishments are handled by the Beach-fx model as periodic events based on nourishment templates, triggers, and nourishment cycles. Nourishment templates are specified at the model reach level and include all relevant information such as order of fill, dimensions, placement rates, unit costs, and borrow-to-placement ratios. Planned nourishments occur when user defined nourishment triggers are exceeded and a mobilization threshold volume is met. At a pre-set interval, all model reaches which have been identified for planned nourishment are examined. In reaches where one of the nourishment threshold triggers is exceeded, the required volume to restore the design template is computed. If the summation of individual model reach level volumes over the extent of the project exceeds the mobilization threshold volume established by the user, then nourishment is triggered and all model reaches identified for planned nourishment are restored to the design template.

4.2.9.3 Nourishment Templates

Beach-fx planned nourishment templates are defined by three dimensions, the template dune height, template dune width, and template berm width. Berm elevations and dune and foreshore slopes remain constant based on the existing profiles. The SBEACH Data Generator was used to develop multiple dune and berm combinations for simulation with SBEACH and the storm suite. Dune combinations included top widths between 5 ft and 45 ft and top elevations from 9.0 ft to 15.0 ft NAVD88. The dunes have a side slope of 3H:1V. Berm widths varied between 0.0 ft and 150 ft at 25 ft increments. A summary of profile template alternatives evaluated is provided in Table 4-8. A berm elevation of 8.0 ft NAVD88 was selected as this is the existing berm elevation noted in OCRM profiles with no scarps. Beach-fx is limited to a single berm at a constant elevation. The 35 ft berm was later added to refine the design between 25 ft and 50 ft berms.

Table 4-8. SBEACH Profile Alternative Templates Analyzed

SBEACH Reach	Dune Elevations (ft)	Dune Top Widths (ft)	Berm Widths (ft)	Total Profiles
1	10, 11, 12, 13, 14, 15	5, 15, 25, 35, 45	0, 25, 35, 50, 75, 100, 125	180
2	11, 12, 13, 14, 15	5, 15, 25, 35, 45	0, 25, 35, 50, 75, 100, 125	150
3	11, 12, 13, 14, 15	5, 15, 25, 35, 45	0, 25, 35, 50, 75, 100, 125	150
4	11, 12, 13, 14, 15	5, 15, 25, 35, 45	0, 25, 35, 50, 75, 100, 125	150
5	12, 13, 14, 15	5, 15, 25, 35, 45	0, 25, 35, 50, 75, 100	100
6	10, 11, 12, 13, 14, 15	5, 15, 25, 35	0, 25, 35, 50, 75, 100	120
7	10, 11, 12, 13, 14, 15	5, 15, 25, 35	0, 25, 35, 50, 75, 100, 125	144
8	9, 10, 11, 12, 13, 14, 15	5, 15, 25, 35	0, 25, 35, 50, 75, 100, 125, 150	196
9	9, 10, 11, 12, 13, 14, 15	5, 15, 25, 35	0, 25, 35, 50, 75, 100, 125, 150	196

4.2.9.4 Nourishment Distance Triggers and Mobilization Threshold

Beach-fx planned nourishment templates have three nourishment distance triggers (1) berm width, (2) dune width, and (3) dune height. Each distance trigger is a fractional amount of the corresponding nourishment template dimension. When the template dimensions fall below the fraction specified by the trigger, a need for re-nourishment is indicated. For Folly Beach the dune width trigger was set to 0.90, dune height trigger was 0.85 and the berm width trigger was set to 0.75.

The mobilization threshold (minimum nourishment volume required to trigger a nourishment cycle) can be set in coordination with the berm trigger to control the nourishment cycles. The berm trigger can be used to maintain an “allowable” minimum berm width if desired. For Folly Beach, rather than a specific minimum berm width, the trigger and threshold were used to ensure an “allowable” minimum volume of material. The berm trigger was set at 0.75, which allows Beach-fx to begin assessing volume deficiencies almost immediately. The mobilization threshold was then set to a volume reflecting expected volume losses between placement events. The mobilization threshold was then set to a volume reflecting expected volume losses between placement events, see Economics Appendix E for additional details.

4.3 Recommended Plan

From the Beach-fx economic analysis of the dune and berm combinations a recommended plan was developed. The recommended plan included a dune and berm combination for economic reaches 2 through 26. The plan includes a continuous 5.0 ft top width dune at elevation 15.0 ft NAVD88 with a 35 ft wide berm on the southwest end and a 50 ft berm along the northeast end, details are provided in Section 5.

The Folly Beach County Park (Reach1) on the southwest end of Folly Island and the Heritage Preserve on the northeast end were determined not economically feasible because of lack of infrastructure with minimum damages or benefits and were initially not included in the Recommended Plan. To address comments on the draft report from the Local Sponsor and from the Agency Technical Review, the Project Development Team decided to include a berm only nourishment plan to meet the Section 111 requirements related to the Charleston Jetty impacts on Folly Island. The 2,200 ft section at the Folly Beach County Park will include a 35 ft wide berm only that will transition into the existing terminal groin on the southwest end during the initial fill and 12 year nourishment cycles. The northeast end at the Heritage Preserve will include a 2,000 ft long 50 ft wide berm only. Detailed design and nourishment volumes for the two sections will be determined following the pre-project beach profiles surveys. For more information on Section 111 see Appendix G.

Alternative engineering designs were considered and rejected. A series of detached breakwaters and submerged reef features along the shoreline were rejected because of cost and as a hazard to navigation. Given that spring tides at Folly Beach can exceed 6 ft the breakwaters or submerged reefs would also create hazardous currents to swimmers along the shoreline. Rock revetments and seawalls designed to approximate USACE standards were rejected based on cost, negative environmental impacts on sea turtle nesting and limited available real estate for the structures along the shoreline and would still require nourishment.

5 Project Design

5.1 Project Length

The Folly Beach Recommended Plan includes four segments that will receive nourishment, see Figure 5-1. The main southeast segment is 16,970 ft in length and extends from station 22+00 to 191+70 and includes a 750 ft transitions on the southeast end that extends into the Folly Beach County Park. The northeast segment is 9,720 ft in length and extends from station 191+70 to 288+90 to end at an existing timber groin. The length of these two main segments of dune and berm is 26,690 ft or 5.1 miles. The transition between these two segments in 750 ft long. Typical profiles of the dune and berm template in the main southwest and northeast segments are provided in Figures 5-2 and 5-3.

The Plan also includes nourishment of the berm only at the Folly Beach County Park on the southwest terminus of Folly Island and Lighthouse Inlet Heritage Preserve on the northeast end as part of the Section 111 requirements of the Charleston Harbor Jetty. The Folly Beach County Park includes a 35 ft wide berm with a length of 2,200 ft. The Heritage Preserve includes a 50 ft wide berm with a length of 2,000 ft. Note that the two ends of Folly Island include terminal groins and the berm design includes filling 35 ft or 50 ft seaward of the OCRM Jurisdictional Baseline. The distance and volume may vary each nourishment cycle given the area influenced by the terminal groins and future beach profile surveys.

A general description of the Recommended Plan is provided below.

<u>Southwest Segment</u>
Station: 22+00 to 191+70
Length: 16,970 ft
Berm Width: 35 ft
Berm Elevation: 8 ft NAVD88
Dune Top Width: 5 ft
Dune Elevation: 15 ft NAVD88

<u>Northeast Segment</u>
Station: 191+70 to 288+90
Length: 9,720 ft
Berm Width: 50 ft
Berm Elevation: 8 ft NAVD88
Dune Top Width: 5 ft
Dune Elevation: 15 ft NAVD88

<u>Folly Beach County Park Segment</u>
Station: 0+00 to 22+00
Length: 2,200 ft
Berm Width: 35 ft
Berm Elevation: 8 ft NAVD88

<u>Heritage Preserve Segment</u>
Station: 288+90 to 308+90
Length: 2,000 ft
Berm Width: 50 ft
Berm Elevation: 8 ft NAVD88

5.2 Project Baseline

The project construction baseline will be seaward of the SCDHEC OCRM Jurisdictional Baseline and in the general vicinity of the landward toe of the existing dune. In regions where the existing dune is ill defined, extrapolation from adjacent areas with dunes, consideration of localized topography, and position infrastructure will be considered. Due to the complexity of the shoreline, involving residential and commercial structures as well as instances of shoreline armor, the exact baseline will not be fully determined until the Planning, Engineering, and Design (PED) phase of the study. During the PED and construction phases coordination with the local sponsor and with private property owners will be required to ensure there is no ponding of stormwater runoff landward of the project and there is adequate stormwater drainage.

5.3 Project Dune

The plan includes raising the dune to a uniform elevation of 15 ft NAVD88 with a minimum top width of 5 ft and 1V:3H side slopes. The peak storm surge and wave heights during Hurricane Hugo in the storm suite defined the dune crest elevation. Existing dunes may be extended seaward depending on the baseline location and elevation and will be better defined during the PED phase. The existing beachfront dune line along Folly Beach is variable with multiple dune lines both seaward and landward of project construction baseline including reaches with no dune. The southwest segment of Folly Beach currently has an established dune line at elevations 10 ft to 13 ft NAVD88 generally landward of the project baseline. The middle section of Folly Beach currently has a dune system at elevation 11 to 15 ft landward and seaward of the project baseline. The exact layout of the dune and berm template will be determined during the PED phase. The northeast segment of the project generally has either no dune or no existing dune above elevation 10 ft NAVD88 and includes extensive armoring.

5.4 Project Berm

The berm elevation is at elevation 8.0 ft NAVD88, which is consistent with the previously authorized project and approximates the natural berm elevation. The berm width is 35 ft wide in the southwest segment and 50 ft in the higher erosion prone northeast segment between Beach-fx Reaches 18 to 26. Restricting the design berm elevation to the natural berm elevation minimizes scarping of the beach fill as it undergoes readjustment. Vertical scarps can hinder the beach access of nesting sea turtles, and may also pose safety problems related to recreational beach use. Other reasons for mimicking the natural berm elevation are related to storm damage protection. A berm constructed at a lower elevation would increase the probability of overtopping by relatively frequent storms, thereby offering less protection to upland development and/or existing dunes. A higher berm elevation could result in problems related to backshore flooding due to excessive rainfall or wave overtopping. A higher berm may also be more susceptible to wind-induced erosion.

5.5 Project Beach Slopes

After adjustment and sorting of the placed material by wave action, the material is expected to adjust to an equilibrium beach slope, similar to the native beach. Beach slopes tend to be variable dependent on location of nearby groin and beach armoring. Beach slopes in the project vary between 1V:20H to 1V:30H. Sand from the various borrow sites may also differ in mean grain size with different slopes after the equilibrium profile and the wave climate is achieved.

It is unnecessary and impractical to artificially grade beach slopes below the low water elevation since they will be shaped by wave action. The front slope of the beach fill placed at the time of construction or future renourishment may differ from that of the natural profile. The angle of repose of the hydraulically placed material depends on the characteristics of the fill material and the wave climate in the project area. With steep initial slopes, the material will quickly adjust to the natural slopes. For design purposes it is assumed that that construction berm will have an approximate slope of 1V:15H.

5.6 Project Cross-Shore Dimensions

The project cross-shore dimensions, seaward of the construction baseline will vary on the existing beach profile at the time of construction. The plan includes a dune with a top width of 5 ft at elevation 15 ft NAVD88 and side slopes of 1V:3H. With a berm elevation of 8.0 ft NAVD88 at the project construction baseline, the base of the proposed dune will be 47 ft wide. With the 35 ft berm the total project cross-shore width would be 82 ft and for the 50 ft berm the width would be 97 ft. The total width of the construction template will be wider and will be determined during the PED phase.

5.7 Project Volumes and Renourishment Interval

Each complete Beach-fx model run consists of 100 iterations, each iteration was estimated that initial construction in the base year (2024) would require approximately 2.2 mcy, followed by an additional 2.1 mcy of re-nourishment on average at 12 year intervals. The selected project layout can be reviewed in Figure 5-1.

Table 5-1 provides the Beach-fx project initial volumes and re-nourishment interval volumes for each of the three sea level change scenarios. Each complete Beach-fx model run consists of 100 iterations, each iteration representing 50 years of analysis. The volumes include overfill ratios for the different borrow area used within Beach-fx. Based on the recommended plan with 100 iterations an average volume was determined for each initial fill event and each subsequent renourishment event. Model runs were made for each of the three sea level rise cases, Base (low), Intermediate, and High. Based on the economic analysis the renourishment interval was set at 12 years. The final nourishment was increase to include two additional years of eroded volume to reach the end of the 50 years of analysis.

Table 5-1. Beach-fx Project Volumes and Renourishment Interval: 50 Year Period

Project Volumes				
Sea level change Case	Volume Description	Initial Fill Volume (cubic yards)	Renourishment Interval (years)	Average Volume per Interval (cubic yards)
Base	Min - Max	---	12	1,376,000 – 2,453,000
	Average	2,042,000		1,914,000
Intermediate	Min - Max	2,108,000 – 2,484,000	12	1,637,000 – 3,013,000
	Average	2,169,000		2,106,000
High	Min - Max	---	12	3,618,000 – 2,486,000
	Average	2,298,000		2,899,000

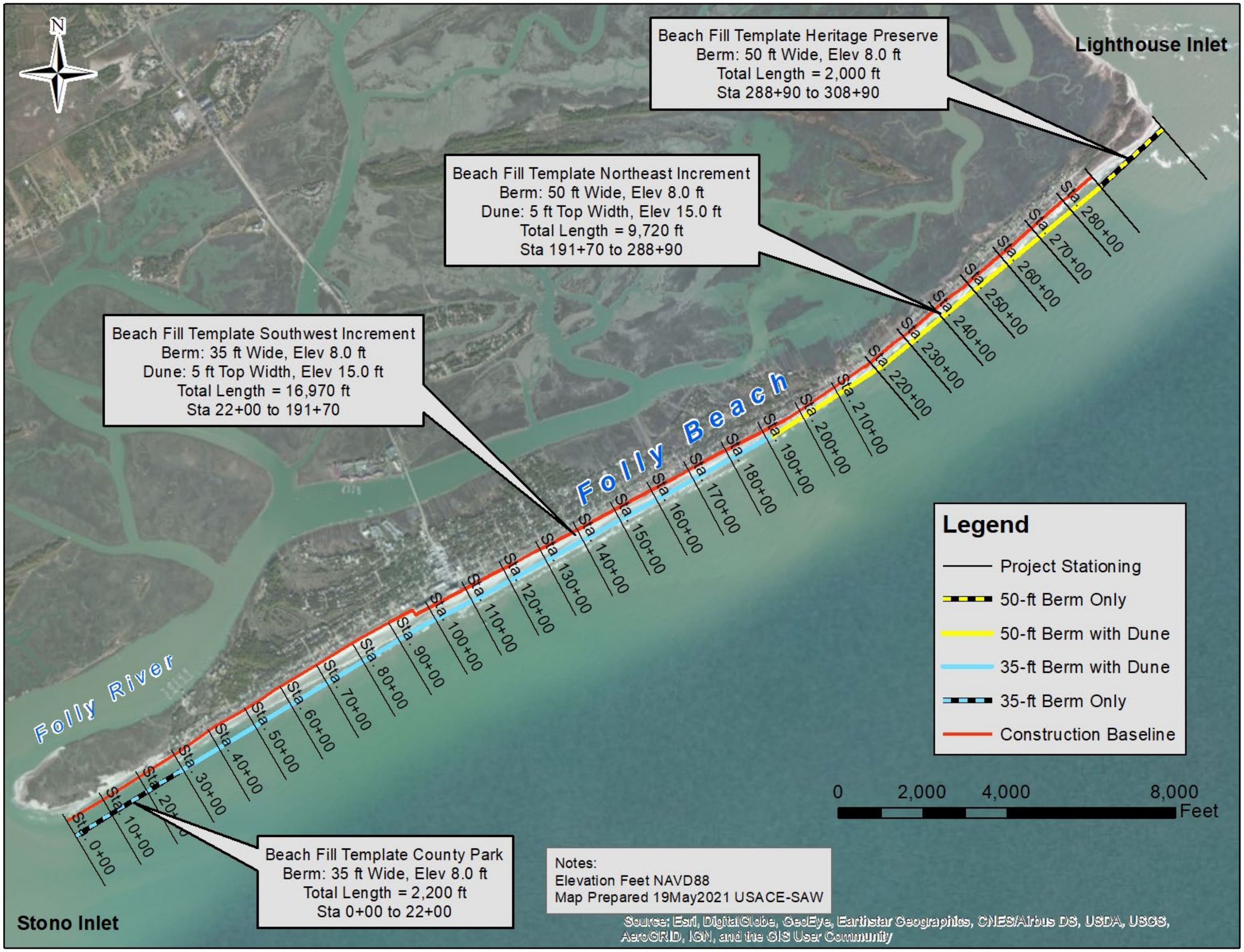


Figure 5-1. Selected Plan Project Layout

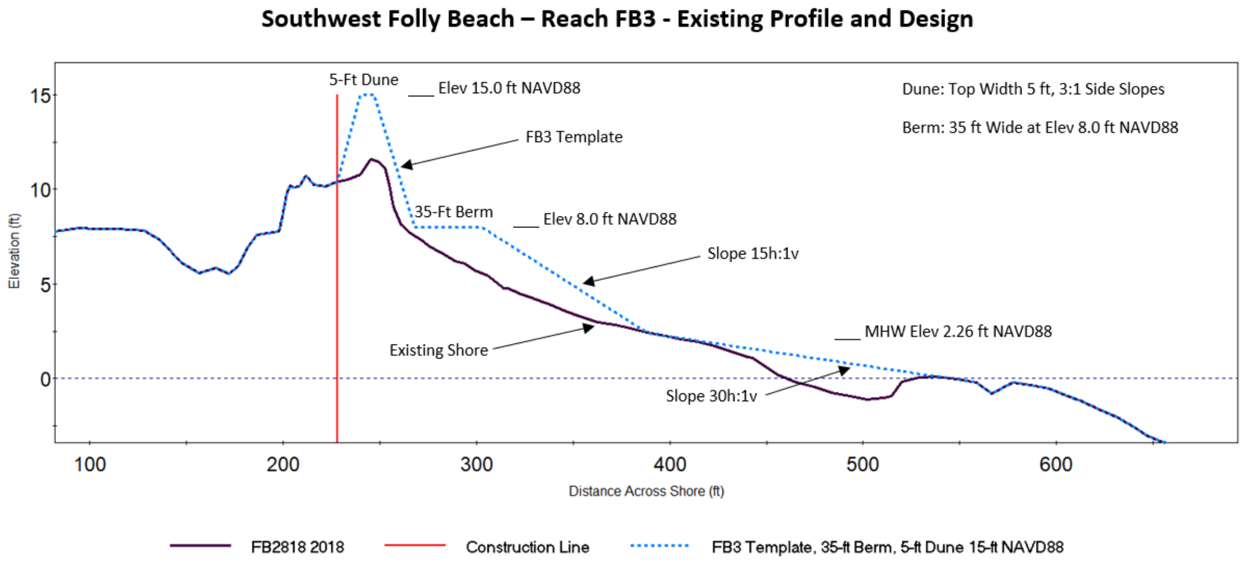


Figure 5-2. Typical Profile Southwest Segment Folly Beach

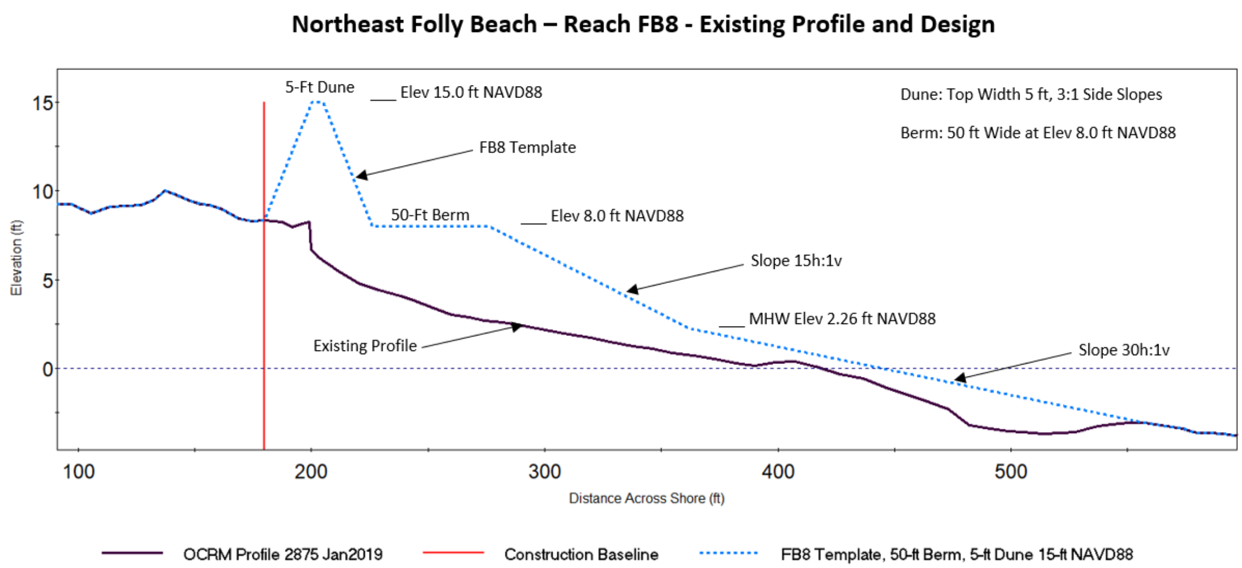


Figure 5-3. Typical Profile Northeast Segment Folly Beach

6 Borrow Area Impact Analysis

The USACE Engineering Research & Development Center's (ERDC) Coastal Hydraulics Laboratory (CHL) conducted two sand borrow area impact studies for the Folly Beach CSRM Study. The first study analyzed five offshore borrow sites. The results of this study are summarized below and with details provided in CHL's final report provided in Sub-Appendix D. The Wilmington District investigated the recharge rate for material excavated from the Folly River in 2013 with details provided in Sub-Appendix E. The second CHL study focused on sediment transport and morphologic changes due to sand dredged from borrow areas within the Folly River and Stono Inlet. The final report for this study is provided in Sub-Appendix F.

6.1 Offshore Borrow Areas

Excavation of sand from locations offshore of Folly Beach for nourishment projects will cause changes in the nearshore bathymetry which will affect the wave transformation in that area. CHL conducted an analysis of the proposed borrow areas on wave propagation at Folly Beach and Kiawah Island using the Steady-state wave model (STWAVE). The software is on the USACE approved list for this application. The map in Figure 6-1 includes the location and the depths of excavation for the five borrow areas initially proposed, note that depths on the NOAA chart are in MLLW datum. The map also includes thirteen reference locations along the Folly Beach shoreline and Stono Inlet for evaluating changes in wave height and direction. The STWAVE model uses wave data from the WIS hindcast Station #63348 from 1980 to 2017 as the offshore boundary condition and bathymetry from FEMA's South Carolina Storm Surge grid.

The borrow areas were evaluated for two wave conditions; monthly mean and monthly maximum. Waves from four directions were evaluated; 60°, 115°, 170° and 225°. The mean monthly condition had a wave height of 3.6 ft and a period of 8.4 second. The monthly maximum condition had a wave height of 8.5 ft with a period of 9 seconds. The areas were also evaluated for the extreme event with a recorded wave direction of 97°, wave height of 20.3 ft and a period of 18 seconds. Figure 6-2 provides a reference for the dominant wave directions along with Kiawah Island, Stono Inlet Shoal and the Charleston Harbor Jetties.

6.1.1 Borrow Areas A and B

Borrow areas A and B (Seaward) are located about 5 miles southeast of Lighthouse Inlet. CHL evaluated this location but the borrow area was ultimately rejected because the volume of suitable sand was depleted during the 2014 Folly Beach nourishment project, no further analysis.

6.1.2 Borrow Areas E and K

Borrow Areas E and K (Stono Ebb Shoal #2) are located about 4.5 miles southeast of the east end of Folly Beach. This borrow area is likely a relict ebb shoal of Stono Inlet developed during periods of lower sea level. The mean grain size is 0.23 mm and the percent of silt and clay fines was 3.08% in Area E and 6.88% in Area K. The in-place volume for Area E is 14,000,000 CY and the volume for Area K is 500,000 CY.

6.1.3 Borrow Area F

Borrow Area F (Lighthouse) is located about 2 miles south of Lighthouse Inlet and 1.5 miles offshore the east end of Folly Beach. Excavation was shallower than the other borrow areas at a depth of -22.0 ft NAVD88. The mean grain size is 0.26 mm and the percent of silt and clay fines was 5.31%. The in-place volume for Area F is 2,800,000 CY.

6.1.4 Borrow Area G

Borrow Area G (Central Folly) is located in the center of the region about 2.5 miles offshore of Folly Beach. The mean grain size is 0.17 mm and the percent of silt and clay fines was 7.68%. The in-place volume for Area G is 8,000,000 CY. Borrow Area G was rejected as a potential borrow site because of the high fines content, no further analysis.

6.1.5 Borrow Areas I and J

Borrow Area I is located within the Stono Inlet throat and Area J is located in the inner ebb tide shoal. Excavation was assumed to be 10 feet in depth below the existing bathymetry. During the initial CHL analysis, the STWAVE results indicated that excavation of these two areas resulted in significant wave height increase to Folly Beach and to the eastern tip of Kiawah Island with a high risk of negative impacts. Wave heights increased by 1.2 feet along the perimeter of the borrow areas for the mean monthly wave conditions. Borrow Areas I and J were rejected as potential borrow sites, no further analysis.

6.1.6 Results – Offshore Borrow Area Effects

From the wave rose in Figure 6-2 approximately 84% of the wave direction at Folly Beach is between 68° (ENE) and 115° (SSE). This summary focuses on the results for borrow Area E & K and Area F from the 115° and 170° dominant wave directions with results provided in Table 6-1. Additional results are provided in CHL's report in Sub-Appendix-D. Wave heights decreased in the immediate area over the borrow sites and increased along the leeward side of the borrow site dependent on the wave direction. Generally the increased wave heights did not propagate towards the shoreline significantly higher than existing conditions. The greatest increase in wave height occurred was during the most oblique wave angle of 225°, this wave direction has a frequency of occurrence of 1.7%.

The extreme condition showed the greatest effects of borrow area excavation but the effects remained isolated within the borrow areas. Wave heights increased over 1.4 ft at all sites but the increased waves did not propagate further inshore compared to existing conditions. There were no increased wave heights along the 13 reference points along the shore.

Use of borrow Area E & K did not show significant impacts along the 13 reference points along Folly Beach or at Stono Inlet and Kiawah Island. The largest wave height increase at the reference locations for the mean monthly wave condition was 0.03 ft at locations 7 and 8. The maximum monthly wave condition showed no increase with wave heights at the reference locations compared to existing conditions.

For the frequent wave direction the use of borrow Area F had a 0.18 ft increase in the mean monthly wave height at reference location 6 for a wave direction 115°. This is a 4.3% increase above existing conditions. Locations to the east of location 6 saw a decrease in wave heights. For waves from 170° there was a 0.1 ft increase or 2.3% at location 6. There was no increase in wave heights for the maximum monthly wave conditions when using borrow area F for the 115° and 170° wave directions. There were increases and decreases along the shoreline at reference locations 5 and 7 from the infrequent oblique wave angles of 225° and 60°.

6.2 Folly River Recharge Rate

The Folly River navigation channel has routinely been dredged since the 1970's. The volume removed average about 30,000 CY with the material placed along the Charleston County Part on the southwest end Folly Beach (CSE, 2002). The first large-scale dredging of Folly River was in 1993 with 3.1 mcy removed from the river and 2.7 mcy placed along Folly Beach. An evaluation of recharge rates of South Carolina borrow sources investigated the post 1993 recovery of the Folly River using annual bathymetric surveys (Van Dolah, 1998). The study estimated an average annual recharging rate of 18% for compete refilling in 5.5 years. The study noted that Bird Key near the confluence of the Folly River and Stono Inlet was eroded following the 1993 project. The 2001 Folly Beach monitoring report of the 1993 project noted that the project likely exacerbated erosion at the southwest end of the island at the Folly Beach County Park (CSE, 2001). Reduced placement of material from the navigation channel likely contributed to the erosion also. A terminal groin was constructed at the southwest end of Folly Beach at the County Park in June, 2013 to address the loss of beach and damage to infrastructure at the Park. The monitoring reports noted that the rapid refilling rate of 18% may have been related loss of sand from the intertidal shoals including Bird Key and from the southwest end of Folly Beach.

An updated estimate of the Folly River recharge rate was conducted based on the May 2013 dredging of the Folly River. The 415,000 CY of material excavated from the Folly River was used to restore the beach at the County Park and to facilitate the construction of the 745 ft long terminal groin. A summary of the recharge analysis is provided in Sub-Appendix E. An average recharge rate of 12.25% was calculated over the four year period. The lower recharge rate compared to the post-1993 project rate of 18% may be related to the smaller volume excavated and the influence of the new terminal groin with the Stono Inlet system.

6.3 Sediment Transport Folly River and Stono Inlet

A detailed analysis of sediment transport at Stono Inlet and of the adjacent beaches was conducted by the USACE Engineer Research and Development Center. The primary objective of this study was to evaluate sediment transport and morphologic changes due to sand dredged from borrow areas and placement on nearshore beaches. The five borrow areas included the Folly River and Areas E, I, J & K in Stono Inlet, see Figure 6-1. The Coastal Modeling System (CMS) was used to simulate the wave climate, current, tide, and sediment transport within and around the immediate vicinity of Stono Inlet, Bird Key/Skimmer Flats, navigation channels, Folly Island, and the eastern end of Kiawah Island. Sediment management alternatives of sand dredged and placement were developed and comparisons between alternative results were conducted under various forcing conditions in the nearshore area of the Stono Inlet and the Folly River. The study included a field data collection effort of tidal and current patterns in the study area. The final study report entitled “Sediment Transport Modeling at Stono Inlet and Adjacent Beaches, South Carolina” and dated December, 2020 is included as Sub-Appendix F. The model simulations included existing Base conditions and with material removed from the five borrow areas, conditions modeled independently. Model simulation periods included 8-day storm simulations (Hurricane Hugo) and a full one-year simulation (2018).

Major findings and conclusions from the CMS model report include:

1. The field data collection effort supported a successful numerical model calibration and validation capturing tidal flushing, current and wave climate of the nearshore estuarine system and a good representation of factors driving sediment transport.
2. Relatively large sediment backfilling occurs in the Folly River borrow area. The majority of the backfill sediment originates from the nearshore area along Folly Beach and a smaller volume from neighboring shallow areas in the Folly River and Stono Inlet. The recharge rate for the actual dredged footprint was 25% during the first year. The recharge for the dredged footprint including neighboring areas is 19% during the first year. The rates of backfilling in following years is expected to gradually decrease as gradients decrease.
3. Relatively large sediment backfilling also occurs within the Stono Inlet Throat (Area I) with recharging sediment originating from neighboring undredged shallow areas within the inlet. This area was rejected because of potential negative wave impacts to adjacent shorelines.
4. Sand removal of offshore areas (Areas E, J and K) does not have significant impact on sediment transport fields due to weak currents with little backfilling.
5. The dominant longshore current and sediment transport along the Folly Beach nearshore is from the northeast towards the southwest. Modeling results noted a potential node in the longshore current and sediment transport in the northeast direction at the area knows as the Washout.
6. Hurricane Hugo resulted in net sediment loss for both the base conditions and for each borrow area conditions. The largest sediment volume change was a 10% loss around Bird Key in comparing the Base conditions and the Folly River borrow area during Hugo.

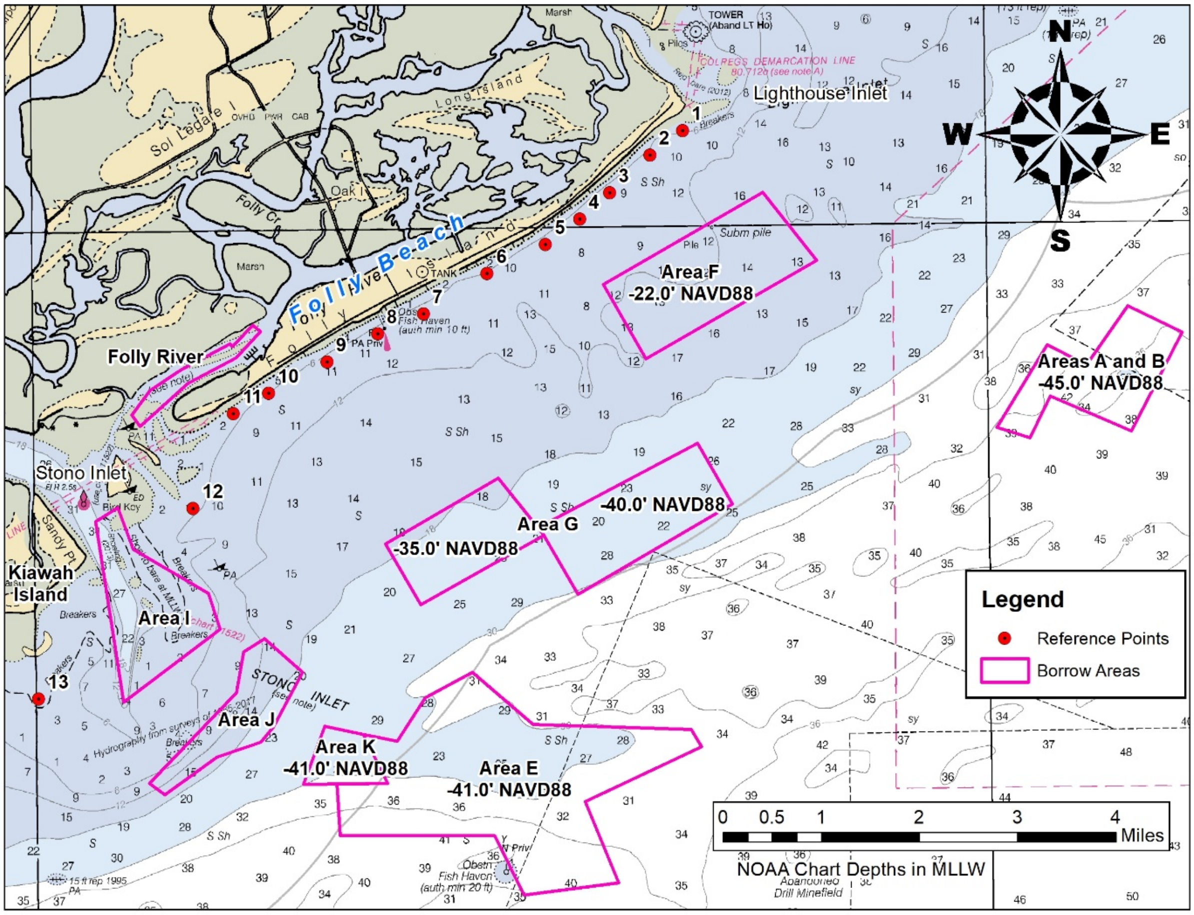


Figure 6-1. Borrow Area Impact Analysis

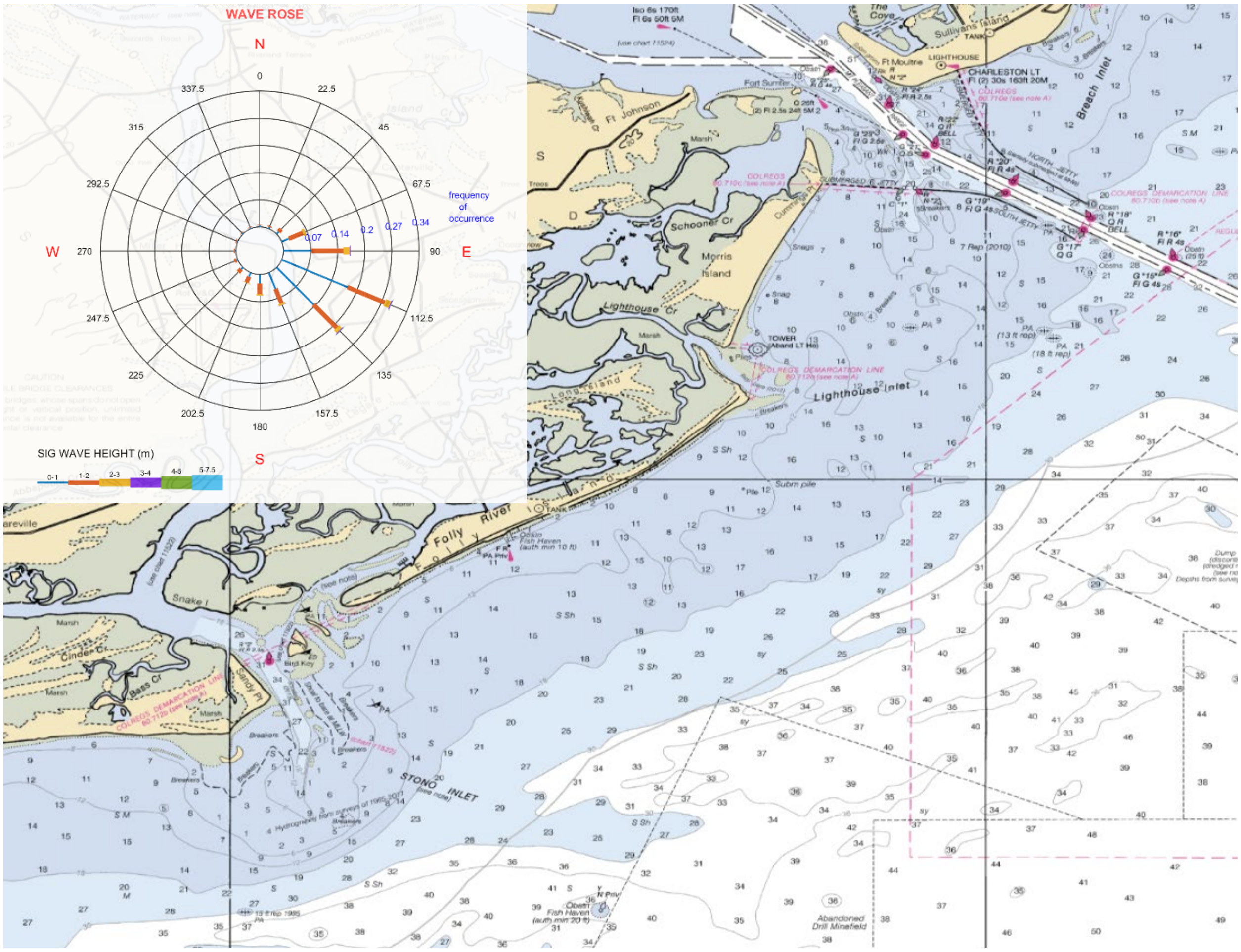


Figure 6-2. Folly Beach Wave Rose Orientation WIS Station #63348

Table 6-1. Borrow Area Impacts – Reference Points

Wave Condition & Direction	Location	Base Condition	Area F - Offshore Lighthouse			Area E & K - Offshore Stono		
		Significant Wave Height (ft)	Significant Wave Height (ft)	Difference (ft)	% Change	Significant Wave Height (ft)	Difference	% Change
Mean60	1	2.52	2.67	0.15	5.8%	2.52	0.00	0.0%
Mean60	2	2.40	2.43	0.03	1.4%	2.40	0.00	0.0%
Mean60	3	2.49	2.46	-0.03	-1.1%	2.49	0.00	0.0%
Mean60	4	2.72	2.45	-0.27	-9.8%	2.72	0.00	0.0%
Mean60	5	2.51	2.28	-0.23	-9.2%	2.51	0.00	0.0%
Mean60	6	2.90	2.72	-0.17	-5.9%	2.90	0.00	0.0%
Mean60	7	2.55	2.69	0.14	5.4%	2.55	0.00	0.0%
Mean60	8	2.36	2.54	0.18	7.5%	2.36	0.00	0.0%
Mean60	9	2.45	2.58	0.13	5.5%	2.45	0.00	0.0%
Mean60	10	2.38	2.44	0.05	2.2%	2.39	0.01	0.3%
Mean60	11	2.74	2.77	0.03	1.1%	2.76	0.02	0.6%
Mean60	12	2.55	2.56	0.01	0.3%	2.62	0.07	2.9%
Mean60	13	3.47	3.47	0.00	0.0%	3.31	-0.16	-4.6%
Mean115	1	3.84	3.82	-0.02	-0.5%	3.84	0.00	0.0%
Mean115	2	3.95	3.91	-0.04	-1.0%	3.95	0.00	0.0%
Mean115	3	4.52	4.37	-0.15	-3.4%	4.52	0.00	0.0%
Mean115	4	4.10	3.91	-0.19	-4.7%	4.11	0.01	0.2%
Mean115	5	4.08	3.81	-0.26	-6.4%	4.09	0.01	0.2%
Mean115	6	4.21	4.39	0.18	4.3%	4.23	0.02	0.5%
Mean115	7	3.92	3.99	0.07	1.9%	3.95	0.03	0.9%
Mean115	8	3.87	3.94	0.07	1.7%	3.91	0.03	0.8%
Mean115	9	3.77	3.82	0.05	1.3%	3.79	0.02	0.6%
Mean115	10	3.99	4.01	0.02	0.5%	3.99	0.00	0.0%
Mean115	11	2.77	2.77	0.00	0.0%	2.77	0.00	0.0%
Mean115	12	2.98	2.98	0.00	0.0%	2.98	0.00	0.0%
Mean115	13	4.68	4.68	0.00	0.0%	4.67	-0.01	-0.2%
Mean170	1	4.23	4.05	-0.18	-4.2%	4.24	0.01	0.3%
Mean170	2	4.21	3.94	-0.27	-6.4%	4.22	0.01	0.3%
Mean170	3	4.35	4.21	-0.14	-3.2%	4.35	0.00	0.0%
Mean170	4	3.94	3.89	-0.04	-1.1%	3.96	0.02	0.6%
Mean170	5	4.61	4.66	0.06	1.2%	4.62	0.02	0.4%
Mean170	6	4.30	4.40	0.10	2.3%	4.29	-0.01	-0.1%
Mean170	7	3.80	3.80	0.00	0.1%	3.76	-0.04	-1.0%
Mean170	8	4.01	4.01	0.00	0.0%	3.95	-0.06	-1.4%
Mean170	9	3.55	3.56	0.00	0.0%	3.50	-0.06	-1.7%
Mean170	10	3.63	3.63	0.00	0.0%	3.58	-0.05	-1.4%
Mean170	11	2.77	2.77	0.00	0.0%	2.77	0.00	0.0%
Mean170	12	2.98	2.98	0.00	0.0%	2.98	0.00	0.0%
Mean170	13	4.52	4.52	0.00	0.0%	4.52	0.01	0.2%
Mean225	1	2.80	2.47	-0.33	-11.9%	2.75	-0.06	-2.1%
Mean225	2	2.90	2.52	-0.37	-12.9%	2.82	-0.08	-2.6%
Mean225	3	2.47	2.46	-0.01	-0.4%	2.38	-0.09	-3.7%
Mean225	4	2.99	3.09	0.10	3.3%	2.89	-0.10	-3.3%
Mean225	5	2.82	3.12	0.30	10.7%	2.72	-0.11	-3.8%
Mean225	6	2.68	2.73	0.05	1.9%	2.57	-0.11	-4.0%
Mean225	7	2.43	2.43	0.00	0.0%	2.38	-0.05	-2.1%
Mean225	8	2.38	2.38	0.00	0.0%	2.36	-0.01	-0.6%
Mean225	9	2.26	2.26	0.00	0.0%	2.27	0.01	0.5%
Mean225	10	2.08	2.08	0.00	0.0%	2.12	0.04	1.8%
Mean225	11	2.34	2.34	0.00	0.0%	2.38	0.03	1.5%
Mean225	12	2.98	2.98	0.00	0.0%	2.98	0.00	0.0%
Mean225	13	4.16	4.16	0.00	0.0%	4.16	0.00	0.0%
Max60	1	4.97	4.97	0.00	0.0%	4.97	0.00	0.0%
Max60	2	4.61	4.61	0.00	0.0%	4.61	0.00	0.0%
Max60	3	5.42	5.41	0.00	0.0%	5.42	0.00	0.0%
Max60	4	6.38	5.95	-0.44	-6.8%	6.38	0.00	0.0%
Max60	5	5.94	5.53	-0.42	-7.0%	5.94	0.00	0.0%
Max60	6	4.69	4.70	0.00	0.1%	4.69	0.00	0.0%
Max60	7	6.22	6.56	0.34	5.5%	6.22	0.00	0.0%
Max60	8	5.72	5.72	0.00	0.0%	5.72	0.00	0.0%
Max60	9	5.78	5.78	0.00	0.0%	5.78	0.00	0.0%
Max60	10	5.06	5.06	0.00	0.0%	5.06	0.00	0.0%
Max60	11	2.78	2.78	0.00	0.0%	2.78	0.00	0.0%
Max60	12	2.99	2.99	0.00	0.0%	2.99	0.00	0.0%
Max60	13	5.35	5.35	0.00	0.0%	5.35	0.00	0.0%

Table 18. continued

Wave Condition & Direction	Location	Base Condition	Area F - Offshore Lighthouse			Area E & K - Offshore Stono		
		Significant Wave Height (ft)	Significant Wave Height (ft)	Difference (ft)	% Change	Significant Wave Height (ft)	Difference	% Change
Max115	2	4.61	4.61	0.00	0.0%	4.61	0.00	0.0%
Max115	3	5.42	5.41	0.00	0.0%	5.42	0.00	0.0%
Max115	4	6.38	6.38	0.00	0.0%	6.38	0.00	0.0%
Max115	5	5.95	5.94	0.00	0.0%	5.95	0.00	0.0%
Max115	6	4.69	4.69	0.00	0.0%	4.69	0.00	0.0%
Max115	7	7.16	7.16	0.00	0.0%	7.16	0.00	0.0%
Max115	8	5.72	5.72	0.00	0.0%	5.72	0.00	0.0%
Max115	9	5.77	5.77	0.00	0.0%	5.77	0.00	0.0%
Max115	10	5.06	5.06	0.00	0.0%	5.06	0.00	0.0%
Max115	11	2.78	2.78	0.00	0.0%	2.78	0.00	0.0%
Max115	12	2.99	2.99	0.00	0.0%	2.99	0.00	0.0%
Max115	13	5.35	5.35	0.00	0.0%	5.35	0.00	0.0%
Max170	1	4.97	4.97	0.00	0.0%	4.97	0.00	0.0%
Max170	2	4.61	4.61	0.00	0.0%	4.61	0.00	0.0%
Max170	3	5.42	5.42	0.00	0.0%	5.42	0.00	0.0%
Max170	4	6.38	6.38	0.00	0.0%	6.38	0.00	0.0%
Max170	5	5.95	5.95	0.00	0.0%	5.95	0.00	0.0%
Max170	6	4.69	4.69	0.00	0.0%	4.69	0.00	0.0%
Max170	7	7.16	7.16	0.00	0.0%	7.16	0.00	0.0%
Max170	8	5.72	5.72	0.00	0.0%	5.72	0.00	0.0%
Max170	9	5.78	5.78	0.00	0.0%	5.77	0.00	0.0%
Max170	10	5.06	5.06	0.00	0.0%	5.06	0.00	0.0%
Max170	11	2.78	2.78	0.00	0.0%	2.78	0.00	0.0%
Max170	12	2.99	2.99	0.00	0.0%	2.99	0.00	0.0%
Max170	13	5.34	5.34	0.00	0.0%	5.34	0.00	0.0%
Max225	1	4.98	4.98	0.00	0.0%	4.98	0.00	0.0%
Max225	2	4.62	4.61	-0.01	-0.1%	4.62	0.00	0.0%
Max225	3	5.41	5.41	0.00	-0.1%	5.41	0.00	0.0%
Max225	4	6.39	6.32	-0.07	-1.1%	6.39	0.00	0.0%
Max225	5	5.96	5.96	0.00	0.0%	5.96	0.00	0.0%
Max225	6	4.70	4.70	0.00	0.0%	4.70	0.00	0.0%
Max225	7	5.40	5.40	0.00	0.0%	5.24	-0.16	-3.0%
Max225	8	5.19	5.19	0.00	0.0%	5.13	-0.05	-1.0%
Max225	9	4.47	4.47	0.00	0.0%	4.49	0.03	0.6%
Max225	10	4.04	4.04	0.00	0.0%	4.15	0.11	2.8%
Max225	11	2.78	2.78	0.00	0.0%	2.78	0.00	0.0%
Max225	12	2.99	2.99	0.00	0.0%	2.99	0.00	0.0%
Max225	13	5.35	5.35	0.00	0.0%	5.35	0.00	0.0%
Extreme	1	5.14	5.14	0.00	0.0%	5.14	0.00	0.0%
Extreme	2	4.76	4.76	0.00	0.0%	4.76	0.00	0.0%
Extreme	3	5.62	5.62	0.00	0.0%	5.62	0.00	0.0%
Extreme	4	6.66	6.66	0.00	0.0%	6.66	0.00	0.0%
Extreme	5	6.19	6.19	0.00	0.0%	6.19	0.00	0.0%
Extreme	6	4.84	4.84	0.00	0.0%	4.84	0.00	0.0%
Extreme	7	7.53	7.53	0.00	0.0%	7.53	0.00	0.0%
Extreme	8	5.95	5.95	0.00	0.0%	5.95	0.00	0.0%
Extreme	9	6.01	6.01	0.00	0.0%	6.01	0.00	0.0%
Extreme	10	5.24	5.24	0.00	0.0%	5.24	0.00	0.0%
Extreme	11	2.83	2.83	0.00	0.0%	2.83	0.00	0.0%
Extreme	12	3.05	3.05	0.00	0.0%	3.05	0.00	0.0%
Extreme	13	5.54	5.54	0.00	0.0%	5.54	0.00	0.0%

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Sub-Appendix A:

Folly Beach

Back Bay Sea Level Change Vulnerability

1.1 Storm Tide and Back Bay Flooding

1.1.1 Back Bay Flooding

Vulnerability to flooding due to sea level change extends beyond Folly Beach and Folly Island, encompassing much of Charleston County. This vulnerability will continue with and be exacerbated by rising sea levels. Shorelines along Folly Island are particularly vulnerable due to low elevation, dense population, and a singular access route.

Folly island is about 6 miles long with a maximum width of 2,800 ft in the center of the island tapering to about 200 ft at the ends. Folly Island is a permanent home to 2,600 residents with a large fluctuating tourist population. Much of the shoreline is generally low-lying and the singular roadway access to the island makes the region highly vulnerable to the potential effects of sea level rise. While historically, residents of South Carolina's coastal communities have been accustomed to thinking of coastal hazards in terms of single event hurricanes or coastal storms it is important to also consider the long-term, sustained effects of SLR on real property, natural habitats, and the ability to sustain growth in the regional economy.

Without adaptation strategies, the following conditions will likely incur substantial social and economic costs:

- Shoreline and beach erosion,
- Flooding of streets, homes, businesses, hospitals, schools, emergency shelters, etc.,
- Impacts to the operations of coastal drainage systems,
- Impairment of coastal water supplies and coastal water treatment facilities and infrastructure,
- Shifts in habitats and reduced ecosystem services.

Regional measurements collected by NOAA show that the Folly Island region is already experiencing sea level rise. There is consensus within the scientific community that this trend will continue. USACE SLC predictions for Charleston are presented in Figure 1. NOAA SLR predictions are compared to USACE predictions in Table 1, the values are relative to a base year of 2006. NOAA sea level inundation predictions in 0.5 ft increments were used to generate graphical depictions of potential flooding extents due to SLR. The 0.5 ft increment maps were then used to extrapolate potential flooding during the 100-year adaptation horizon at the 4 NOAA SLR scenarios. This analysis does not include tidal stage or storm surge effects. Figure 2-Figure 6 show the RSLR maps for Folly Island and the singular roadway leading to the island in 2034, 2054, 2074, 2094, and 2124. Figure 7-Figure 11 show the RSLR maps for Folly Beach in 2034, 2054, 2074, 2094, and 2124.

Currently, coastal areas of Folly Island experience nuisance flooding of low lying areas during King Tide events. By 2034, all four NOAA SLR curves result in some expected flooding, predominantly along the backside of Folly Island in mostly unpopulated areas. By 2054 most of the southern and northern ends of Folly Island are experiencing significant flooding, and the more populated area in the middle of the island is also experiencing flooded effects. By 2074, most of the island is experiencing flooding and storm surge in conjunction with SLC could flood the only access to the island. By 2124, most of the island is experiencing flooding with all SLR scenarios.

Along Folly Island, the raising and widening of the dune system (to 15.0 ft-NAVD88) will help to alleviate some of the flooding potential due to rising sea levels and storm surge. However, this will not protect the low lying regions adjacent to the back bay. The scope of the current study did not include a back bay analysis or consider comprehensive flood control alternatives in addition to the coastal storm risk reduction alternatives considered for Folly Beach. This was due to both the approaching expiration of the existing Federal project and the understanding that the level and physical extent of the vulnerable regions of Folly Island would require a large scale, potentially phased study involving extensive coordination with local authorities and integration of solutions with existing and planned SLR mitigation efforts. This was not possible within the bounds of the current study authority. Instead, the present study focuses on the shoreline component of what will eventually be a comprehensive series of solutions.

Project		USACE			NOAA			
Year	Year	Low	Int	High	Low	Int-Low	Int-High	High
Base	2019	0.06	0.12	0.33	0.04	0.11	0.25	0.41
Start	2024	0.11	0.20	0.49	0.09	0.18	0.38	0.61
	2034	0.21	0.37	0.87	0.19	0.34	0.69	1.09
	2044	0.32	0.56	1.32	0.28	0.52	1.05	1.66
	2054	0.42	0.76	1.85	0.38	0.72	1.48	2.34
	2064	0.52	0.99	2.45	0.47	0.94	1.96	3.12
End	2074	0.63	1.23	3.12	0.57	1.17	2.49	4.01
	2084	0.73	1.48	3.87	0.73	1.48	3.15	5.05
	2094	0.83	1.76	4.69	0.83	1.76	3.81	6.15
	2104	0.94	2.05	5.59	0.94	2.05	4.52	7.35
	2114	1.04	2.36	6.56	1.04	2.36	5.29	8.64
	2124	1.14	2.69	7.60	1.14	2.69	6.12	10.05
50-Year Increase =		0.52	1.03	2.63	0.48	0.99	2.11	3.40
100-Year Increase =		1.03	2.49	7.11	1.05	2.51	5.74	9.44

Table 1. USACE and NOAA SLR predictions (ft NAVD88) for Charleston, SC relative to 2006.

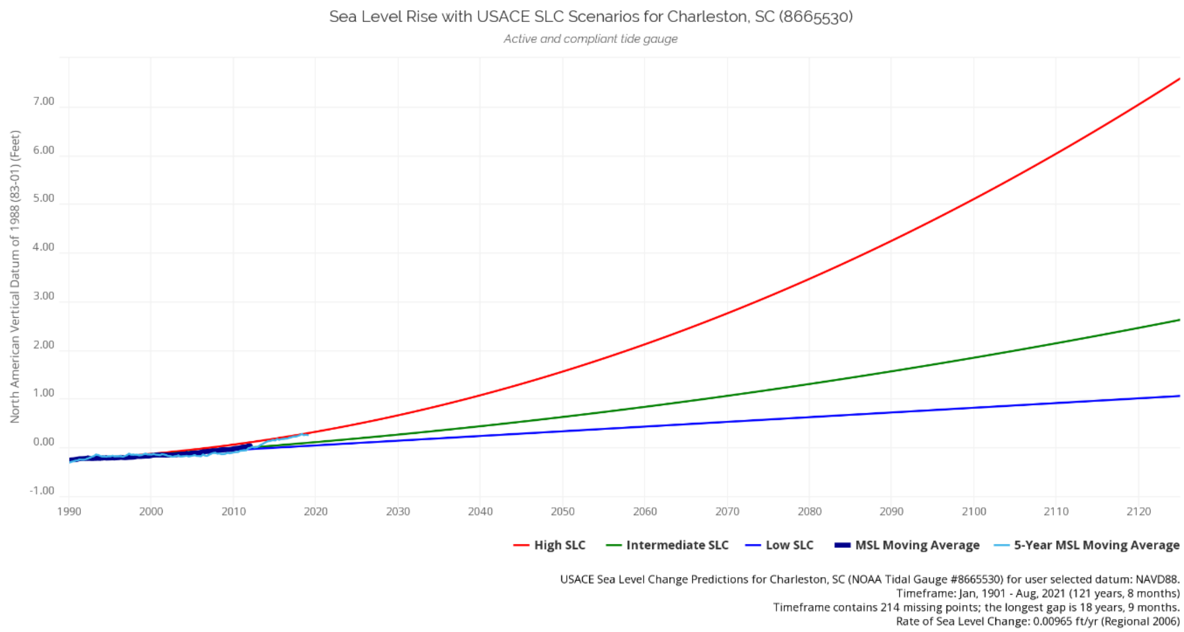

Figure 1. SLR (feet-NAVD 88) Projections for Charleston SC Relative to 2006.

Figure 2. NOAA Projected SLR Impacts (ft NAVD88) for Folly Island in 2034

Figure 3. NOAA Projected SLR Impacts (ft NAVD88) for Folly Island in 2054

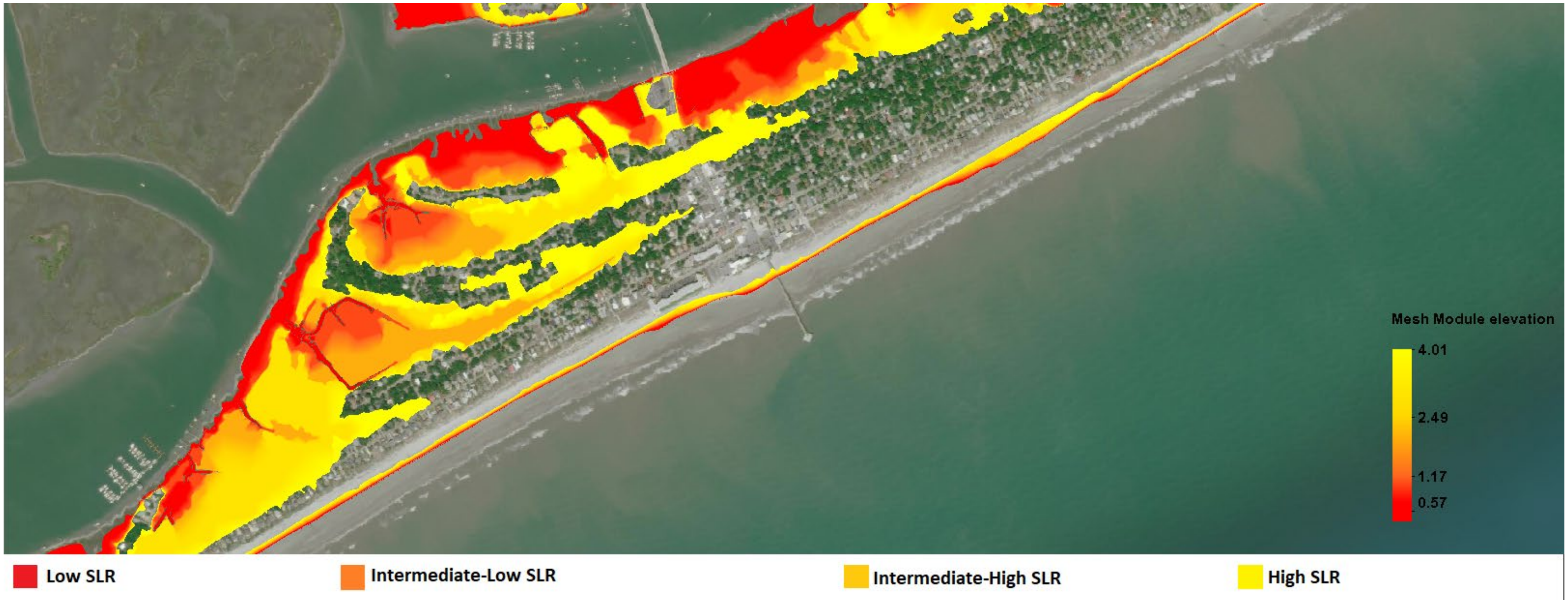

Figure 4. NOAA Projected SLR impacts (ft NAVD88) for Folly Island in 2074.

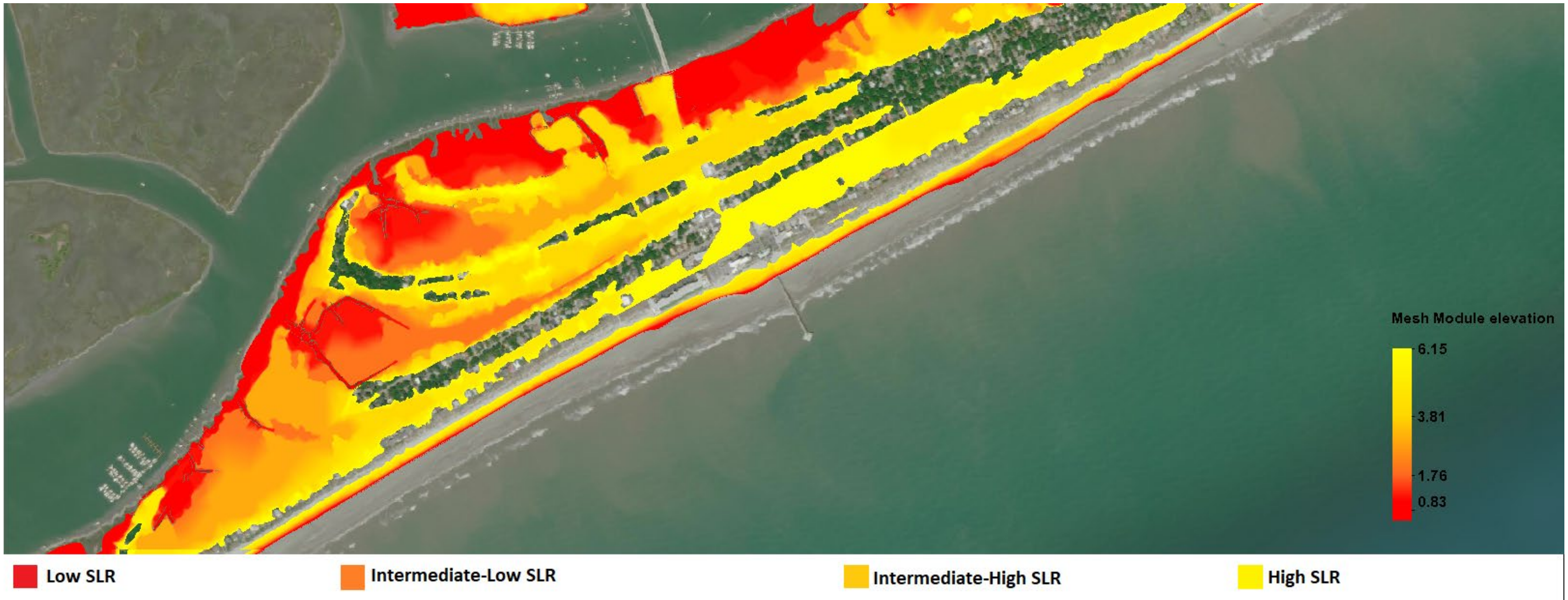

Figure 5. NOAA Projected SLR impacts (ft NAVD88) for Folly Island in 2094.

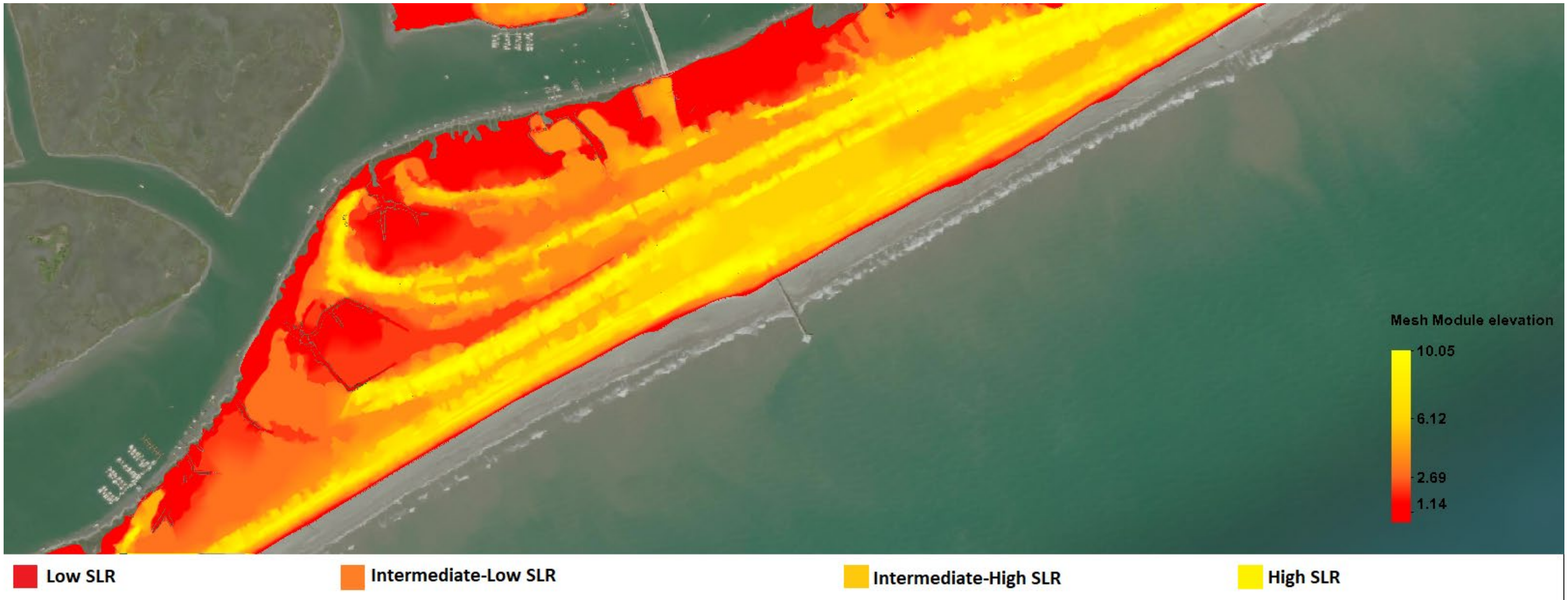

Figure 6. NOAA Projected SLR impacts (ft NAVD88) for Folly Island in 2124.

Figure 7. NOAA Projected SLR impacts (ft NAVD88) for Folly Beach in 2034.

Figure 8. NOAA Projected SLR impacts (ft NAVD88) for Folly Beach in 2054.

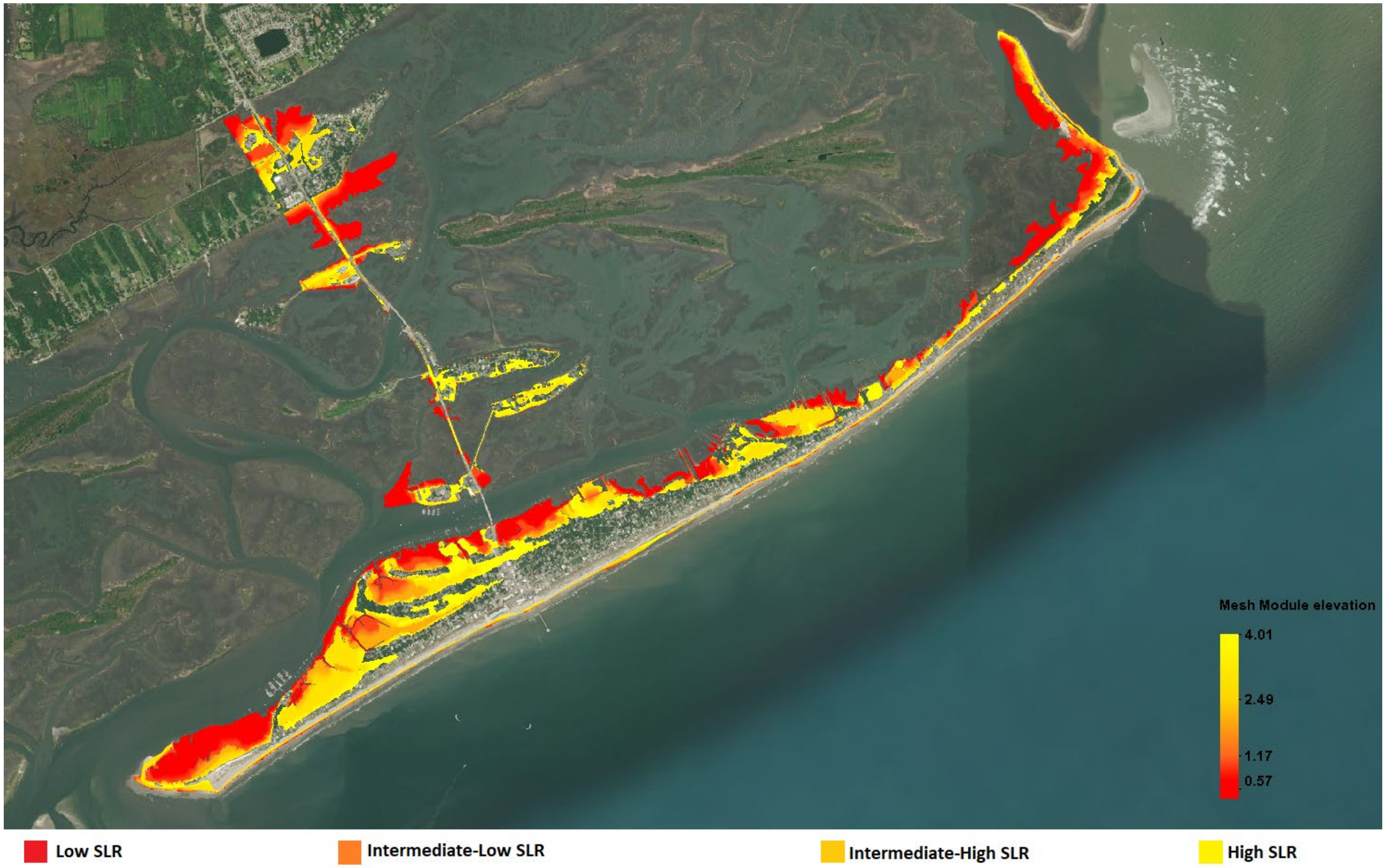

Figure 9. NOAA Projected SLR impacts (ft NAVD88) for Folly Beach in 2074.

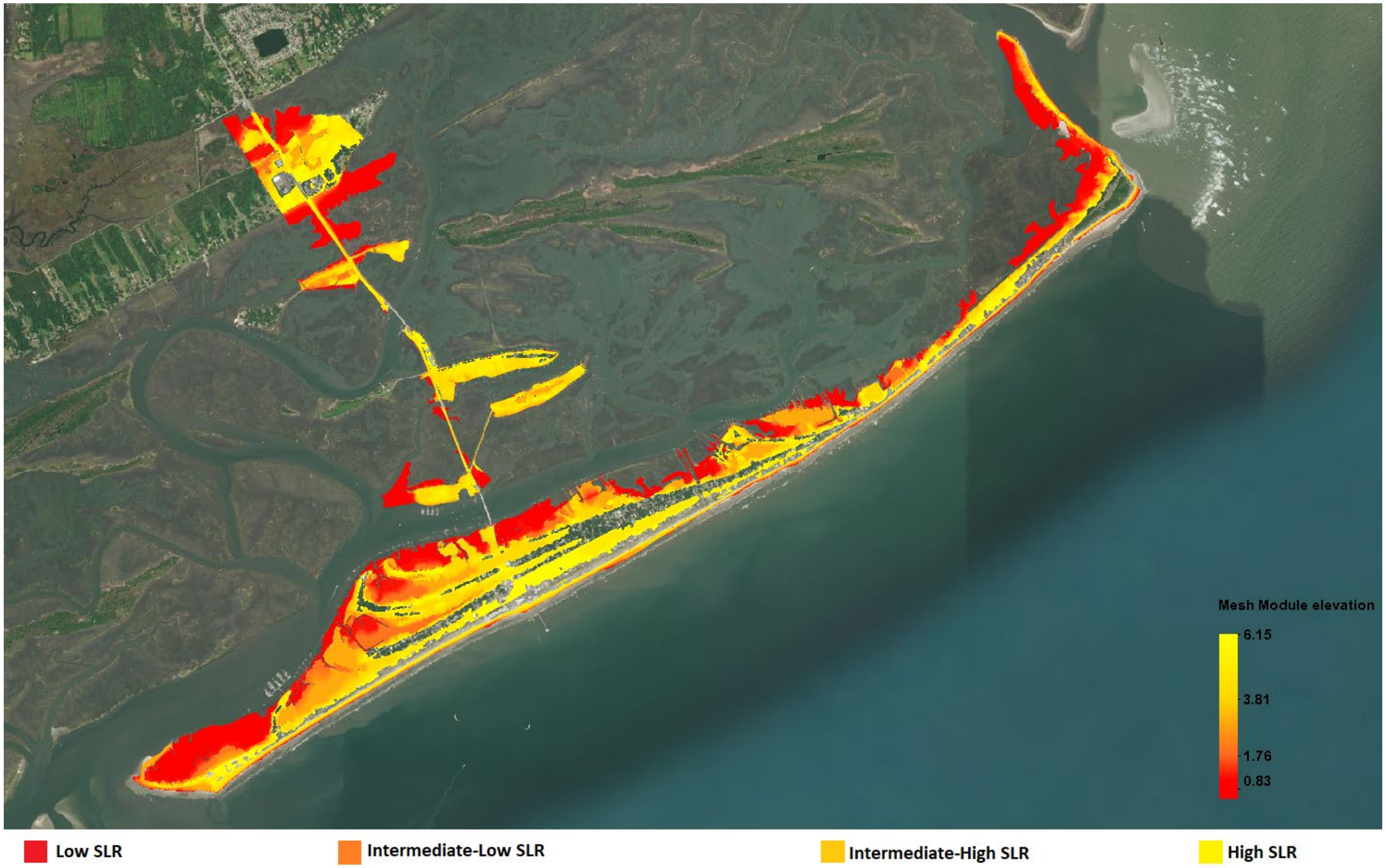

Figure 10. NOAA Projected SLR impacts (ft NAVD88) for Folly Beach in 2094.

Figure 11. NOAA Projected SLR impacts (ft NAVD88) for Folly Beach in 2124.

1.1.2 Critical Thresholds

As sea level rises, low lying areas will begin to experience flooding that will impact critical infrastructure. In the vicinity of the project and the adjacent back bay areas, approximate elevation ranges were determined from available lidar topographic data for residential roads; primary roads; residential, commercial, and service buildings (based on lot elevation); and (on the Sound side of the barrier islands) the shoreline dune system. The infrastructure type and associated approximate elevations (critical thresholds) are presented in Table 2.

The NOAA SLR curves presented for Charleston SC provide higher estimates of sea level rise than the three USACE curves discussed previously (Section 4.6: Sea Level Rise). The Beach-fx model internally applies the USACE curves and the study is being formulated to the USACE Intermediate SLR scenario, however the High SLR curve was used to estimate the years in which the identified critical thresholds are projected to be reached. Based on USACE High SLR projections, the project area and back bay will experience moderate flooding of residential roadways beginning approximately 18 years prior to the end of the 50-year project life (2074). Some of these residential roadways are part of the emergency evacuations routes and flooding could impede evacuation, however the causeway access is not impacted at this level. Beyond the project life and through the 100-year Adaptation Horizon (2124), flooding is expected to extensively impact primary roadways (evacuation routes) and much of the developed regions (first floor elevation) of the island. Initial impacts to causeway access are also present at this level but are not widespread. Most of the households on Folly Island are on septic tanks, and with the High USACE SLR most of the homes will be condemned once the soil is saturated, however the exact impact and timing of this is unknown.

Infrastructure Type	Approximate Elevations (ft-NAVD88)	Year of Impact	
		Initial	Widespread
Residential Roads (Bay Side)	2.5 – 4.0	2064	2104
Residential Roads (Ocean Side)	4.0 – 6.0	2094	2144
Primary/Main Roads	4.0 – 7.0	2094	2164
Causeway Access	4.0 – 7.0	2094	2164
Residential, Commercial, Service Buildings (Bay Side)	5.0 – 7.0	2124	2164
Residential, Commercial, Service Buildings (Ocean Side)	5.0 – 9.0	2124	2194
Shoreline Dune System	0.0 – 15.0 (15.0 post-project)	2019 (NA)	2044 (NA)

Table 2. Critical Thresholds

In general, RSLC (Baseline, Intermediate, and High) will not affect the overall function of the proposed project. Relative vulnerability to flooding during extreme events is consistent between both with and without project conditions. However, low elevation of the interior shore of the island make them highly vulnerable to back bay flooding along with shorelines in the region. Therefore, the primary protection offered by the proposed renourishment project is against localized erosion and wave damages, with limited flood mitigation. Increasing sea levels will increase the back bay vulnerability making it likely that the project will eventually be combined with additional flood management measures.

Sub-Appendix B:

Development of Storm Suite

Storm Suite Development – Folly Beach, SC

Data

Oceanweather (OWI) GROW-FINE U.S. East Coast: Global Reanalysis of Ocean Waves – U.S. East Coast 2018 (GF-EC) reanalysis data was used to develop a storm suite for the region. This dataset consists of 147 historical tropical events over the period 1924– 2017 and 48 extra-tropical cyclones over the period 1957 – 2016. The modeling system consists of the 2-Dimensional hydrodynamic model ADCIRC (ADvanced CIRCulation) and the OWI high-resolution 3rd generation spectral wave model known as OWI3G. This data has been validated (OWI 2018) and previously used in the development of storm suites.

For the Folly Beach, SC storm suite, output point 10452 was used (Figure 1). This output location is located about 4.5 miles southeast of Folly Beach, SC (32.6° N, 79.9° W) at a model depth of 10.25 m. A number of time series of variables were output at this location including: Date, Water Level, Significant Wave Height and Wave Period which were needed for input into the cross-shore change model.

For all datum conversions NOAA Station 8665530, Charleston, Cooper River Entrance was used.

Tropical Storm Selection

The storm selection process followed the general direction of Gravens and Sanderson (2018) Technical Note. In the Technical Note (TN) data from the North Atlantic Comprehensive Coastal Study (NACCS) were used. The NACCS data was developed using a high-fidelity numerical hydrodynamic and wind wave modeling system similar to that used in the OWI study. The main difference is that the NACCS study also examined the water level return period so the associated probability of occurrence for different water levels was available. In the absence of this data for the OWI dataset, instead of 'binning' the storms based on return period, the data were based binned based on an evaluation of storm surge and wave height elevation as discussed below.

The 147 tropical storms in the OWI dataset were separated based on the time/date of occurrence and output interval at the save point. In doing so, 144 unique events were identified. This is because there are a few instances were storms overlap in time, creating longer continuous time series that feature signals from two events. Of the 144 storm time series they were first "de-tided" to remove the influence of the astronomical tide. To accomplish this the U-Tide (Codiga, 2011) Matlab software package was used. Using the NOAA Charleston (8665530) station predicted tide levels, U-Tide was run to create a model of predicted tide level. This model was applied to the date/time of each instance of surge within the OWI record and subtracted from the overall water level to obtain the surge height. Next the peak surge and wave height for each storm was determined and all storms which produced a peak surge height less than 2.0 ft and peak wave height less than 2.0 ft were discarded. While these cut off values are somewhat subjective with the goal of reducing the storm suite to a manageable number of storms, they represent values that are likely to lead to minimal impacts on the shoreline. The Preliminary FEMA Flood Insurance Study (FIS) for Folly Beach (FEMA 2016) this reports 10, 50, 100, and 500 year return period stillwater levels. Along Folly Beach the 10 year return period stillwater level is approximately 5.5

ft NAVD. Likewise NOAA reports exceedance levels at the Charleston Station (8665530) which show a 10 year return period of about 4 ft NAVD. Setting a minimum surge height of 2.0 ft, allows for elimination of many storms with negligible impacts while still ensuring that storms with a return period much more frequent than 10 years is accounted for. This initial screening left 30 storm events for evaluation with peak surge heights between approximately 2.0 and 4.5 ft and peak wave heights between 2 and 28 ft. The storms were then binned based on maximum surge height into 0.5 ft increments. The distribution of storms within the bins is shown in Table 1 and a histogram is shown in Figure 2. Within each bin the hydrographs of each storm were examined. A subsample of the storms were chosen based on the shape of the hydrograph (peaked versus long duration, etc.) and the corresponding wave height (high wave, low wave and average wave conditions). For bins with only a few storms, all storms in the bin were selected. After this evaluation 21 of the 30 storms were selected.

Each of the selected storms was modulated to reflect three statistically defined tide ranges (high, medium, and low amplitude) at four surge-tide phases. The statistically defined tide range reflect the upper quartile, middle half and lower quartile of the tidal ranges. The three tidal ranges and four phase shifts result in 12 plausible total water elevation time series for a single representative storm.

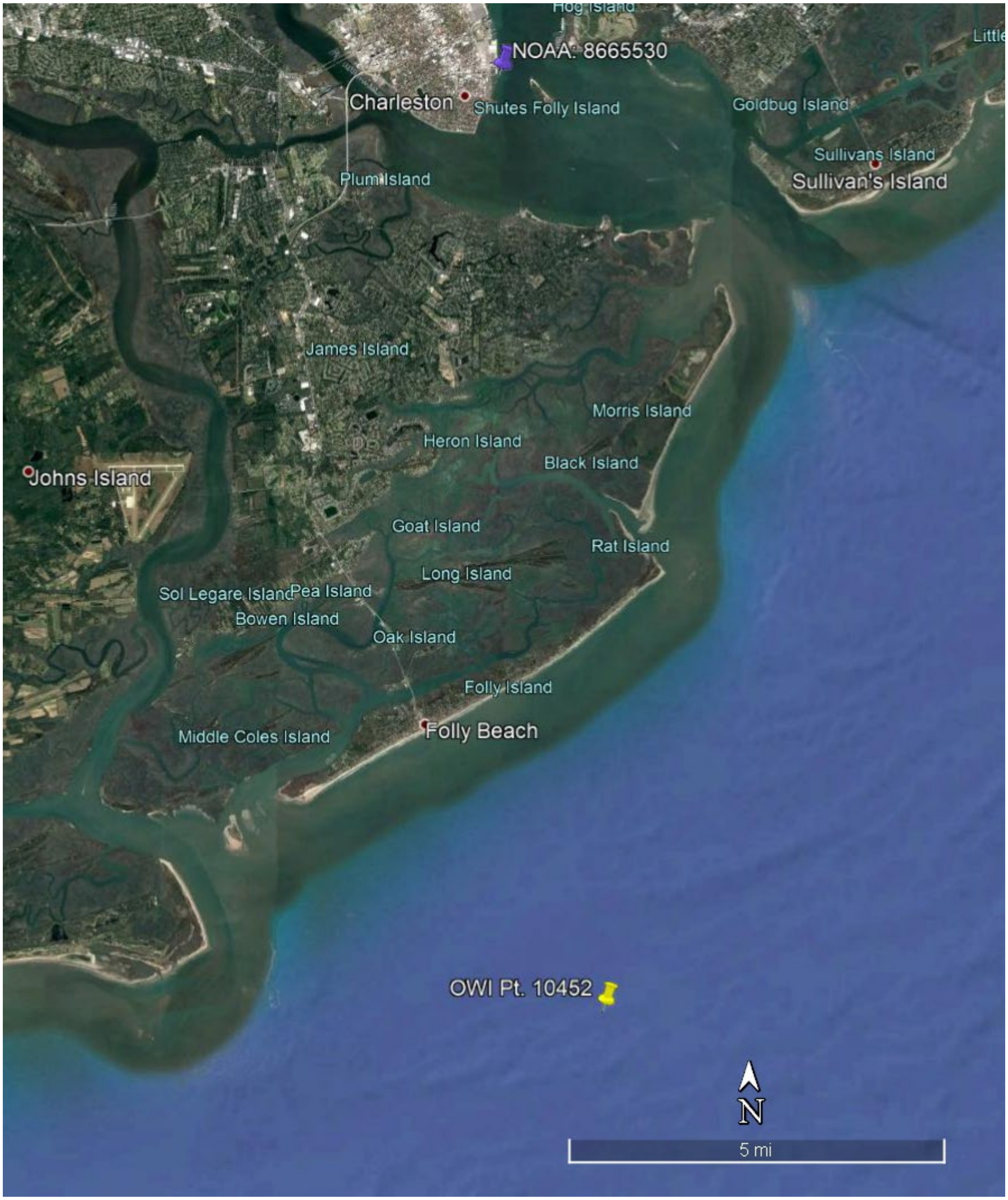

Figure 1: Oceanweather GFEC 2018 grid point 10452 which was used for development of the Folly Beach, SC storm suite. Also pictured is NOAA Station 8665530 which was used to develop the predicted tides and establish datum conversions for the study

Table 1: Distribution of tropical storms by bin

Surge Height (ft)	Number of Storms	Chosen Storms
2.0 – 2.5	12	7
2.5 – 3.0	9	6
3.0 – 3.5	5	4
3.5 – 4.0	1	1
4.0 – 4.5	0	0
4.5 – 5.0	3	3
5.0 – 5.5	0	0
5.5 – 6.0	0	0
6.0 – 6.5	0	0
6.5 – 7.0	0	0
7.0 – 7.5	0	0
7.5 – 8.0	0	0
8.0 – 8.5	0	0
8.5 – 9.0	0	0
9.0 – 9.5	0	0
9.5 – 10.0	0	0
10.0 – 10.5	0	0
10.5 – 11.0	0	0
11.0 – 11.5	0	0
11.5 – 12.0	0	0
12.0 – 12.5	0	0
12.5 – 13.0	0	0
Total	30	21

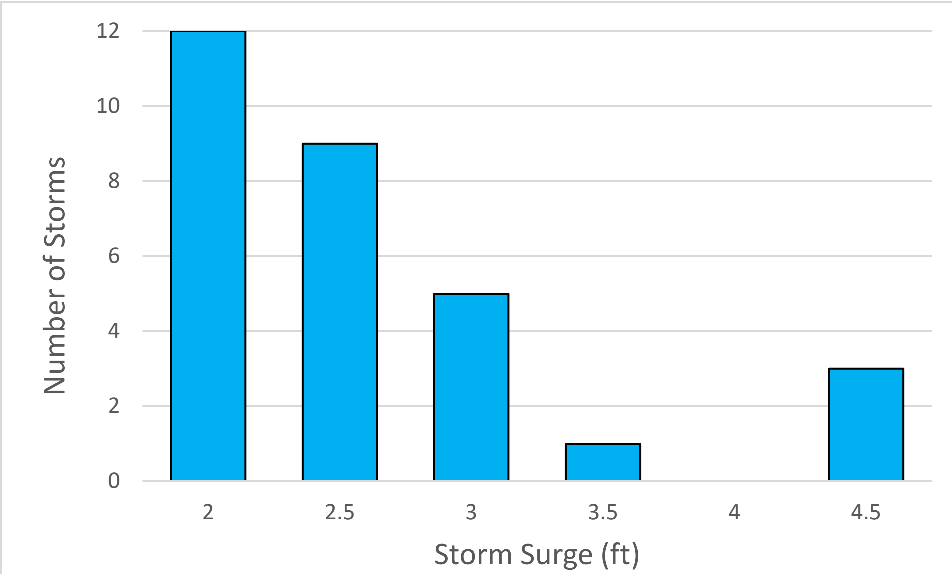

Figure 2: Histogram showing the distribution of peak surge height for tropical storms

Extra-Tropical Storm Selection

The Extra-Tropical storm selection process mirrors the Tropical Storm selection process for Folly Beach. For Folly Beach a total of 48 storms were identified in the record. After applying the same filters of 2.0 ft peak surge and 2.0 peak wave height as applied for the tropical storms the 48 extra-tropical storms was reduced to 25 storms. The remaining storms were binned (Figure 3) and the shape of the hydrograph and peak wave heights were analyzed to ensure the storm suite contained a diverse array of storms. This resulted in a total of 16 extra-tropical storms being selected for the Folly Beach storm suite. A summary of the storm breakdown is presented in Table 2. Similar to the tropical storms, the extra-tropical storms were modulated by the twelve tide conditions resulting in 192 total extra-tropical storm hydrographs.

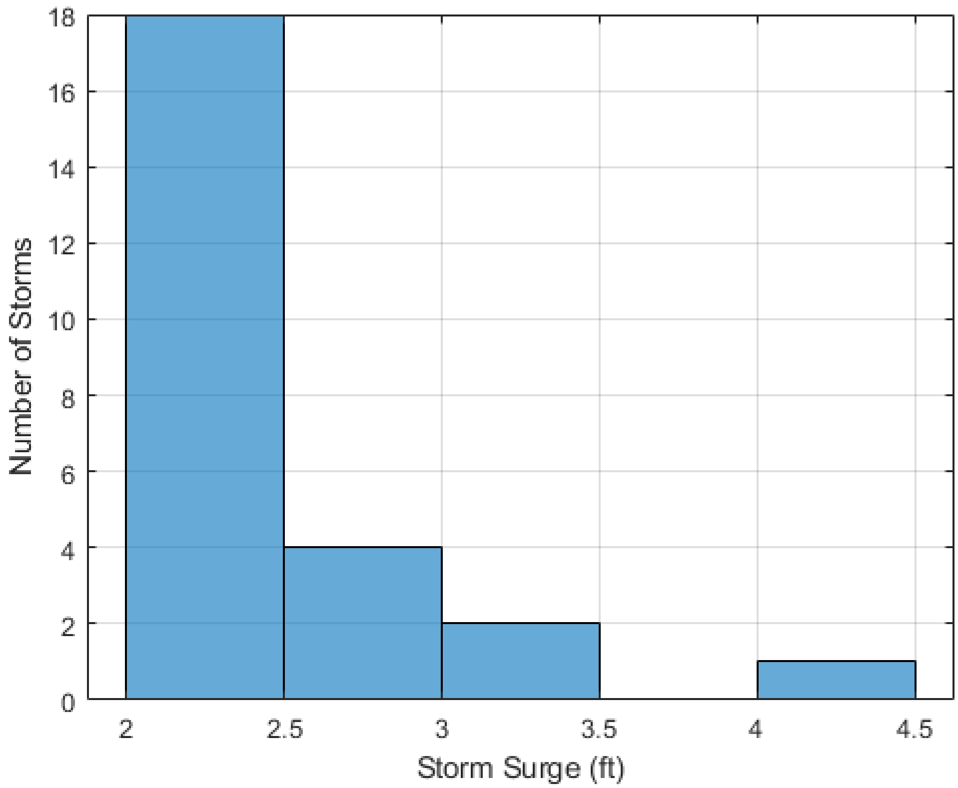

Figure 3: Histogram showing the distribution of peak surge height for extra-tropical storms

Table 2: Distribution of extra-tropical storms by bin

Surge Height (ft)	Number of Storms	Chosen Number
2.0 – 2.5	18	9
2.5 - 3.0	4	4
3.0 - 3.5	2	2
3.5 - 4.0	0	0
4.0 - 4.5	1	1
Total	25	16

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Sub-Appendix C:

Folly Beach Shoreline Change Rate Analysis

GenCade Planform Rates

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MEMORANDUM

To: John Hazelton, P.E., Kevin Conner, P.E., USACE Wilmington District Office

From: Jeff Shelden, P.E., Yong Chen, Ph.D, P.E., Brian Joyner, P.E.

Date: June 16, 2020

Subject: Folly Beach Shoreline Change Rate Analysis

M&N Job No.: 10514-01

1. Introduction

1.1. Study Purpose and Scope

Moffatt & Nichol (M&N) was contracted by USACE Water Resources Section, Wilmington District Office to conduct the Folly Beach shoreline change rate analysis. The objective of this memo is to compile and query historical data necessary to develop and calibrate a planform evolution model and to conduct analytical calculations relative to post-nourishment shoreline change rates of Future With Project (FWP) scenarios in the various reaches of the project area.

The currently authorized federal coastal storm risk management project at Folly Beach, South Carolina, was initially constructed in 1993 and to date has been nourished seven times including initial construction. Due to a sediment deficiency and the desire to optimize the project, a General Revaluation Report was authorized to locate additional offshore borrow sources and determine optimal project templates for the project.

A component of the feasibility study is the Beach-fx economic model of economic damages over time to structures and infrastructure along the town's shoreline. The project area being evaluated in Beach-fx consists of approximately 5.47 miles of shoreline (See Figure 1-1). Among other inputs, Beach-fx requires estimates of shoreline change rates for existing conditions, and Future With Project conditions.

1.2. Engineering Study Approach

Existing Data Collection and Review

M&N compiled and reviewed existing available data sets regarding profile conditions, shoreline positions, waves, tides and sediment characteristics in the study area. The existing available data sets include:

- Beach profiles (see Figure 1-2) and other survey data
- Existing groin locations (see Figure 1-3), length and elevation
- Historical reports and data sets describing beach and nearshore sediment characteristics
- Tidal levels at NOAA tidal station of Charleston, Cooper River Entrance, SC, ID: 8665530
- USACE WIS wind and wave hindcast data sets
- NOAA WaveWatch3 (WW3) wind and wave hindcast data sets
- Wave data sets at NOAA buoy #41004
- Oceanweather hindcast data at station 10452
- Wind data sets at NDBC FBIS1

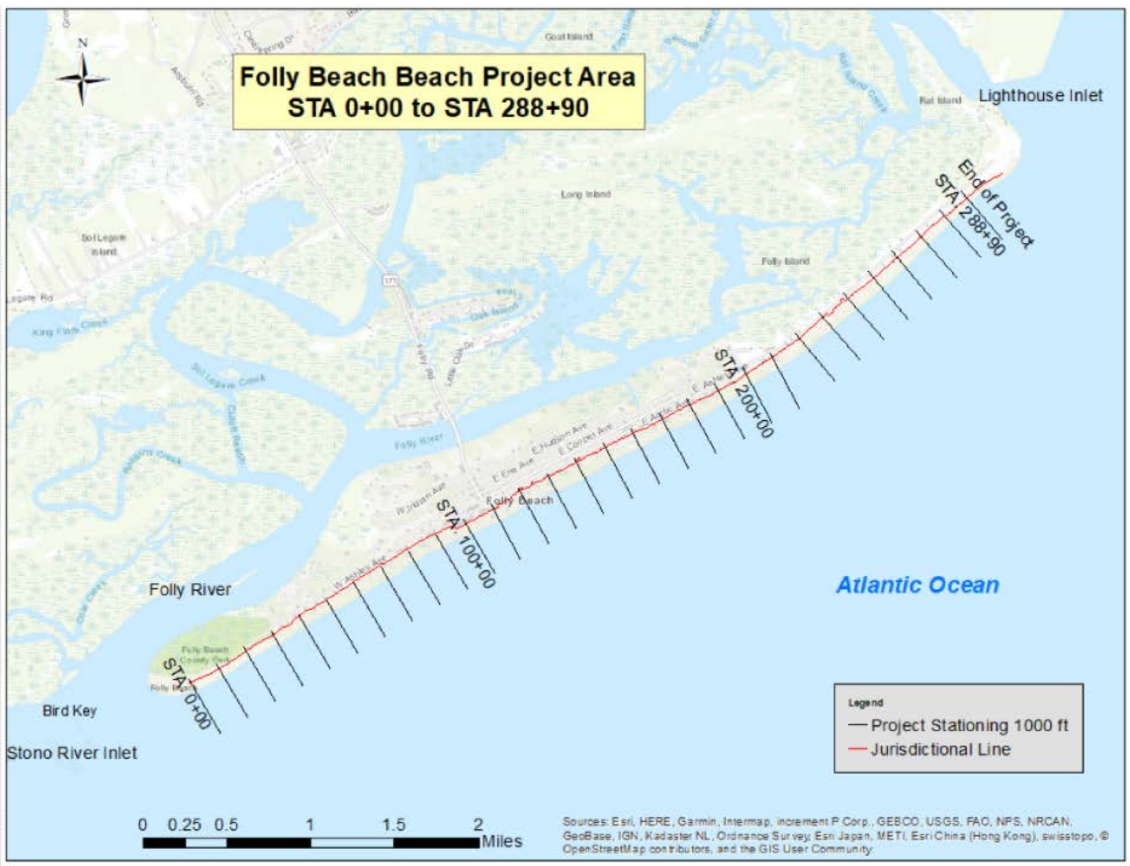

Figure 1-1: Folly Beach Project Area

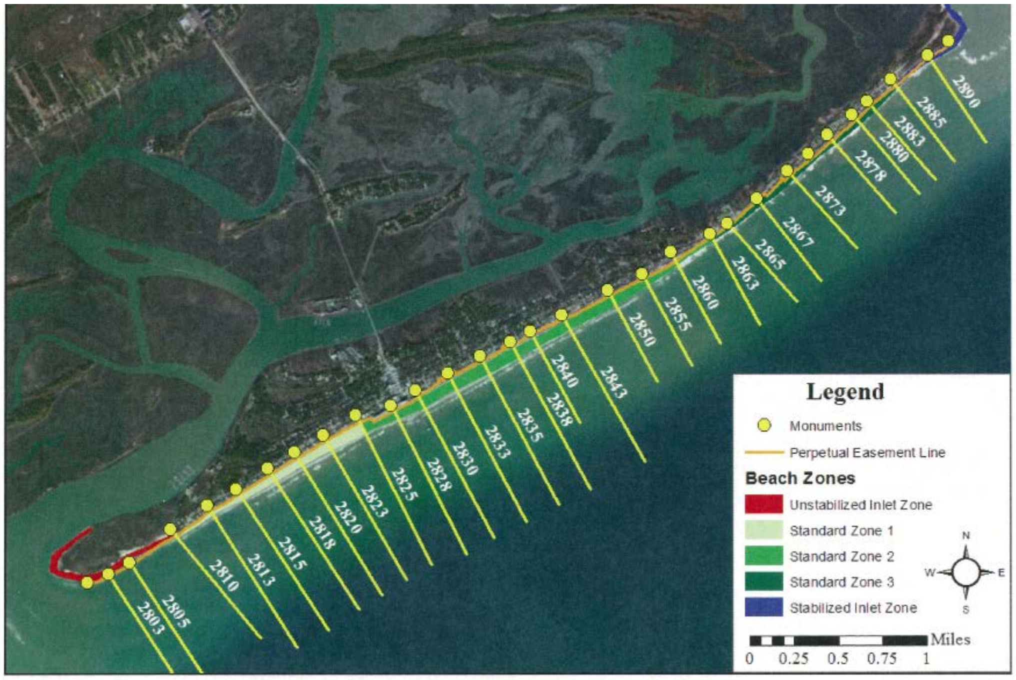

Figure 1-2: Monument Locations

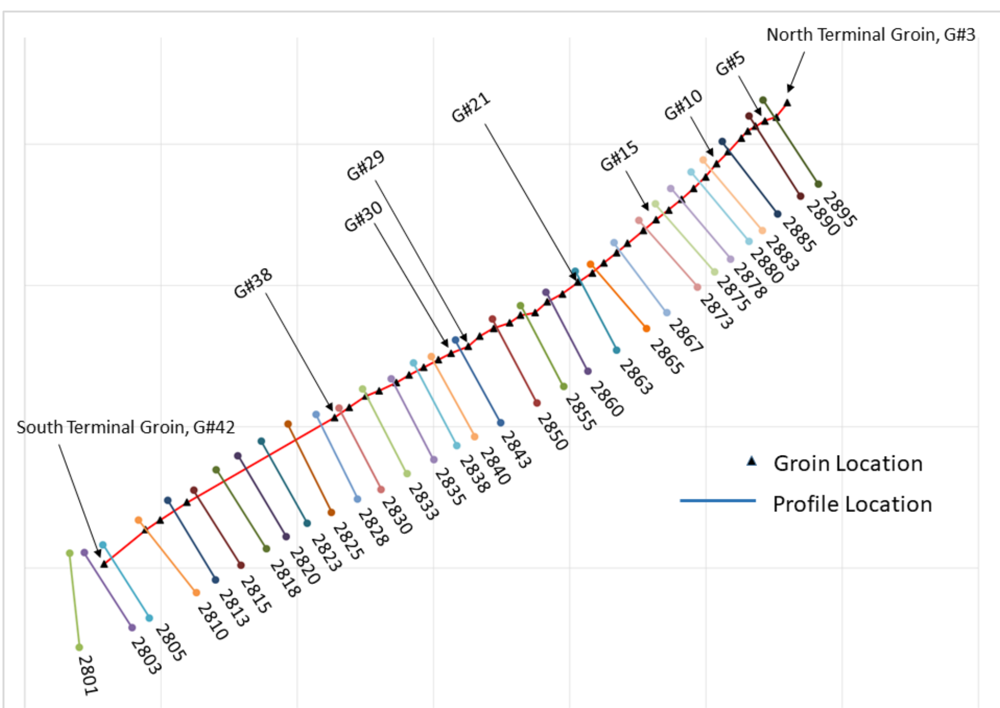

Figure 1-3: Monument Locations and Groin Locations

Wave Transformation Modeling

Evaluation of the shoreline change rates depends on expected wave conditions and associated longshore sediment transport rates. M&N applied a numerical spectral wave model, the MIKE 21 Spectral Waves software by DHI, to transform a continuous time series of offshore wave conditions to a high-resolution grid of nearshore locations along the study area.

The spectral wave model was calibrated and validated using the NDBC buoy wave data, Oceanweather data, and WIS wave data. The calibrated and validated spectral wave model was used to transform time series of offshore waves to the nearshore project site.

One-Line Shoreline Model Calibration

M&N applied the USACE GenCade one-line shoreline evolution model for the Folly Beach shoreline change rate analysis. The GenCade shoreline model was developed utilizing the shoreline locations, existing groin fields at Folly Beach including the terminal groin on the north and south ends of Folly Island, and model simulated nearshore wave conditions. The shoreline model was calibrated and validated using the historical shoreline positions and historical longshore sediment transport rates.

Estimation of Shoreline Change Rates of Future With Project Scenarios

Four Future With Project scenarios were designed by USACE and provided to M&N. M&N applied the calibrated/validated shoreline evolution model to estimate post-nourishment shoreline position changes in representative segments of the project shoreline for representative “typical annual” wave conditions. M&N utilized the results to estimate and recommend representative shoreline change rates for each of the Beach-fx reaches for each of the Future With Project scenarios. The following project features were evaluated:

- Beach nourishment project constructed in January 2024
- Beach nourishment project constructed in January 2036
- Beach nourishment project constructed in January 2048
- Beach nourishment project constructed in January 2060

2. Data Collection and Review

M&N collected and reviewed the existing data sets including water levels, winds, waves, beach profiles and beach nourishment projects as presented in Table 2-1.

Table 2-1: Timeline of the Historical Data

	Year	80	90	98	99	00	01	02	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18
Water level	NOAA 8665530																							
Wind	NDBC FBIS1																							
	WIS Wind																							
	WW3 Wind																							
Wave	WIS Wave																							
	WW3 Wave																							
	NDBC 41004																							
	Oceanweather GROW-FINE																							
Profile																								
Beach nourishment																								
Possible beach nourishment																								
Potential shoreline calibration period																								

The following periods for the shoreline change model calibration and validation were recommended by M&N and approved by the USACE, in order to avoid the near-term effects of beach nourishment projects on the measured shorelines:

- December 2008 to March 2010
- March 2010 to December 2012
- December 2016 to December 2017

Figure 2-1 shows the collected wave data locations. In Figure 2-1, the OW10452 represents the Oceanweather data station 10452, WIS63348 represents USACE WIS wave data station 63348, the NDBC41004 represents NDBC buoy 41004 data location, and WW3 represents WaveWatch 3 data location.

Figure 2-1: Wave Data Locations

3. Spectral Wave Modeling

The wave transformation study was conducted utilizing the MIKE 21 Spectral Waves (SW) model (DHI, 2014) to calculate wave conditions approaching Folly Beach. MIKE 21 SW simulates the growth, decay and transformation of wind-generated waves and swells in offshore and nearshore coastal areas. M&N developed and calibrated a spectral wave model, then utilized the wave model to transform waves from offshore to the nearshore project area.

The model domain computational mesh resolution and model bathymetry are illustrated in Figure 3-1 and Figure 3-2. The horizontal mesh resolution varies between approximately 2.5 miles offshore and approximately 100 feet at study area.

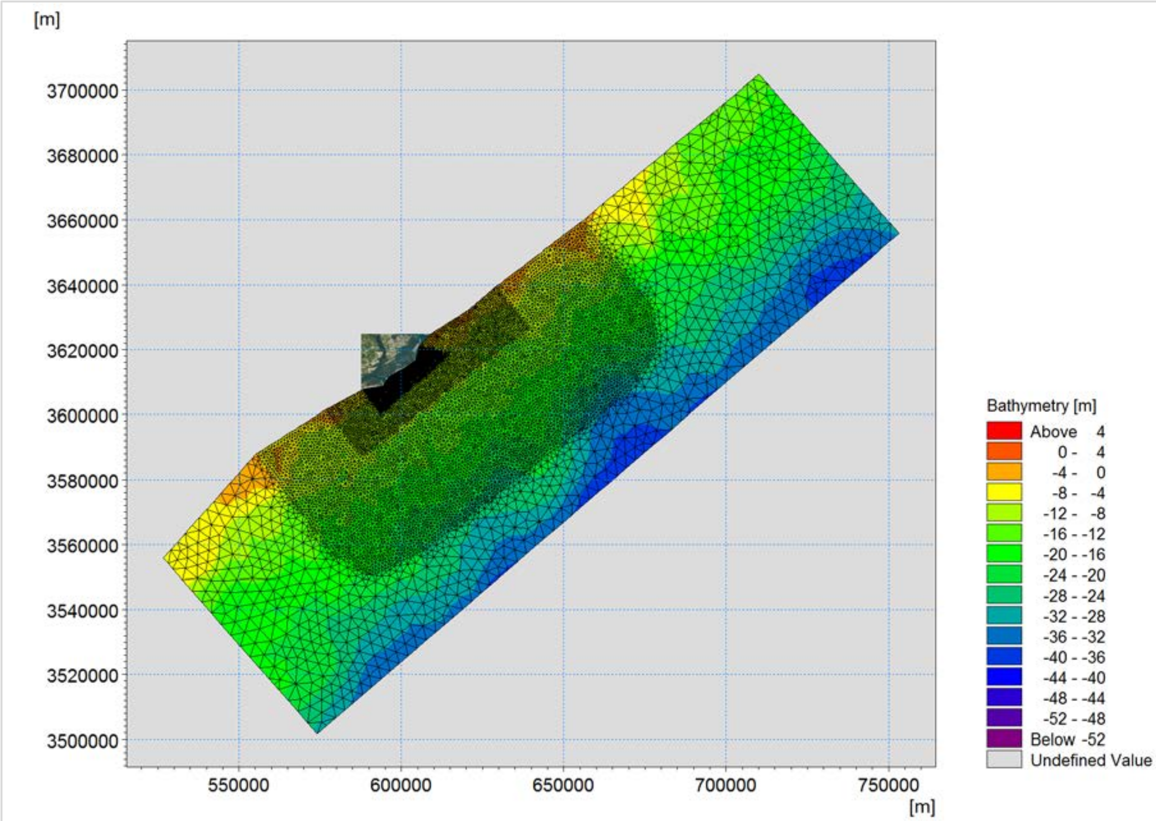

Figure 3-1: Spectral Wave Model Mesh (Entire Area)

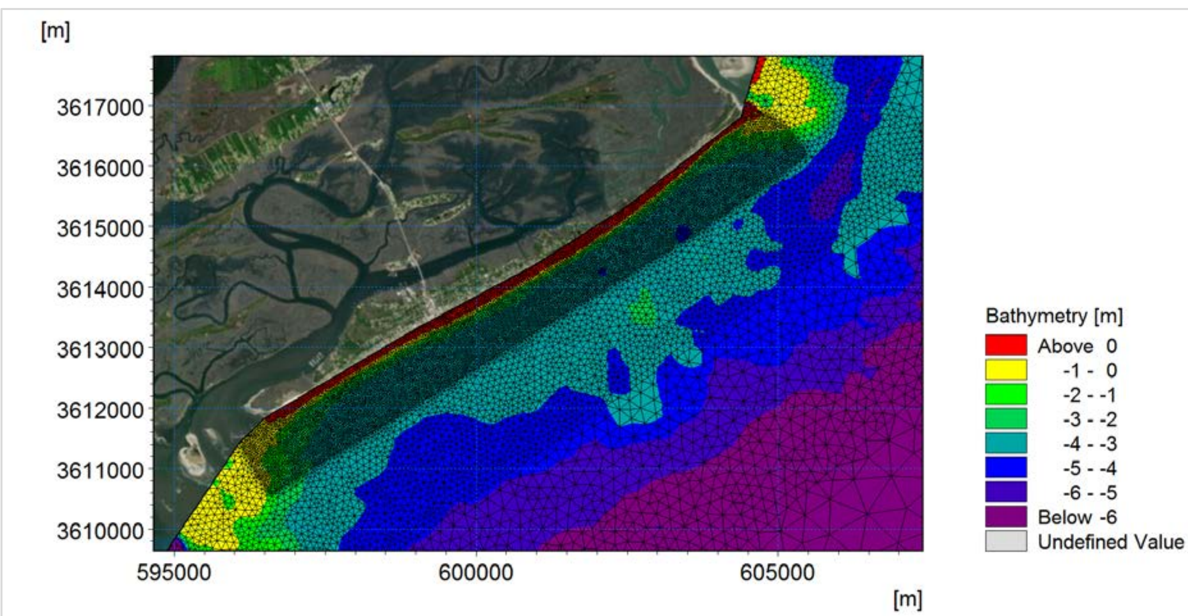

Figure 3-2: Spectral Wave Model Mesh (Project Site)

3.1. Wave Model Calibration and Validation

The MIKE 21 SW wave model used NOAA hydrographic data, profile data, water level time series, and wind and wave data as inputs to the wave transformation simulation. The calibrated model parameters for the spectral wave model are presented in Table 3-1. For the wave model calibration, the NDBC 41004 buoy data were used as open boundary conditions. The calculated wave heights, wave periods and wave directions were compared with the Oceanweather data at station 10452 for two selected storms as shown in Figure 3-3 through Figure 3-8.

Table 3-1: MIKE 21 Spectral Wave Model Parameters

Parameter Name	Type	Value
Frequency discretization logarithmic	Number of frequencies	12
	Minimum frequency	0.07 Hz
	Frequency factor	1.15
Directional discretization	Directional sector, number of directions	18
Wind forcing	Coupled, Charnock parameter	0.012
Wave breaking	Functional form, Ruessink et al. (2003)	1
Bottom friction	Friction factor, f_w	0.03
White Capping	Constant	3.0, 0.6
Solution method	Newton-Raphson iteration, low order	

Generally, the M&N calculated wave heights, wave periods and wave directions agree with the Oceanweather hindcasted wave data at station 10452. The Oceanweather wave data curves show some data smoothing. In the wave model calibration, the measured wave data were used as open boundary conditions. Therefore, the M&N calculated wave conditions could be more accurate compared to the Oceanweather hindcasted wave data.

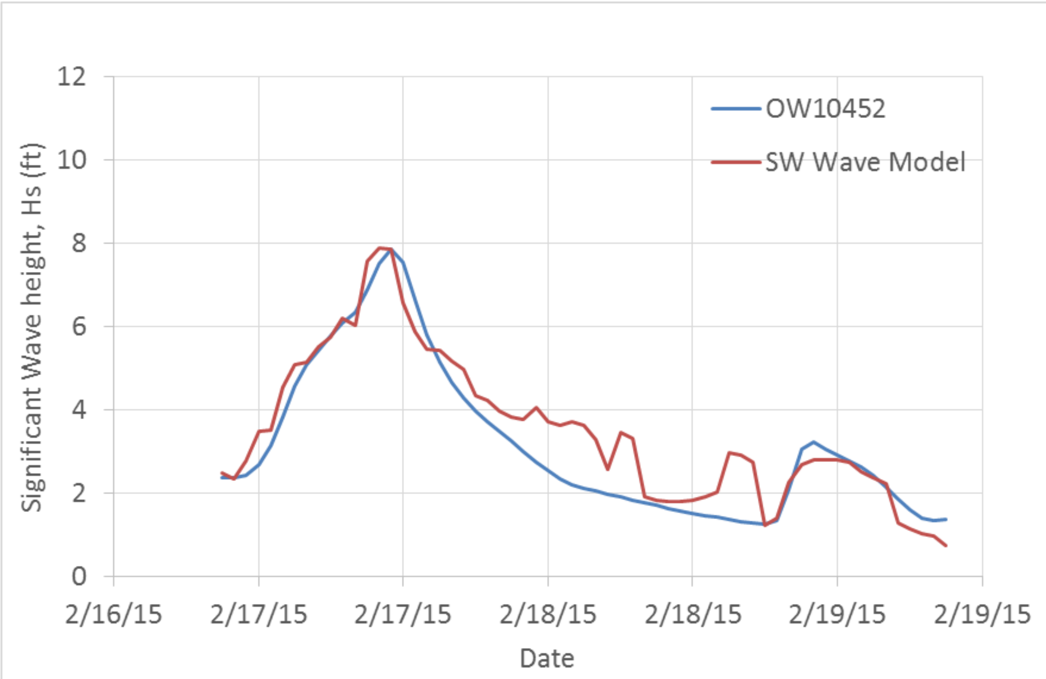

Figure 3-3: Calculated Wave Heights Compared to OW10452 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

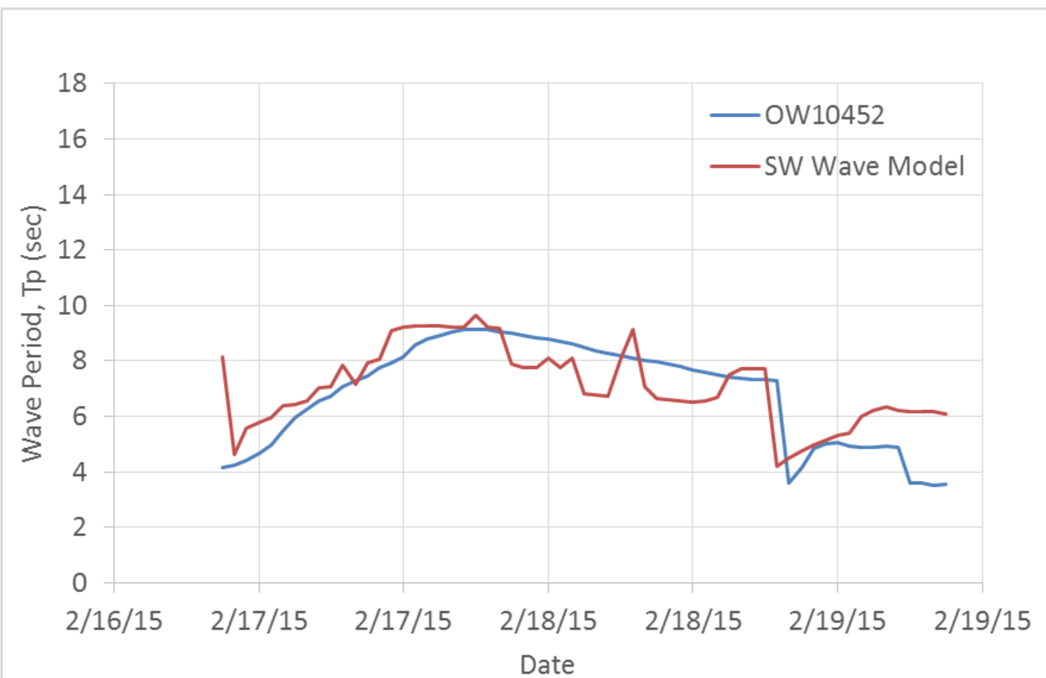

Figure 3-4: Calculated Wave Periods Compared to OW10452 Wave Periods (Open Boundary Conditions: NDBC 41004 Buoy Data)

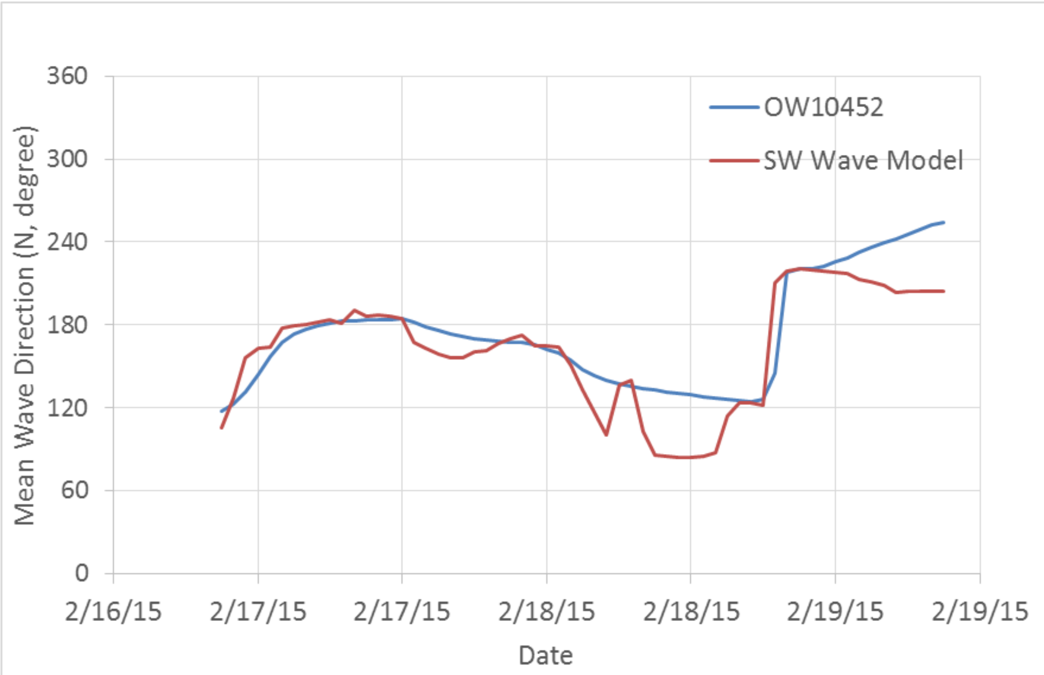

Figure 3-5: Calculated Wave Directions Compared to OW10452 Wave Directions (Open Boundary Conditions: NDBC 41004 Buoy Data)

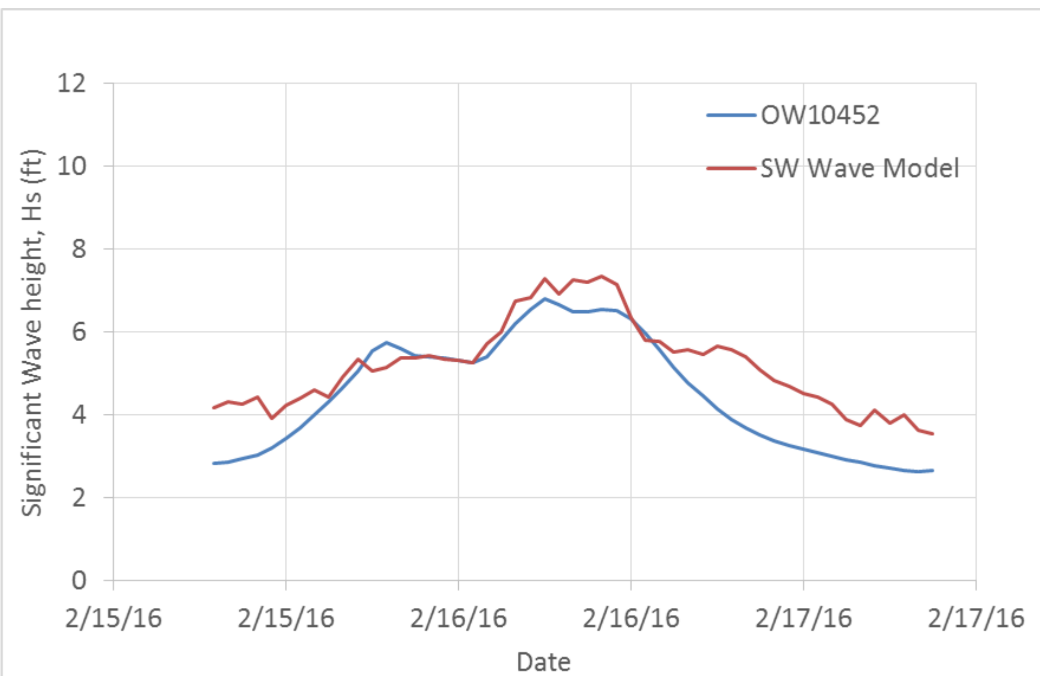

Figure 3-6: Calculated Wave Heights Compared to OW10452 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

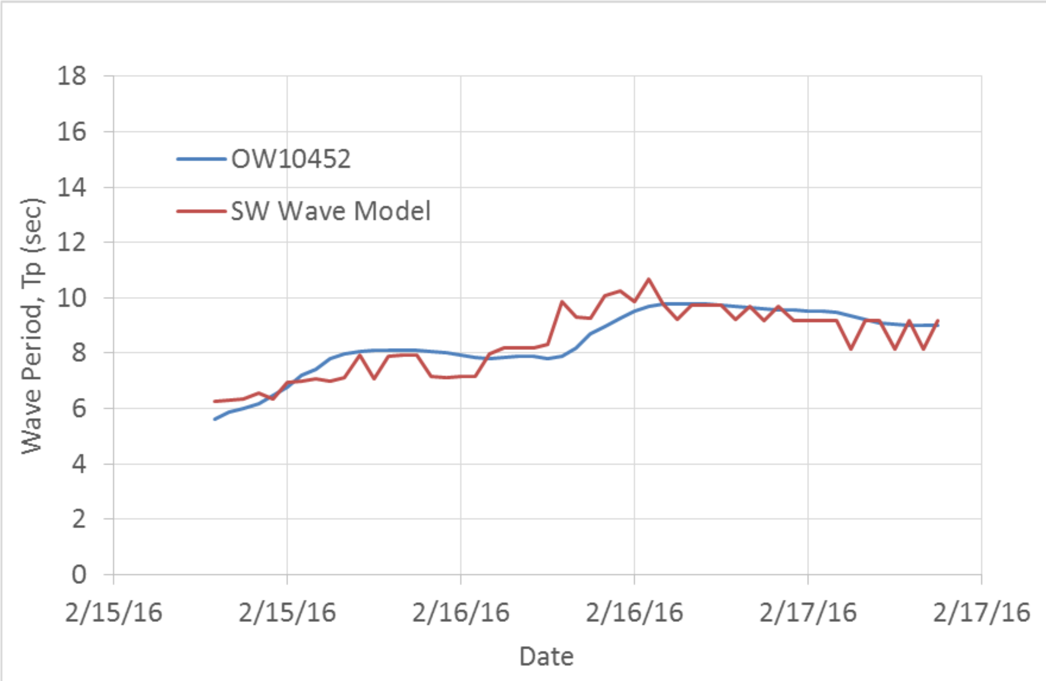

Figure 3-7: Calculated Wave Periods Compared to OW10452 Wave Periods (Open Boundary Conditions: NDBC 41004 Buoy Data)

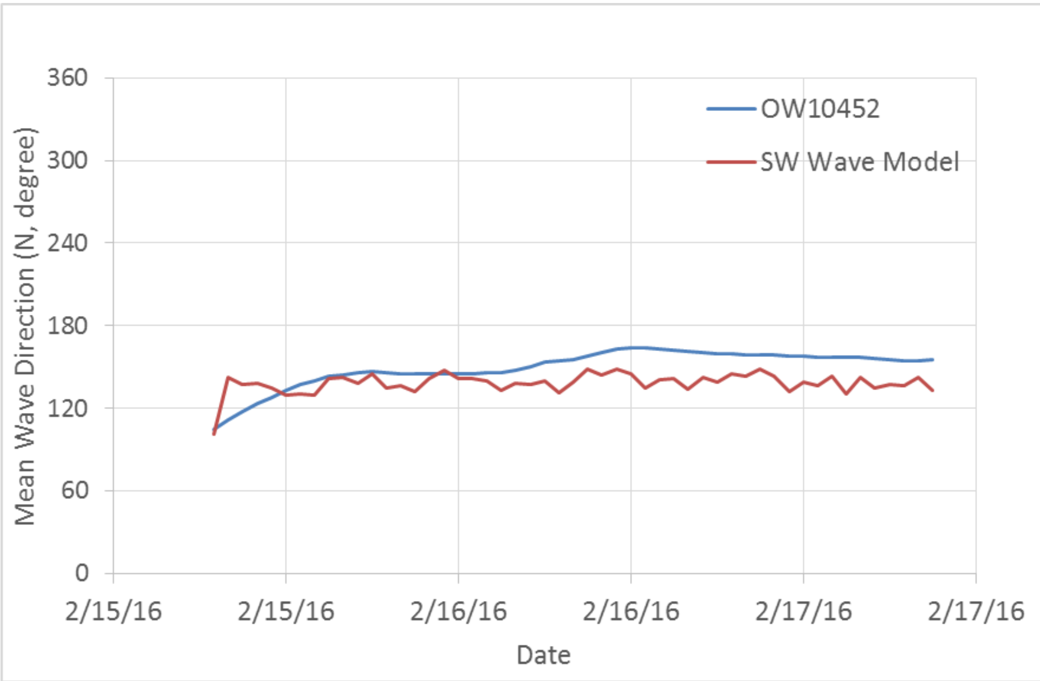

Figure 3-8: Calculated Wave Directions Compared to OW10452 Wave Directions (Open Boundary Conditions: NDBC 41004 Buoy Data)

The same wave model parameters were validated for the following two cases:

- Using the NDBC 41004 buoy data as open boundary conditions, and comparing calculated wave climates to the WIS wave data at station 63348 (see Figure 3-9 through Figure 3-14)
- Using the WW3 wave data as open boundary conditions, and comparing calculated wave climates to the NDBC 41004 buoy data (see Figure 3-15 through Figure 3-20)

Generally, the wave model calibration and validation results indicate the degree to which the wave transformation model agrees with the wave gauge data and existing wave model data. Figure 3-9 and Figure 3-18 show large peak wave height discrepancies in a short duration. In Figure 3-9, the measured data had a short-duration higher peak wave height while the WIS data had a smoothed lower peak wave height. In Figure 3-18, the calculated storm shape matches well to the measured data, however, the measured data caught a short-duration higher peak wave height. The wave model wave height results indicate that directly applying measured wave data as open boundary conditions would get more accurate short-duration peak wave heights at nearshore area.

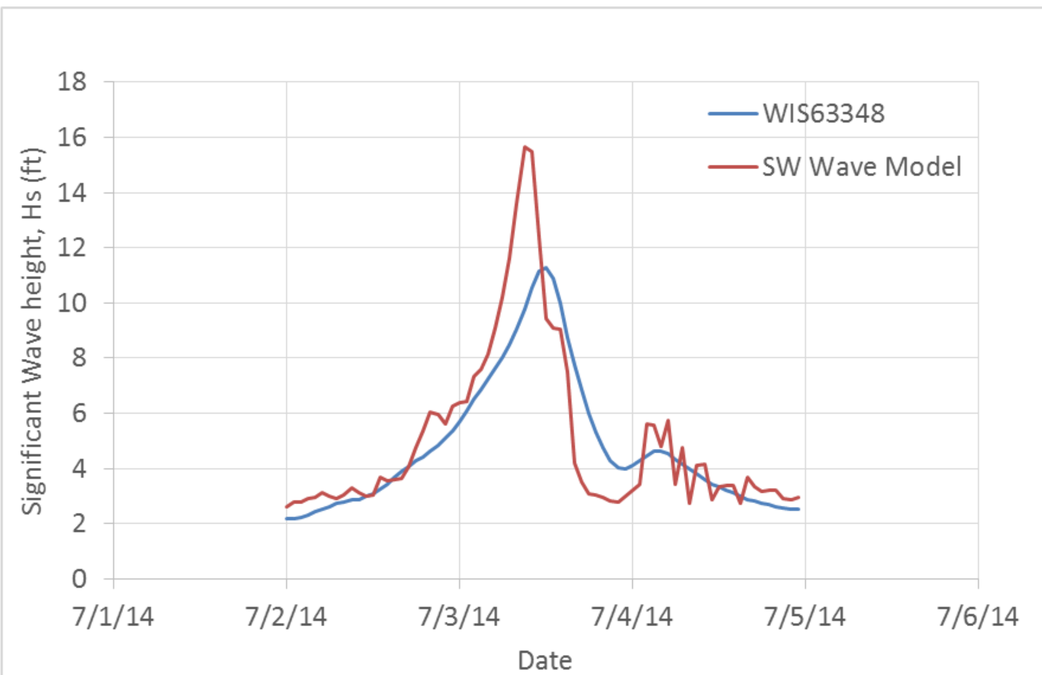

Figure 3-9: Calculated Wave Heights Compared to WIS63348 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

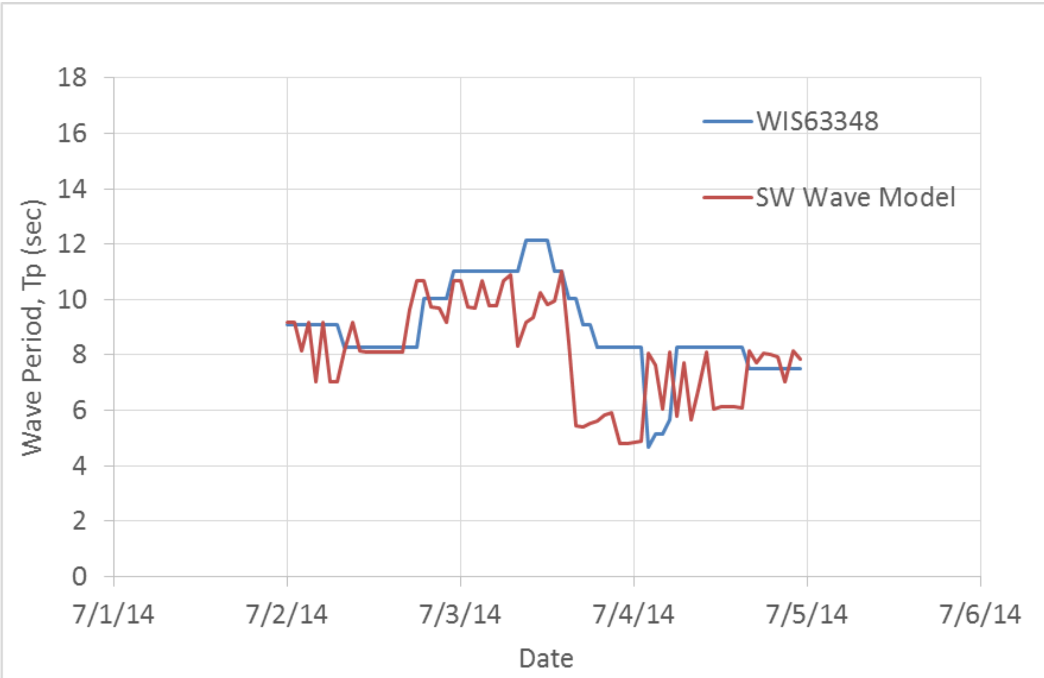

Figure 3-10: Calculated Wave Periods Compared to WIS63348 Wave Periods (Open Boundary Conditions: NDBC 41004 Buoy Data)

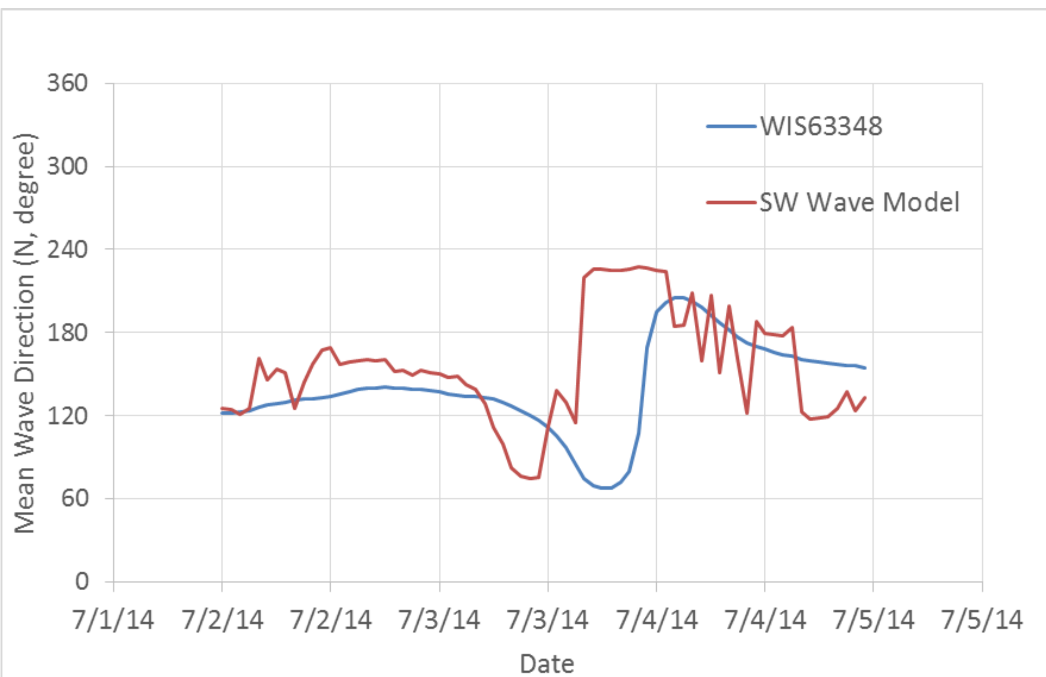

Figure 3-11: Calculated Wave Directions Compared to WIS63348 Wave Directions (Open Boundary Conditions: NDBC 41004 Buoy Data)

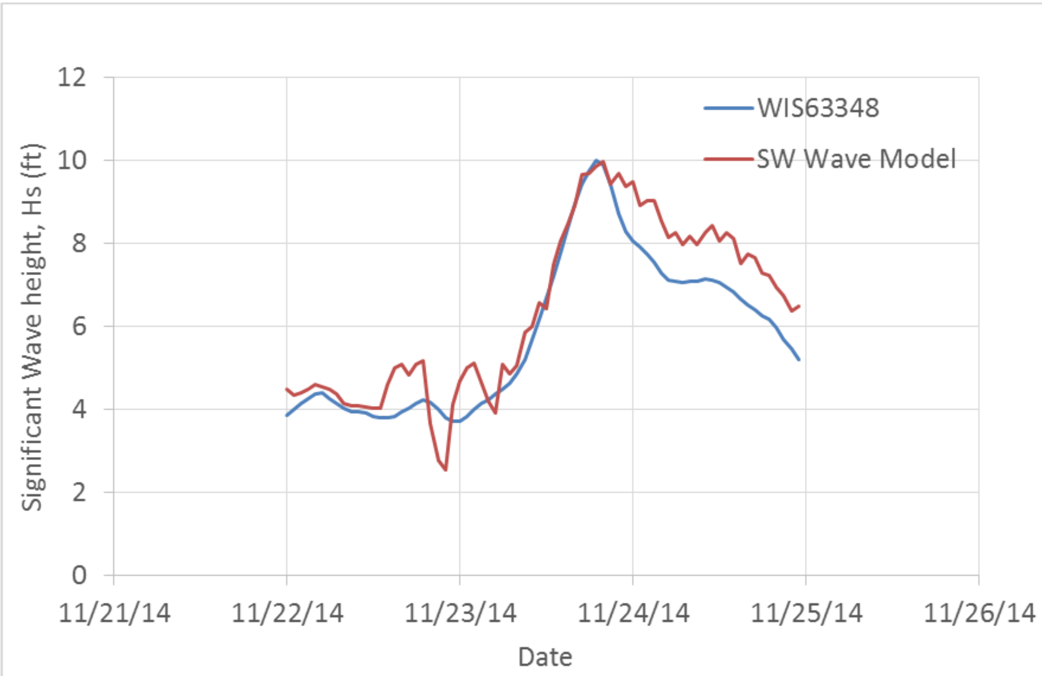

Figure 3-12: Calculated Wave Heights Compared to WIS63348 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

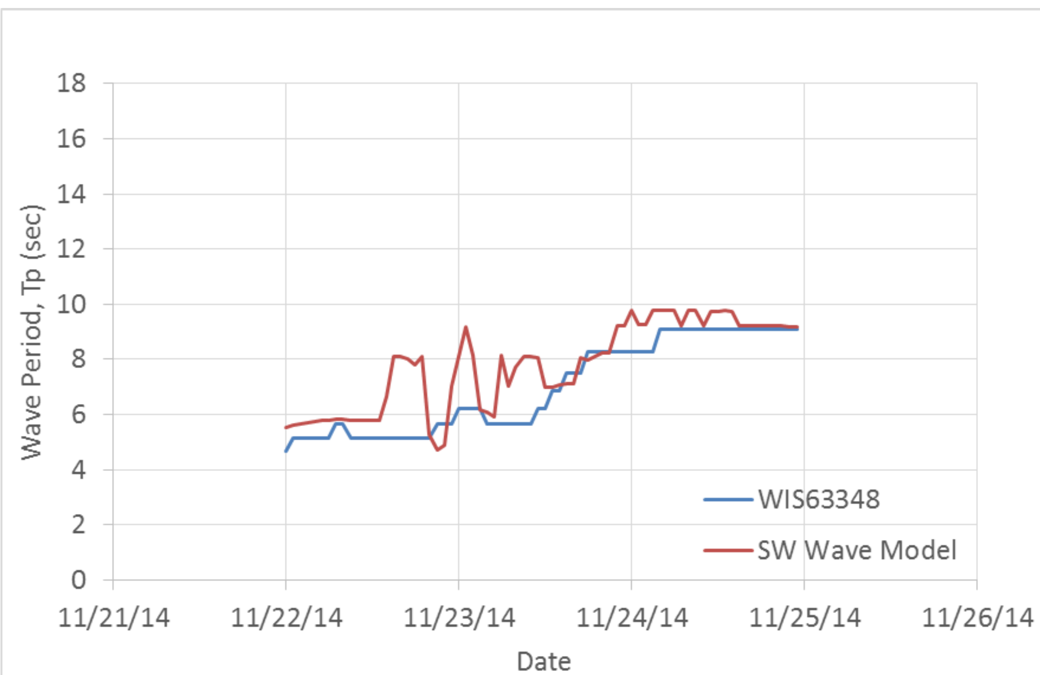

Figure 3-13: Calculated Wave Periods Compared to WIS63348 Wave Periods (Open Boundary Conditions: NDBC 41004 Buoy Data)

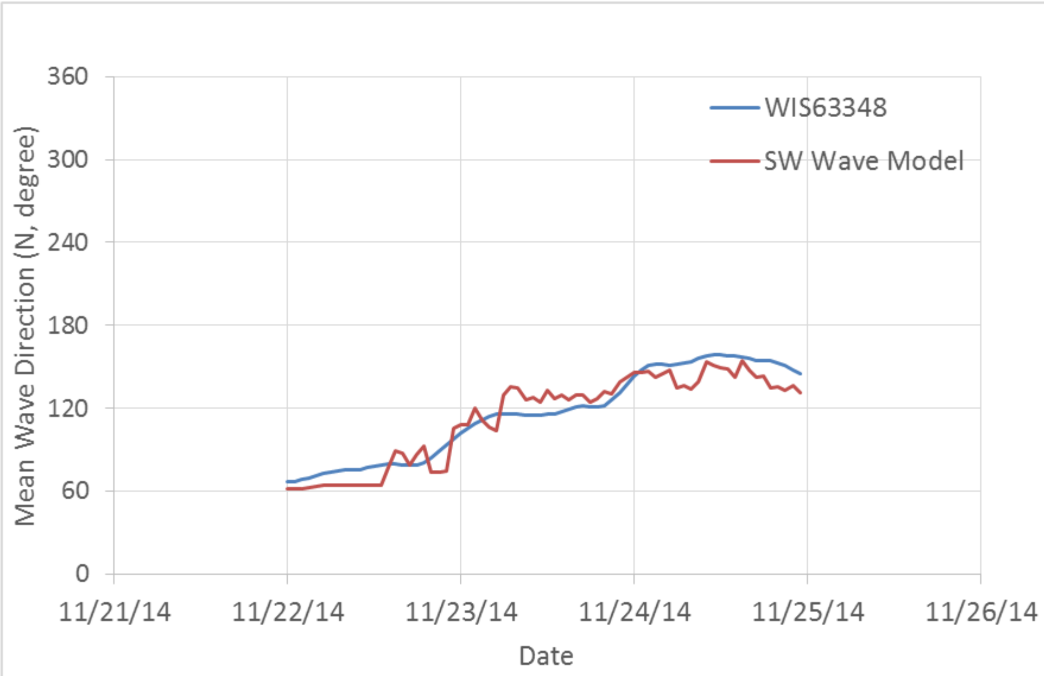

Figure 3-14: Calculated Wave Directions Compared to WIS63348 Wave Directions (Open Boundary Conditions: NDBC 41004 Buoy Data)

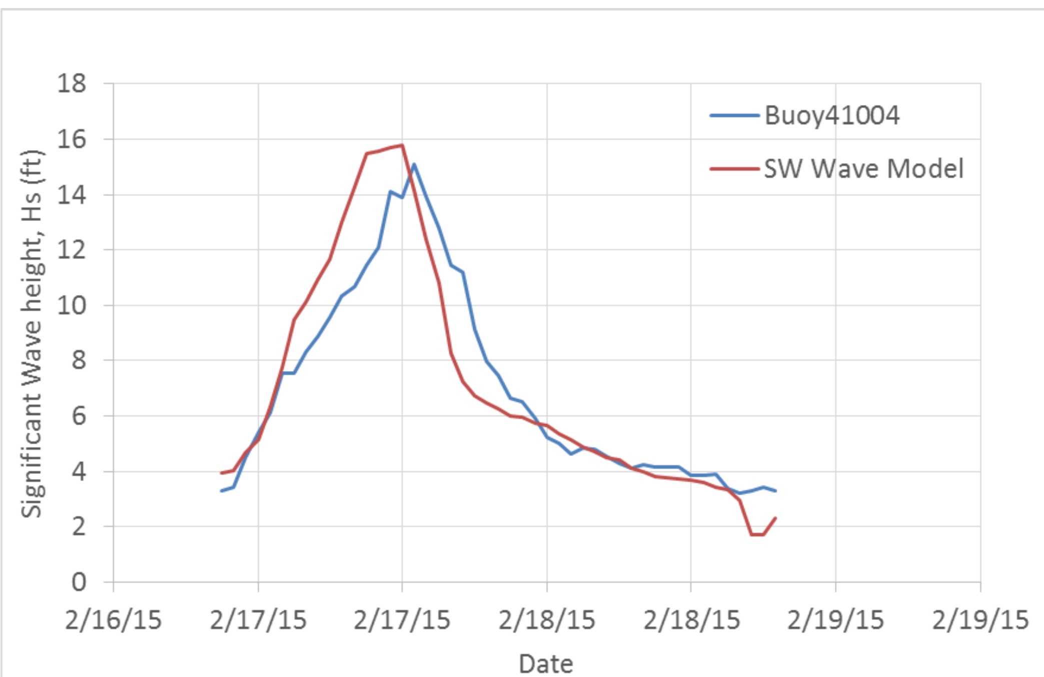

Figure 3-15: Calculated Wave Heights Compared to Buoy41004 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

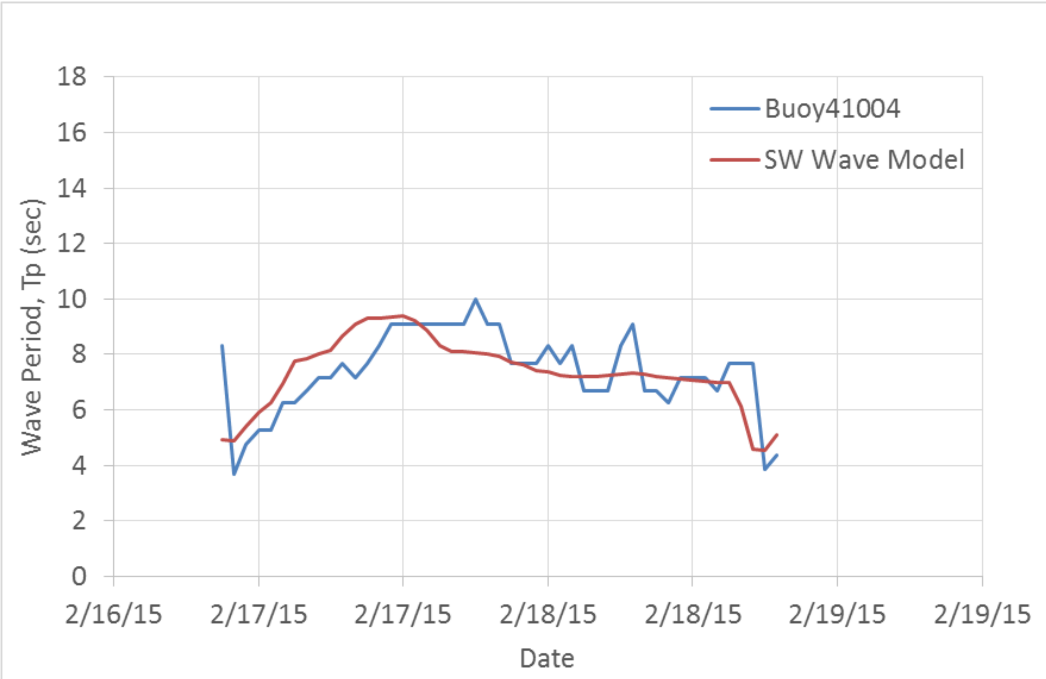

Figure 3-16: Calculated Wave Periods Compared to Buoy41004 Wave Heights (Open Boundary Conditions: NDBC 41004 Buoy Data)

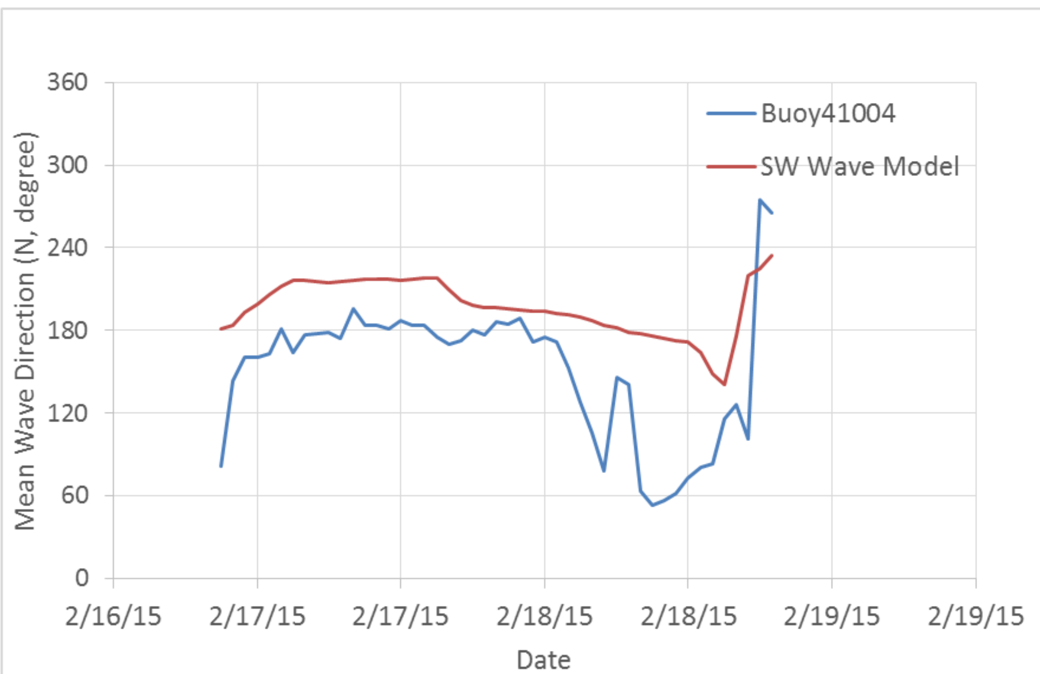

Figure 3-17: Calculated Wave Directions Compared to Buoy41004 Wave Directions (Open Boundary Conditions: NDBC 41004 Buoy Data)

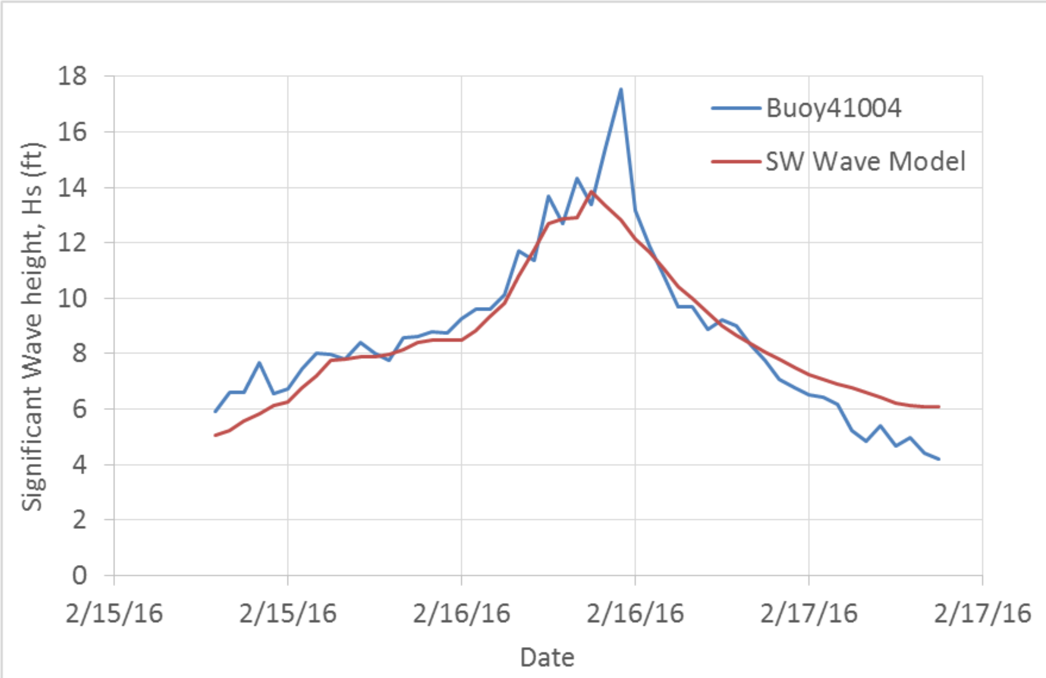

Figure 3-18: Calculated Wave Heights Compared to Buoy41004 Wave Heights (Open Boundary Conditions: WaveWatch 3 Data)

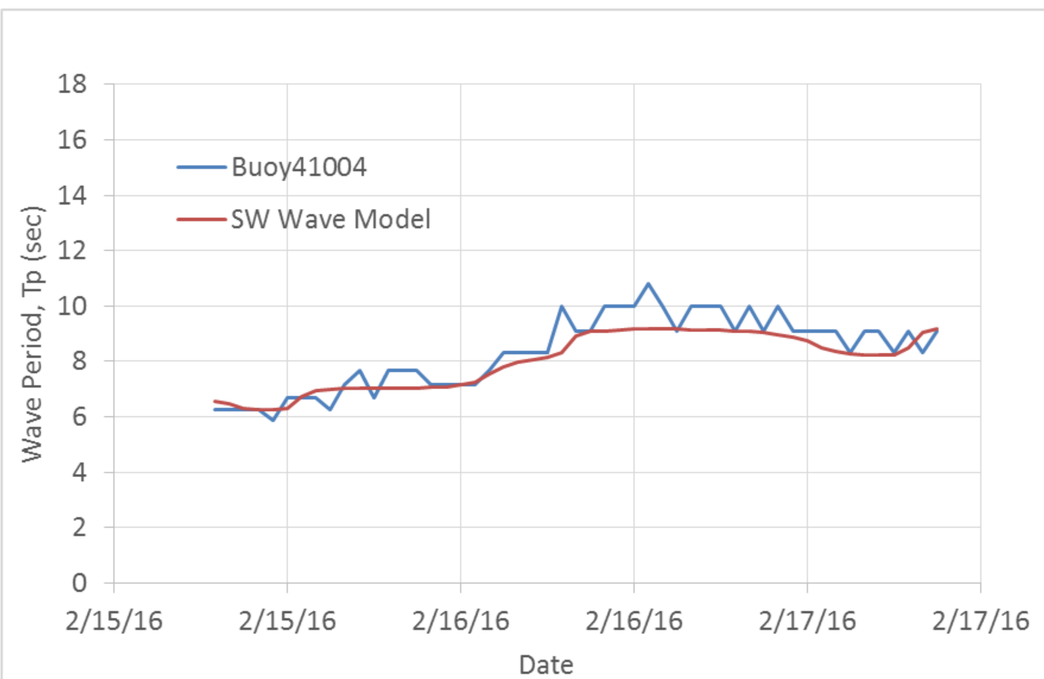

Figure 3-19: Calculated Wave Periods Compared to Buoy41004 Wave Periods (Open Boundary Conditions: WaveWatch 3 Data)

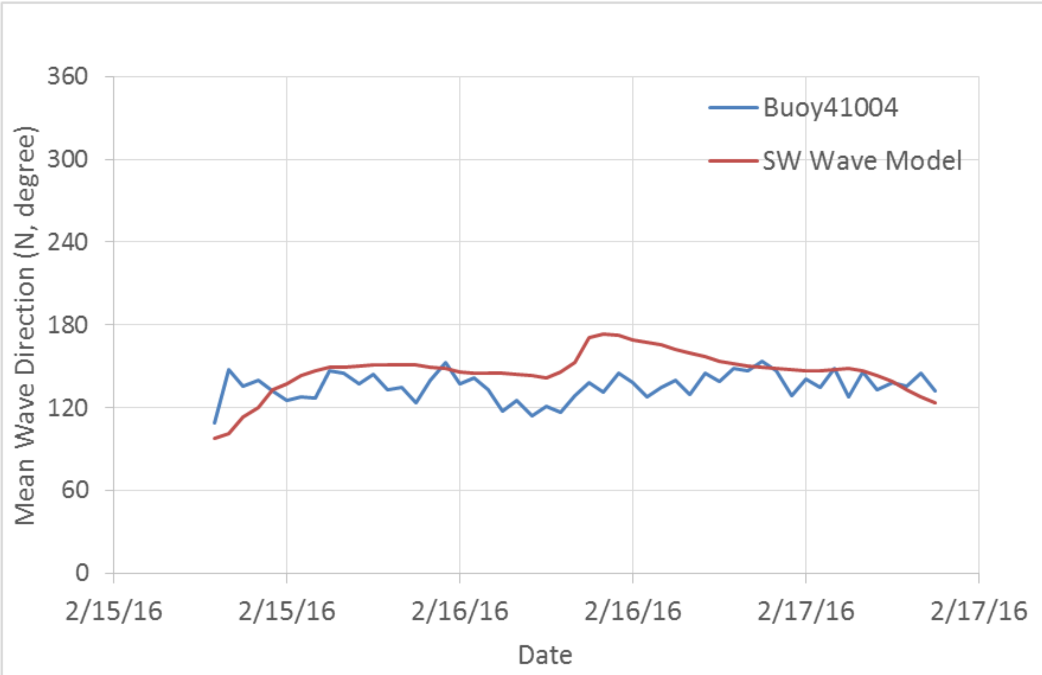

Figure 3-20: Calculated Wave Directions Compared to Buoy41004 Wave Directions (Open Boundary Conditions: WaveWatch 3 Data)

3.2. Wave Model Transformation

The calibrated and validated spectral wave model was applied for wave transformation from offshore to nearshore project site. Based on wave model calibration and validation, applying measured wave data as wave model open boundary conditions would get more accurate short-period peak wave heights at nearshore area compared to using model data, such as WaveWatch 3. For the long-term wave transformation simulations, NDBC measured wave data was used for the open boundary conditions in the time periods that it was available. When NDBC measured data was not available, the WaveWatch 3 data was used to provide wave model open boundary conditions. The wave transformation model was thus run for the following time periods:

- Offshore WaveWatch 3 wave data from January 2008 to December 2014
- NDBC 41004 wave data from April 2014 to December 2019

The model simulated nearshore wave conditions were extracted along the 4.0 meters (13.1 feet) mean sea level (MSL) depth contours at 22 locations along the Folly Beach. The nearshore wave data, which include both storm and non-storm wave conditions, were utilized as representative wave conditions to evaluate long-term shoreline changes.

4. One-Line Shoreline Model Calibration

The USACE GenCade shoreline evolution model was used to estimate the shoreline change rates for the Folly Beach shoreline study. The GenCade model is a one-line, one-dimensional shoreline change model developed by the USACE's Coastal Inlets Research Program (CIRP) to combine and improve upon the capabilities of previous shoreline response models Cascade and GENESIS. The GenCade shoreline model calculates shoreline changes based on differential wave-driven longshore sediment transport rates.

A GenCade shoreline model was developed for the Folly Beach as illustrated in Figure 4-1.The total shoreline length simulated within the shoreline model is approximately 6.3 miles. The shoreline at mean high water level (MHW) was represented by grid points with a spacing of 20.0 feet in the alongshore direction. The GenCade shoreline model was calibrated using the measured shorelines in December 2016 and December 2017.

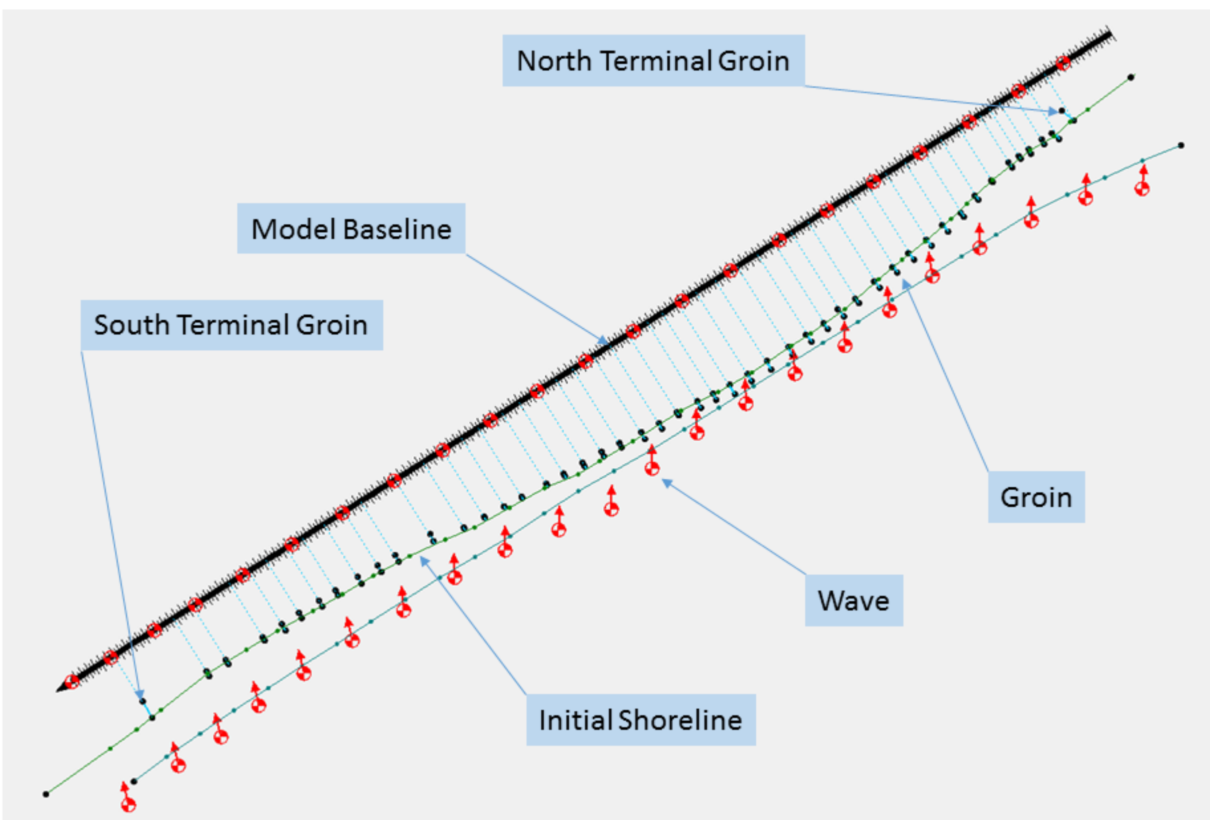

Figure 4-1: GenCade Model Setup

The GenCade model setup parameters are presented in Table 4-1.

Table 4-1: Calibrated GenCade Model Setup Parameters

No.	Parameter Name	Value
1	Cell size	20 ft
2	Grain size	0.17 mm
3	Average berm height	7.5 ft
4	Closure depth	10 ft
5	Longshore sand transport coefficient, K1	0.15
6	Longshore sand transport coefficient, K2	0.25
7	Lateral boundary for Northeast boundary	Moving
8	Lateral boundary for Southwest boundary	Pinned

The grain size was selected from the 1994 Coastal Engineering Journal article about Folly Beach (Billy et.al, 1994). The average berm height and the closure depth were determined from the historical survey profiles. The default longshore sand transport coefficient K1 is 0.5, and the default longshore sand transport coefficient K2 is 0.25. The groin permeability parameters were initially estimated considering the groin elevations provided by USACE, aerial images, and engineering judgement. Then the estimated groin permeability parameters were adjusted and calibrated using the measured shoreline locations. The final calibrated groin permeability parameters are presented in Table 4-2.

In the GenCade shoreline model, the sand bypassing is assumed to take place if the water depth at the tip of the structure is less than the depth of active longshore transport.

In the Coastal Engineering Manual, the "closure depth" is defined using the concept of "the seaward limit of effective profile fluctuation over long-term (seasonal or multi-year) time scales." The USACE GenCade Model Theory and User's Guide states that "The depth of closure, the seaward limit beyond which the profile does not exhibit significant change in depth, must be specified by the user. Empirically, the location of the closure depth is difficult to identify precisely, as small bathymetric change in deeper water is extremely difficult to measure. This situation usually results in a depth of closure located somewhere in a wide range of values, requiring judgement to be exercised to specify a single value." M&N applied the closure depth values of 10 feet and 15 feet to estimate shoreline change rates. Based on the shoreline model calibration and validation, using the closure depth of 10 feet resulted in better shoreline change results from the GenCade shoreline model.

Table 4-2: Calibrated GenCade Shoreline Model Groin Permeability Values

Groin	Permeability	Groin	Permeability
#3	0.3	#30	0.1
#4	0.5	#31	0.1
#5	0.8	#32	0.1
#6	0.8	#33	0.1
#7	0.8	#34	0.1
#8	0.8	#35	0.1
#9	0.8	#36	0.1
#10	0.5	#37	0.1
#11	0.5	#38	0.1
#12	0.5	#39	0.8
#13	0.5	#40	0.8
#14	0.5	#41	0.8
#15	0.5	#42	0.8
#16	0.5	#43	0.8
#17	0.5	#44	0.8
#18	0.8	#45	0.8
#19	0.5	#46	0.8
#20	0.5	#47	0.8
#21	0.5	#48	0.8
#22	0.5	#49	0.8
#23	0.5	#50	0.3
#24	0.5		
#25	0.5		
#26	0.5		
#27	0.6		
#28	0.6		
#29	0.6		

Figure 4-2 illustrates the comparisons between the observed (blue dots) annual shoreline changes at each survey transect and model simulated shoreline changes (solid red line) at the MHW water location between December 2016 and December 2017. The purple dots show the groin locations in the figures. In general, the simulated shoreline changes are in reasonable agreement with the observed shoreline changes at most locations along the Folly Beach.

The same shoreline model parameters presented in Table 4-1 and Table 4-2 were validated for two periods: a) March 2010 to December 2012, and b) December 2008 to March 2010.

Figure 4-3 shows the comparisons between the observed and model simulated annual shoreline changes at each survey transect between March 2010 and December 2012. For this period, the shoreline model simulated shoreline change rates are underestimated compared to the measured data. However, the model simulated trends of shoreline erosion and accretion are similar with the measured data.

Figure 4-4 shows the comparisons between the observed and model simulated annual shoreline changes at each survey transect between December 2008 and March 2010. In general, the simulated shoreline changes are in agreement with the observed shoreline changes at most locations along the Folly Beach shoreline between December 2008 and March 2010.

The calculated net longshore sediment transport rates during December 2016 and December 2017 are illustrated in Figure 4-5. In Figure 4-5 the positive sediment transport is from northeast to southwest, and the negative sediment transport is from southwest to northeast. The shoreline model simulated net longshore sediment transport rates are between -32,000 cy/yr (sediment transport to northeast) and 99,000 cy/yr (sediment transport to southwest) during December 2016 and December 2017.

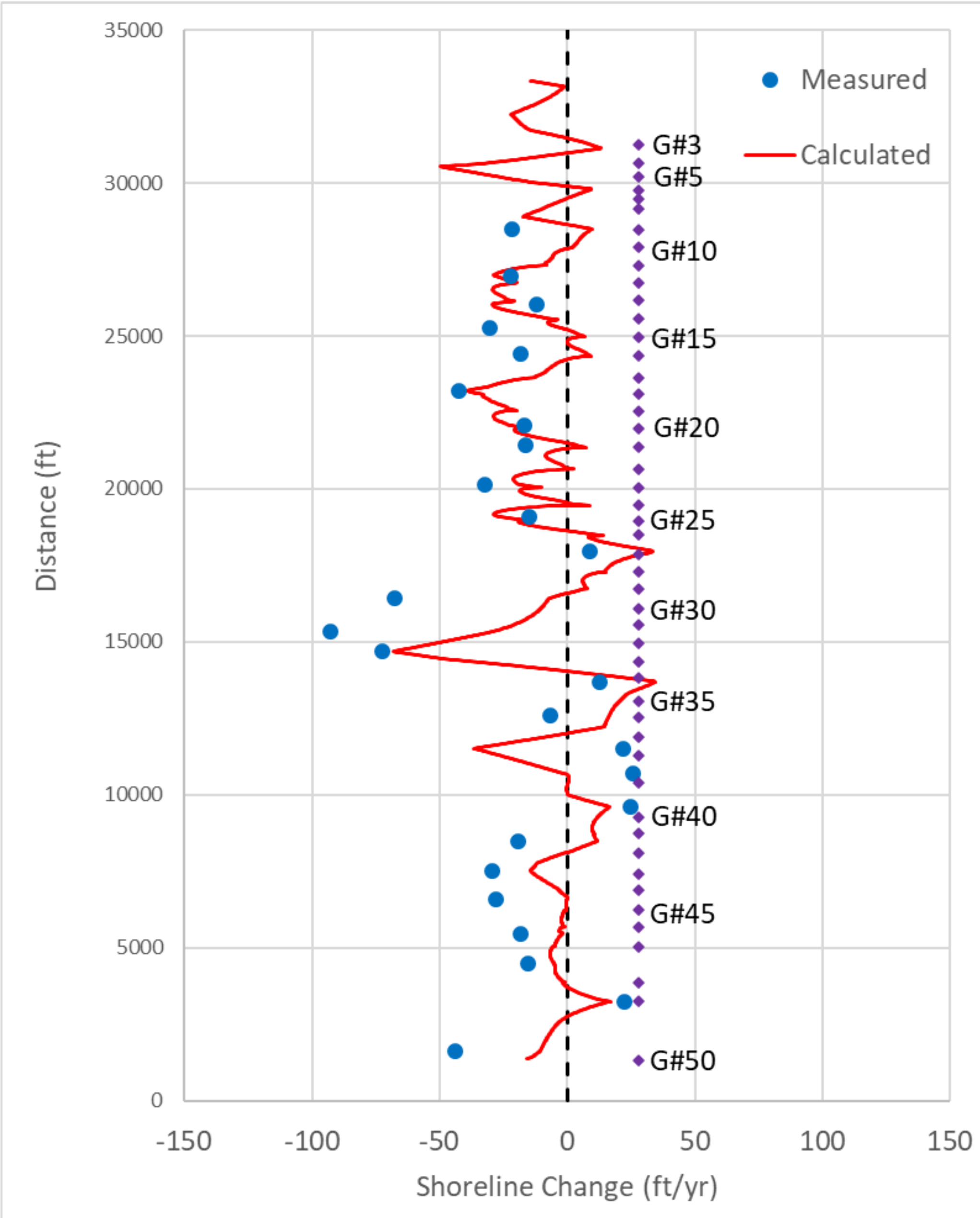

Figure 4-2: Shoreline Model Calibration (Dec. 2016 to Dec. 2017)

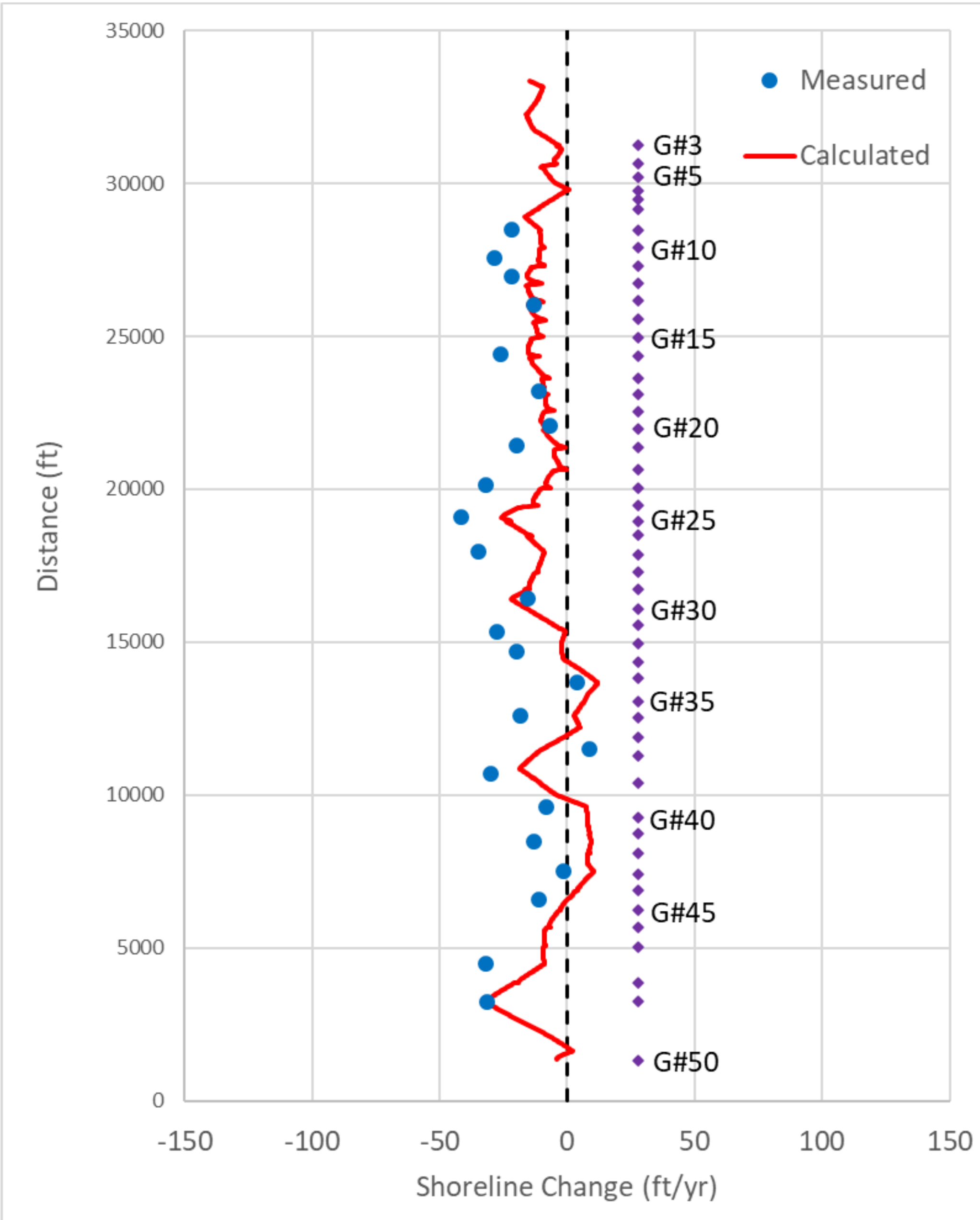

Figure 4-3: Shoreline Model Validation (Mar. 2010 to Dec. 2012)

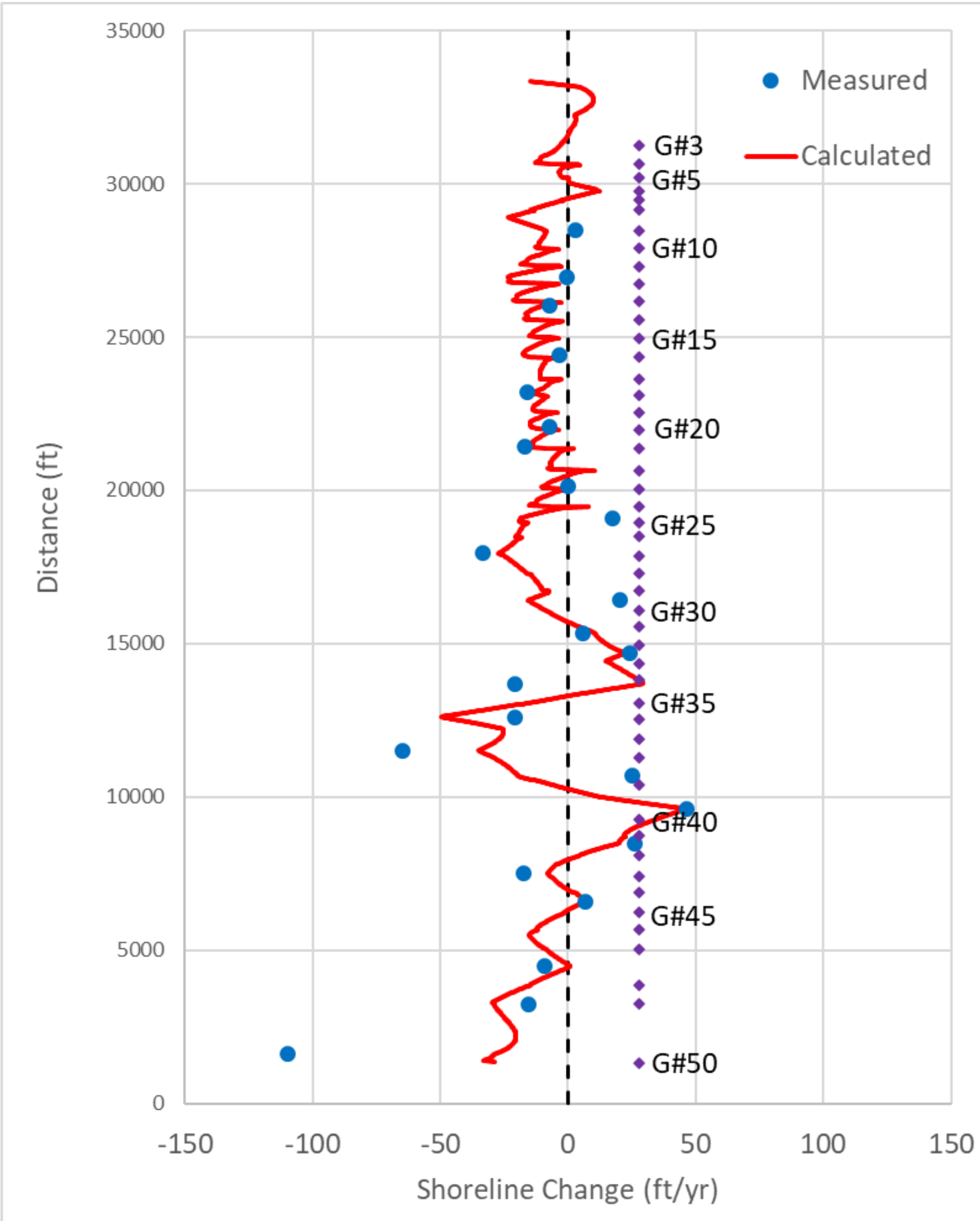

Figure 4-4: Shoreline Model Validation (Dec. 2008 to Mar. 2010)

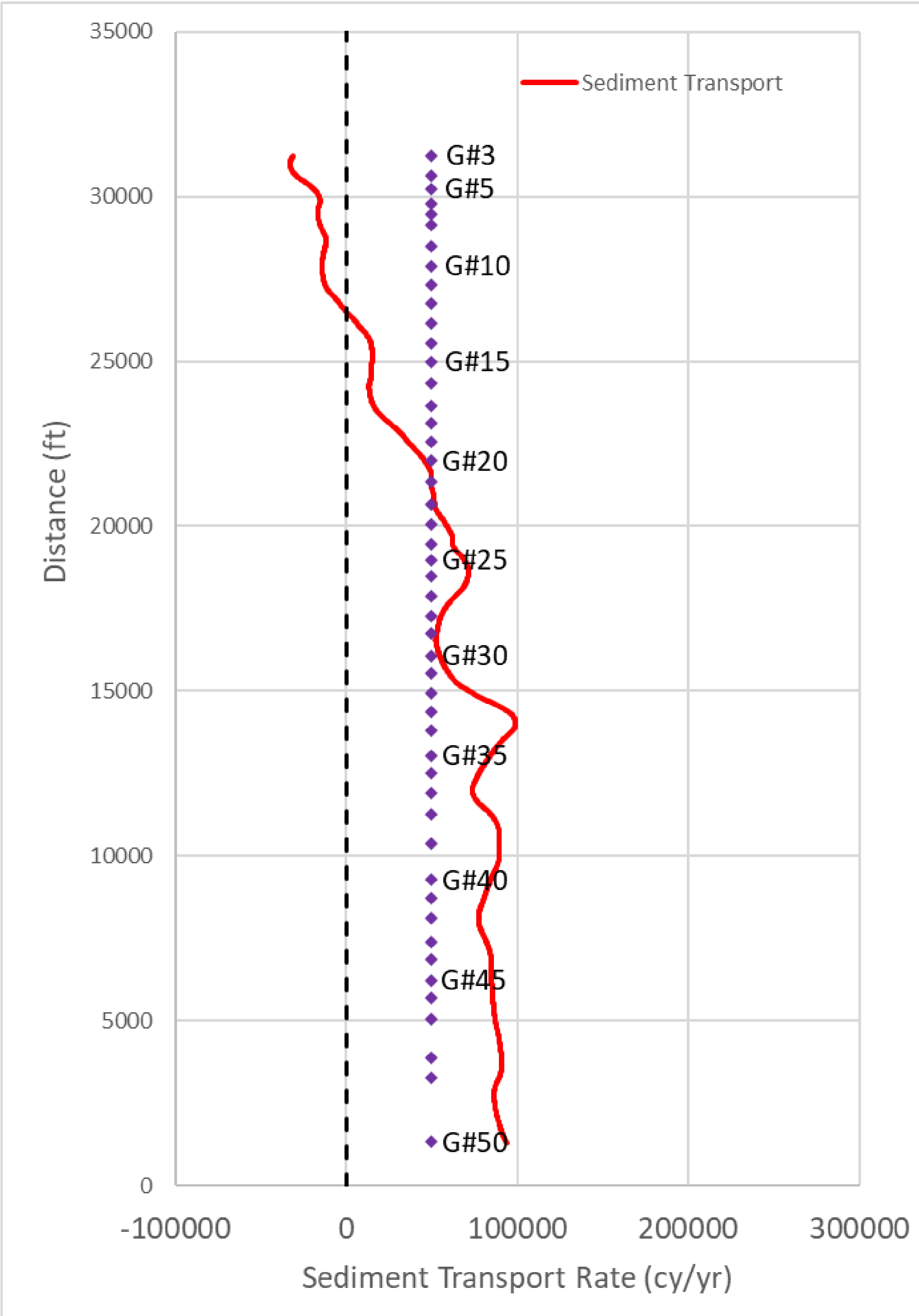

Figure 4-5: Calculated Net Longshore Sediment Transport Rates (Dec. 2016 to Dec. 2017)

5. Estimation of Shoreline Change Rates of Future With Project Scenarios

Four future beach nourishment projects were developed by USACE. The future beach nourishment template and the beach reaches for Beach-fx model are illustrated in Figure 5-1. Table 5-1 and Table 5-2 present the detailed beach nourishment design and future project schedule. The project interval of the future beach nourishment projects will be 12 years. The grain sizes from the proposed sand sources will be in the range of 0.16 mm and 0.20 mm.

The calibrated GenCade shoreline model was used to estimate the shoreline change rates for the Future With Project scenarios.

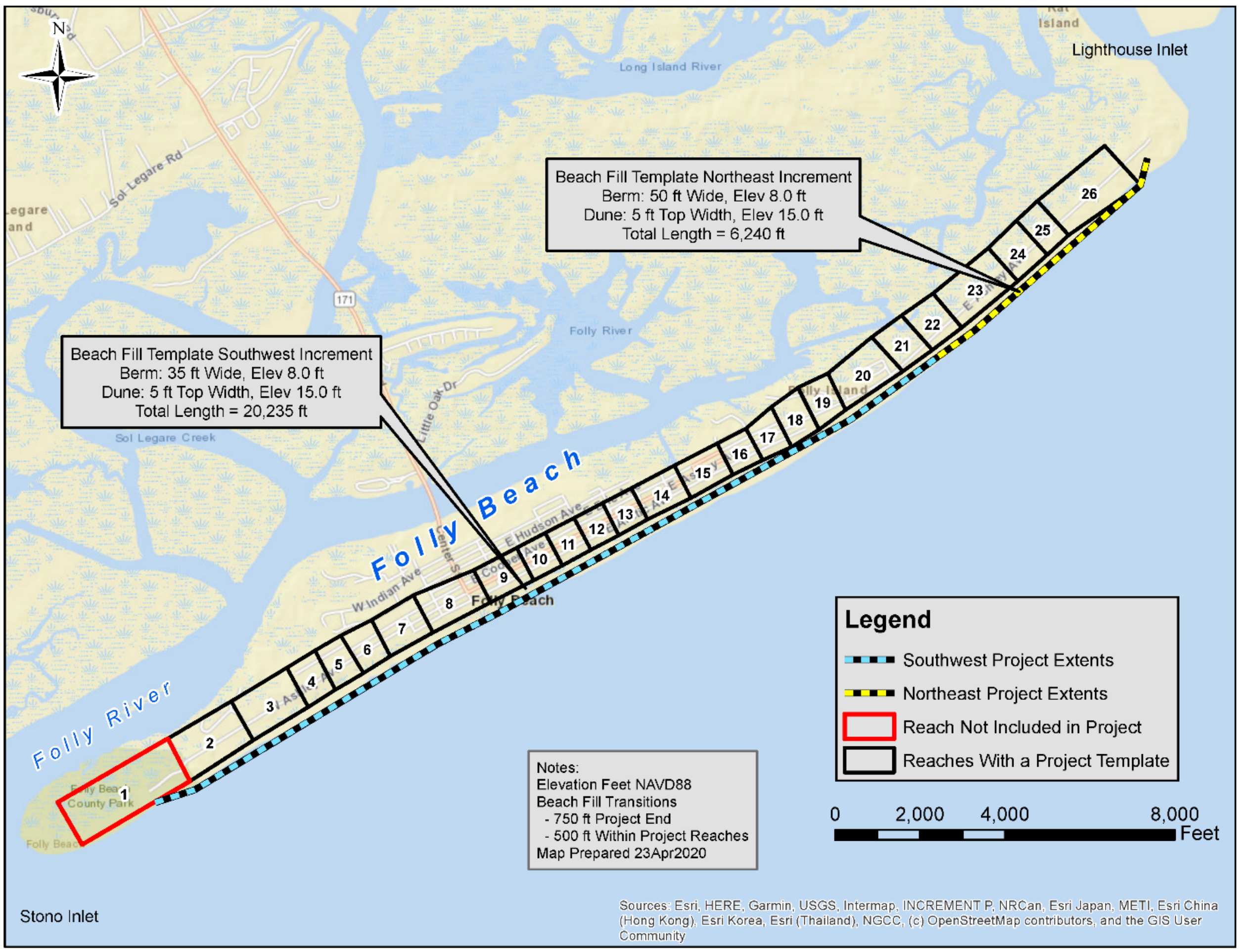

Figure 5-1: Beach Nourishment Template and Beach-fx Model Reaches

Table 5-1: Beach Nourishment Design

Design	Beach-fx Reach 2 to Beach-fx Reach 21	Beach-fx Reach 22 to Beach-fx Reach 26
Dune elevation	15.0 ft, NAVD	15.0 ft, NAVD
Top dune width	5.0 ft	5.0 ft
Dune slope	1v:5h	1v:5h
Berm elevation	8.0 ft, NAVD	8.0 ft, NAVD
Berm width	35.0 ft	50.0 ft
Foreshore slope to MHW	1v:15h	1v:15h
Offshore slope to existing profile	1v:30h	1v:30h

Table 5-2: Beach Nourishment Schedule

Project	Date	Grain Size (d_{50}, mm)
Nourishment #1	January 2024	0.20
Nourishment #2	January 2036	0.19
Nourishment #3	January 2048	0.19
Nourishment #4	January 2060	0.16

The 12-year time series (2008 - 2019) nearshore wave conditions were extracted from the spectral wave model results. The annual longshore sediment transport rates were estimated using a profile-based model of longshore sediment transport (LITDRIFT, developed by DHI). The estimated annual longshore sediment transport rates are between approximately 60,000 cy/yr and 158,000 cy/yr from 2008 to 2019. The estimated 2010 annual longshore sediment transport rate represents the lowest rate between 2008 and 2019. Thus, the time series wave conditions in 2010 were selected as representative waves to represent milder (non-storm) wave climates for the shoreline model since storms will be modeled in Beach-fx. The 2010 time series waves were repeated every year for 12-year period for the GenCade shoreline model.

The procedure of the shoreline change simulations is summarized as below:

- January 2024 beach nourishment project: A shoreline model was developed based on the December 2018 shoreline and the proposed January 2024 beach nourishment project. The shoreline changes from January 1, 2024 to December 31, 2035 were calculated.

- January 2036 beach nourishment project: A shoreline model was developed based on the simulated December 31, 2035 shoreline and the proposed January 2036 beach nourishment project. The shoreline changes from January 1, 2036 to December 31, 2047 were calculated.

- January 2048 beach nourishment project: A shoreline model was developed based on the simulated December 31, 2047 shoreline and the proposed January 2048 beach nourishment project. The shoreline changes from January 1, 2048 to December 31, 2059 were calculated.

- January 2060 beach nourishment project: A shoreline model was developed based on the simulated December 31, 2059 shoreline and the proposed January 2060 beach nourishment project. The shoreline changes from January 1, 2060 to December 31, 2071 were calculated.

The simulated shoreline change rates for the Future With Project scenarios were analyzed for both the Beach-fx model reaches and the SBEACH model reaches as illustrated in Figure 5-2. The estimated annual and average shoreline change rates for the Beach-fx model reaches are presented in Table 5-3 through Table 5-8. The estimated annual and average shoreline change rates for the SBEACH model reaches are presented in Table 5-9 through Table 5-14.

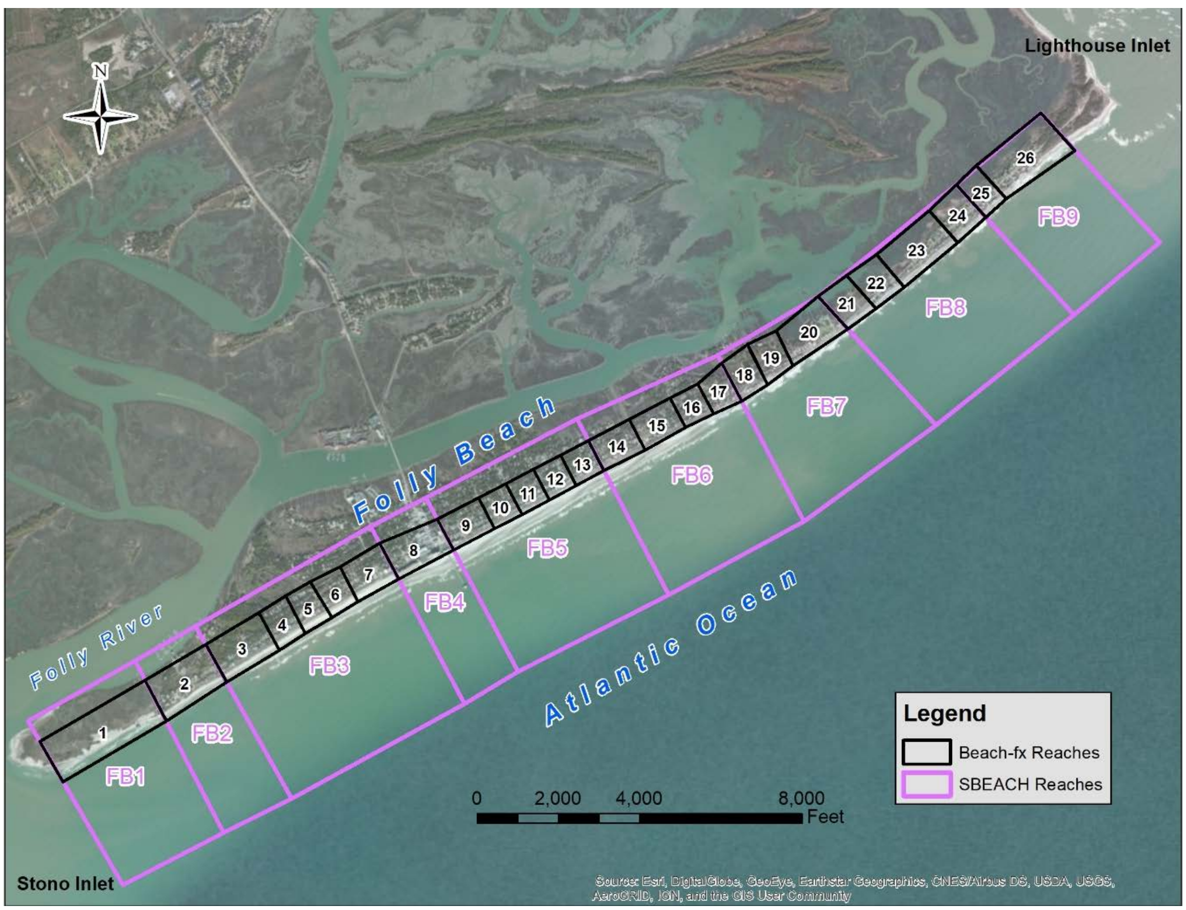

Figure 5-2: Beach-fx Model Reaches and SBEACH Model Reaches

Table 5-3: Annual Shoreline Change Rates for 2024 Beach Nourishment Project at Beach-fx Model Reaches

Beach -fx Reach	Shoreline Change Rates (ft/yr)											
	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035
R#2	-3.0	-6.3	-7.5	-6.7	-4.1	-1.3	0.9	2.2	2.9	3.1	3.1	3.0
R#3	-1.2	1.2	2.0	1.8	1.2	0.9	1.0	1.2	1.5	1.7	1.9	1.9
R#4	8.4	8.6	7.7	6.0	4.4	3.5	2.7	2.1	1.8	1.6	1.5	1.5
R#5	12.5	13.7	9.6	6.9	5.2	3.7	2.8	2.1	1.7	1.4	1.2	1.1
R#6	30.5	12.9	7.2	4.8	3.6	2.7	2.1	1.6	1.3	1.0	0.8	0.7
R#7	2.8	-1.3	-1.1	-0.5	0.1	0.3	0.3	0.3	0.2	0.1	0.0	-0.1
R#8	-33.8	-14.3	-7.6	-4.7	-3.3	-2.5	-2.0	-1.7	-1.5	-1.4	-1.3	-1.3
R#9	18.5	2.7	-1.5	-2.8	-3.2	-3.2	-3.1	-2.9	-2.7	-2.6	-2.5	-2.4
R#10	18.3	6.2	0.3	-2.0	-3.0	-3.3	-3.3	-3.3	-3.1	-3.0	-2.9	-2.9
R#11	-4.4	-3.1	-2.8	-3.2	-3.4	-3.5	-3.5	-3.4	-3.4	-3.3	-3.2	-3.1
R#12	-29.6	-10.6	-6.2	-4.4	-3.6	-3.3	-3.2	-3.1	-3.1	-3.2	-3.2	-3.3
R#13	-11.0	-10.0	-6.2	-4.3	-3.4	-3.0	-2.9	-3.0	-3.1	-3.2	-3.3	-3.4
R#14	0.3	-1.9	-2.2	-2.2	-2.2	-2.4	-2.6	-2.8	-3.0	-3.2	-3.3	-3.4
R#15	8.5	4.8	1.6	-0.4	-1.7	-2.6	-3.2	-3.6	-3.9	-4.1	-4.3	-4.2
R#16	6.9	2.1	-0.6	-2.4	-3.5	-4.3	-4.8	-5.1	-5.3	-5.4	-5.5	-5.8
R#17	-3.6	-5.0	-5.6	-5.9	-6.2	-6.3	-6.4	-6.4	-6.4	-6.4	-6.4	-6.8
R#18	-18.6	-13.5	-11.8	-10.7	-9.9	-9.3	-8.9	-8.6	-8.5	-8.3	-8.2	-7.9
R#19	-19.1	-18.0	-15.5	-13.7	-12.6	-11.8	-11.3	-10.9	-10.5	-10.3	-10.1	-9.8
R#20	-21.2	-17.2	-15.9	-14.9	-14.2	-13.6	-13.1	-12.7	-12.4	-12.0	-11.7	-11.5
R#21	-3.5	-12.0	-13.2	-13.5	-13.6	-13.5	-13.4	-13.2	-13.0	-12.7	-12.4	-12.1
R#22	-12.6	-11.4	-12.1	-12.5	-12.7	-12.8	-12.8	-12.7	-12.5	-12.2	-12.0	-11.8
R#23	-13.4	-11.9	-11.6	-11.7	-11.8	-11.9	-11.8	-11.6	-11.4	-11.2	-11.0	-10.8
R#24	-9.3	-10.2	-11.5	-11.7	-11.5	-11.2	-11.0	-10.7	-10.6	-10.4	-10.2	-10.2
R#25	-4.8	-11.5	-12.4	-11.8	-11.1	-10.5	-10.1	-9.8	-9.7	-9.7	-9.8	-10.0
R#26	-21.7	-13.0	-9.3	-7.6	-6.9	-6.7	-6.6	-6.9	-7.2	-7.6	-8.1	-8.5

Table 5-4: Annual Shoreline Change Rates for 2036 Beach Nourishment Project at Beach-fx Model Reaches

Beach -fx Reach	Shoreline Change Rates (ft/yr)											
	2036	**2037**	**2038**	**2039**	**2040**	**2041**	**2042**	**2043**	**2044**	**2045**	**2046**	**2047**
R#2	4.2	-1.1	-4.3	-5.3	-4.3	-1.8	0.5	1.9	2.7	2.9	2.9	2.8
R#3	2.1	2.3	1.8	0.7	-0.1	-0.6	-0.6	-0.3	0.1	0.5	0.8	1.0
R#4	2.2	1.5	1.4	1.4	0.9	0.5	0.2	0.1	0.0	0.1	0.2	0.4
R#5	0.1	0.9	0.9	0.8	0.8	0.6	0.4	0.3	0.2	0.1	0.2	0.2
R#6	1.1	0.4	0.3	0.4	0.5	0.6	0.5	0.4	0.3	0.2	0.1	0.1
R#7	-0.4	-0.7	-0.3	0.1	0.3	0.4	0.4	0.3	0.1	0.0	-0.1	-0.2
R#8	-2.9	-0.1	0.5	0.5	0.3	0.1	-0.1	-0.3	-0.4	-0.5	-0.6	-0.7
R#9	7.7	3.6	1.4	0.1	-0.6	-1.0	-1.3	-1.4	-1.4	-1.4	-1.5	-1.5
R#10	5.1	1.5	-1.1	-2.0	-2.3	-2.4	-2.3	-2.3	-2.2	-2.1	-2.2	-2.2
R#11	2.9	-5.5	-5.0	-4.5	-3.9	-3.5	-3.1	-2.9	-2.7	-2.6	-2.5	-2.5
R#12	-30.7	-11.5	-7.5	-5.5	-4.4	-3.8	-3.4	-3.2	-3.2	-3.1	-3.0	-3.0
R#13	-13.1	-10.5	-6.9	-5.1	-4.1	-3.6	-3.3	-3.2	-3.2	-3.3	-3.3	-3.3
R#14	0.1	-2.3	-2.6	-2.6	-2.6	-2.8	-2.9	-3.0	-3.2	-3.3	-3.4	-3.5
R#15	8.5	4.8	1.6	-0.6	-1.9	-2.8	-3.4	-3.8	-4.1	-4.2	-4.4	-4.3
R#16	7.3	2.3	-0.5	-2.3	-3.6	-4.3	-4.9	-5.2	-5.4	-5.5	-5.6	-5.9
R#17	-3.3	-4.9	-5.5	-5.9	-6.1	-6.3	-6.4	-6.5	-6.5	-6.5	-6.5	-6.8
R#18	-18.9	-13.6	-11.7	-10.6	-9.9	-9.3	-8.9	-8.6	-8.5	-8.3	-8.2	-8.0
R#19	-19.4	-18.2	-15.6	-13.8	-12.6	-11.9	-11.3	-10.9	-10.5	-10.3	-10.1	-9.9
R#20	-21.3	-17.4	-16.0	-15.0	-14.3	-13.7	-13.2	-12.8	-12.4	-12.1	-11.8	-11.5
R#21	-3.5	-12.0	-13.3	-13.6	-13.7	-13.6	-13.5	-13.3	-13.0	-12.8	-12.5	-12.2
R#22	-12.7	-11.5	-12.2	-12.6	-12.8	-12.9	-12.9	-12.8	-12.6	-12.3	-12.0	-11.9
R#23	-13.4	-11.9	-11.6	-11.8	-11.9	-11.9	-11.8	-11.6	-11.4	-11.3	-11.1	-10.9
R#24	-9.4	-10.3	-11.5	-11.7	-11.5	-11.3	-11.0	-10.8	-10.6	-10.4	-10.3	-10.3
R#25	-5.0	-11.6	-12.4	-11.9	-11.2	-10.5	-10.1	-9.9	-9.8	-9.8	-9.9	-10.1
R#26	-21.5	-13.0	-9.2	-7.6	-6.9	-6.7	-6.6	-6.9	-7.3	-7.7	-8.2	-8.6

Table 5-5: Annual Shoreline Change Rates for 2048 Beach Nourishment Project at Beach-fx Model Reaches

Beach -fx Reach	Shoreline Change Rates (ft/yr)											
	2048	**2049**	**2050**	**2051**	**2052**	**2053**	**2054**	**2055**	**2056**	**2057**	**2058**	**2059**
R#2	3.8	-1.3	-4.3	-5.1	-3.9	-1.3	0.8	2.1	2.8	2.9	2.9	2.7
R#3	1.3	1.6	1.1	0.2	-0.5	-0.9	-0.8	-0.5	0.0	0.4	0.7	0.9
R#4	1.2	0.8	0.8	0.6	0.4	0.1	-0.1	-0.2	-0.2	0.0	0.1	0.3
R#5	-0.6	0.3	0.3	0.3	0.3	0.3	0.1	0.1	0.0	0.0	0.1	0.1
R#6	0.5	-0.1	-0.1	0.1	0.3	0.3	0.3	0.2	0.2	0.1	0.0	0.0
R#7	-0.5	-0.8	-0.3	0.2	0.4	0.5	0.4	0.3	0.2	0.1	0.0	-0.1
R#8	-2.3	0.7	1.2	1.1	0.8	0.4	0.2	-0.1	-0.2	-0.4	-0.5	-0.6
R#9	10.1	5.0	2.3	0.8	-0.1	-0.6	-0.9	-1.1	-1.2	-1.3	-1.3	-1.4
R#10	6.4	2.0	-0.7	-1.7	-2.1	-2.1	-2.1	-2.0	-2.0	-2.0	-2.1	-2.1
R#11	0.4	-6.0	-5.2	-4.5	-3.8	-3.3	-3.0	-2.7	-2.6	-2.5	-2.4	-2.4
R#12	-31.7	-12.3	-7.9	-5.8	-4.6	-3.9	-3.5	-3.3	-3.2	-3.0	-3.0	-2.9
R#13	-13.4	-11.0	-7.3	-5.4	-4.4	-3.8	-3.5	-3.3	-3.3	-3.3	-3.3	-3.3
R#14	0.1	-2.4	-2.8	-2.8	-2.8	-2.9	-3.0	-3.1	-3.3	-3.4	-3.4	-3.5
R#15	8.5	4.8	1.5	-0.6	-2.0	-2.9	-3.5	-3.9	-4.1	-4.3	-4.4	-4.4
R#16	7.3	2.3	-0.5	-2.4	-3.6	-4.4	-4.9	-5.2	-5.4	-5.6	-5.7	-5.9
R#17	-3.3	-4.9	-5.5	-5.9	-6.2	-6.3	-6.4	-6.5	-6.5	-6.5	-6.5	-6.9
R#18	-18.9	-13.6	-11.7	-10.6	-9.9	-9.3	-8.9	-8.7	-8.5	-8.4	-8.3	-8.0
R#19	-19.4	-18.2	-15.6	-13.8	-12.6	-11.9	-11.3	-10.9	-10.5	-10.3	-10.2	-9.9
R#20	-21.3	-17.4	-16.0	-15.0	-14.3	-13.7	-13.2	-12.8	-12.4	-12.1	-11.8	-11.5
R#21	-3.5	-12.0	-13.3	-13.6	-13.7	-13.6	-13.5	-13.3	-13.1	-12.8	-12.5	-12.2
R#22	-12.7	-11.5	-12.2	-12.6	-12.8	-12.9	-12.9	-12.8	-12.6	-12.3	-12.0	-11.9
R#23	-13.4	-11.9	-11.6	-11.8	-11.9	-11.9	-11.8	-11.6	-11.4	-11.3	-11.1	-10.9
R#24	-9.4	-10.3	-11.5	-11.7	-11.5	-11.3	-11.0	-10.8	-10.6	-10.4	-10.3	-10.3
R#25	-5.0	-11.6	-12.4	-11.9	-11.2	-10.5	-10.1	-9.9	-9.8	-9.8	-9.9	-10.1
R#26	-21.5	-13.0	-9.2	-7.6	-6.9	-6.7	-6.6	-6.9	-7.3	-7.7	-8.2	-8.6

Table 5-6: Annual Shoreline Change Rates for 2060 Beach Nourishment Project at Beach-fx Model Reaches

Beach -fx Reach	Shoreline Change Rates (ft/yr)											
	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070	2071
R#2	4.0	-1.4	-4.8	-6.3	-6.8	-6.2	-3.7	-0.5	1.4	2.2	2.1	1.6
R#3	1.2	1.5	0.9	0.0	-0.9	-1.6	-2.0	-2.0	-1.4	-0.7	-0.3	0.0
R#4	0.8	0.5	0.6	0.5	0.2	-0.2	-0.5	-0.7	-0.9	-1.0	-0.7	-0.5
R#5	-0.9	0.0	0.2	0.2	0.2	0.1	0.0	-0.2	-0.5	-0.6	-0.6	-0.5
R#6	0.3	-0.3	-0.2	0.0	0.2	0.3	0.2	0.1	-0.1	-0.3	-0.4	-0.4
R#7	-0.4	-0.8	-0.3	0.2	0.4	0.4	0.4	0.3	0.1	-0.1	-0.2	-0.4
R#8	-2.1	0.9	1.4	1.2	0.8	0.5	0.2	-0.1	-0.2	-0.4	-0.5	-0.7
R#9	10.6	5.2	2.3	0.7	-0.1	-0.7	-0.9	-1.0	-1.2	-1.2	-1.3	-1.4
R#10	6.2	1.7	-0.9	-1.8	-2.1	-2.2	-2.0	-2.0	-2.0	-2.0	-2.0	-2.0
R#11	-0.8	-6.5	-5.5	-4.6	-3.9	-3.3	-2.9	-2.7	-2.5	-2.3	-2.3	-2.2
R#12	-32.2	-12.5	-8.0	-5.7	-4.5	-3.8	-3.5	-3.2	-2.9	-2.8	-2.7	-2.7
R#13	-13.3	-10.8	-7.1	-5.1	-4.1	-3.5	-3.2	-3.1	-3.0	-2.9	-2.9	-3.0
R#14	1.2	-2.0	-2.4	-2.4	-2.5	-2.6	-2.7	-2.8	-2.9	-3.0	-3.1	-3.2
R#15	8.8	5.5	2.1	-0.1	-1.6	-2.5	-3.1	-3.5	-3.8	-4.0	-4.1	-4.2
R#16	8.5	3.0	0.1	-1.9	-3.1	-3.9	-4.5	-4.8	-5.0	-5.2	-5.2	-5.3
R#17	-2.1	-4.5	-5.1	-5.5	-5.8	-5.9	-6.1	-6.1	-6.2	-6.2	-6.2	-6.5
R#18	-20.0	-13.7	-11.7	-10.3	-9.6	-9.0	-8.6	-8.4	-8.2	-8.1	-8.0	-7.7
R#19	-20.6	-18.8	-16.0	-13.9	-12.6	-11.8	-11.3	-10.8	-10.5	-10.2	-10.0	-9.8
R#20	-21.6	-17.8	-16.4	-15.4	-14.6	-14.0	-13.4	-13.0	-12.6	-12.2	-11.8	-11.5
R#21	-3.2	-12.2	-13.6	-14.0	-14.1	-14.0	-13.8	-13.6	-13.3	-13.0	-12.6	-12.3
R#22	-12.9	-11.6	-12.3	-12.8	-13.1	-13.2	-13.2	-13.0	-12.7	-12.4	-12.1	-12.0
R#23	-13.5	-12.0	-11.7	-11.9	-12.0	-12.0	-11.9	-11.7	-11.5	-11.4	-11.2	-11.0
R#24	-9.4	-10.4	-11.6	-11.8	-11.7	-11.4	-11.2	-11.0	-10.9	-10.7	-10.6	-10.6
R#25	-5.6	-11.7	-12.5	-12.0	-11.5	-10.8	-10.4	-10.3	-10.2	-10.2	-10.4	-10.7
R#26	-21.0	-12.7	-9.1	-7.5	-6.9	-6.7	-6.8	-7.1	-7.5	-8.0	-8.5	-9.0

Table 5-7: Total 12 Years Average Shoreline Change Rates for the Future with Project Scenarios at Beach-fx Model Reaches

Beach-fx Reach	Average Shoreline Change Rates (ft/yr)			
	Jan2024 Fill	Jan2036 Fill	Jan2048 Fill	Jan2060 Fill
R#2	-1.1	0.1	0.2	-1.5
R#3	1.3	0.6	0.3	-0.4
R#4	4.1	0.7	0.3	-0.2
R#5	5.2	0.5	0.1	-0.2
R#6	5.8	0.4	0.2	-0.1
R#7	0.1	0.0	0.0	0.0
R#8	-6.3	-0.4	0.0	0.1
R#9	-0.5	0.2	0.9	0.9
R#10	-0.2	-1.2	-0.9	-0.9
R#11	-3.3	-3.0	-3.2	-3.3
R#12	-6.4	-6.9	-7.1	-7.0
R#13	-4.7	-5.3	-5.4	-5.2
R#14	-2.4	-2.7	-2.8	-2.4
R#15	-1.1	-1.2	-1.3	-0.9
R#16	-2.8	-2.8	-2.8	-2.3
R#17	-6.0	-5.9	-6.0	-5.5
R#18	-10.3	-10.4	-10.4	-10.3
R#19	-12.8	-12.9	-12.9	-13.0
R#20	-14.2	-14.3	-14.3	-14.5
R#21	-12.2	-12.2	-12.2	-12.5
R#22	-12.4	-12.4	-12.4	-12.6
R#23	-11.7	-11.7	-11.7	-11.8
R#24	-10.7	-10.8	-10.8	-10.9
R#25	-10.1	-10.2	-10.2	-10.5
R#26	-9.2	-9.2	-9.2	-9.2

Table 5-8: Last 8 Years Average Shoreline Change Rates for Future with Project Scenarios at Beach-fx Model Reaches

Beach-fx Reach	Average Shoreline Change Rates (ft/yr)			
	Jan2024 Fill	Jan2036 Fill	Jan2048 Fill	Jan2060 Fill
R#2	1.2	1.0	1.1	-1.2
R#3	1.4	0.1	-0.1	-1.1
R#4	2.4	0.3	0.0	-0.5
R#5	2.4	0.3	0.1	-0.3
R#6	1.7	0.3	0.2	0.0
R#7	0.1	0.2	0.2	0.1
R#8	-1.9	-0.3	0.0	-0.1
R#9	-2.8	-1.3	-1.0	-1.0
R#10	-3.1	-2.2	-2.1	-2.0
R#11	-3.3	-3.0	-2.8	-2.8
R#12	-3.3	-3.4	-3.4	-3.3
R#13	-3.2	-3.4	-3.5	-3.2
R#14	-2.8	-3.1	-3.2	-2.8
R#15	-3.4	-3.6	-3.7	-3.3
R#16	-5.0	-5.0	-5.1	-4.6
R#17	-6.4	-6.5	-6.5	-6.1
R#18	-8.7	-8.7	-8.7	-8.5
R#19	-10.9	-10.9	-11.0	-10.9
R#20	-12.7	-12.7	-12.7	-12.9
R#21	-13.0	-13.1	-13.1	-13.3
R#22	-12.4	-12.5	-12.5	-12.7
R#23	-11.4	-11.5	-11.5	-11.6
R#24	-10.7	-10.8	-10.8	-11.0
R#25	-10.1	-10.2	-10.2	-10.6
R#26	-7.3	-7.3	-7.3	-7.6

Table 5-9: Annual Shoreline Change Rates for 2024 Beach Nourishment Project at SBEACH Model Reaches

SBEACH Reach	Shoreline Change Rates (ft/yr)											
	2024	**2025**	**2026**	**2027**	**2028**	**2029**	**2030**	**2031**	**2032**	**2033**	**2034**	**2035**
FB#1	-32.8	-28.1	-21.3	-12.2	-3.0	2.4	4.7	5.0	4.6	4.0	3.3	2.8
FB#2	-3.1	-6.0	-7.3	-6.6	-4.1	-1.3	0.8	2.1	2.8	3.1	3.1	3.0
FB#3	8.0	5.4	4.1	3.1	2.4	1.8	1.5	1.3	1.2	1.1	1.1	1.1
FB#4	-33.6	-14.1	-7.5	-4.7	-3.3	-2.5	-2.0	-1.7	-1.6	-1.4	-1.4	-1.3
FB#5	0.3	-2.5	-3.1	-3.3	-3.3	-3.3	-3.2	-3.1	-3.1	-3.0	-3.0	-3.0
FB#6	3.3	0.3	-1.4	-2.5	-3.2	-3.7	-4.0	-4.3	-4.5	-4.6	-4.7	-4.8
FB#7	-20.1	-16.6	-14.8	-13.7	-12.8	-12.2	-11.7	-11.3	-11.0	-10.8	-10.5	-10.2
FB#8	-10.4	-11.4	-12.0	-12.2	-12.3	-12.2	-12.1	-11.9	-11.7	-11.5	-11.3	-11.1
FB#9	-17.6	-12.5	-9.9	-8.5	-7.8	-7.5	-7.4	-7.5	-7.7	-8.1	-8.4	-8.8

Table 5-10: Annual Shoreline Change Rates for 2036 Beach Nourishment Project at SBEACH Model Reaches

SBEACH Reach	Shoreline Change Rates (ft/yr)											
	2036	**2037**	**2038**	**2039**	**2040**	**2041**	**2042**	**2043**	**2044**	**2045**	**2046**	**2047**
FB#1	-34.8	-30.0	-23.4	-15.9	-5.9	1.7	5.1	5.8	5.5	4.7	3.9	3.1
FB#2	4.1	-0.8	-4.0	-5.1	-4.2	-1.8	0.4	1.8	2.6	2.9	2.9	2.7
FB#3	1.0	1.0	0.8	0.6	0.4	0.2	0.1	0.1	0.1	0.2	0.3	0.3
FB#4	-2.8	0.0	0.6	0.5	0.3	0.0	-0.2	-0.3	-0.5	-0.6	-0.7	-0.7
FB#5	-4.3	-3.8	-3.4	-3.1	-2.9	-2.7	-2.6	-2.5	-2.5	-2.4	-2.4	-2.4
FB#6	3.4	0.2	-1.5	-2.6	-3.3	-3.8	-4.2	-4.4	-4.6	-4.7	-4.8	-4.9
FB#7	-20.3	-16.7	-14.9	-13.7	-12.9	-12.2	-11.8	-11.4	-11.1	-10.8	-10.6	-10.3
FB#8	-10.5	-11.5	-12.0	-12.3	-12.3	-12.3	-12.2	-12.0	-11.8	-11.6	-11.4	-11.2
FB#9	-17.5	-12.5	-9.8	-8.5	-7.8	-7.5	-7.4	-7.6	-7.8	-8.2	-8.5	-8.9

Table 5-11: Annual Shoreline Change Rates for 2048 Beach Nourishment Project at SBEACH Model Reaches

SBEACH Reach	Shoreline Change Rates (ft/yr)											
	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059
FB#1	-34.0	-29.1	-22.4	-14.5	-4.4	2.7	5.5	6.0	5.5	4.6	3.8	2.9
FB#2	3.7	-1.0	-4.0	-4.9	-3.8	-1.4	0.7	2.0	2.6	2.9	2.8	2.6
FB#3	0.4	0.4	0.4	0.3	0.1	-0.1	-0.1	-0.1	0.0	0.1	0.2	0.3
FB#4	-2.2	0.8	1.3	1.1	0.8	0.4	0.1	-0.1	-0.2	-0.4	-0.5	-0.6
FB#5	-4.1	-3.7	-3.3	-3.0	-2.8	-2.6	-2.5	-2.4	-2.4	-2.3	-2.3	-2.4
FB#6	3.4	0.2	-1.6	-2.7	-3.4	-3.9	-4.3	-4.5	-4.7	-4.8	-4.8	-4.9
FB#7	-20.3	-16.7	-14.9	-13.7	-12.9	-12.3	-11.8	-11.4	-11.1	-10.8	-10.6	-10.3
FB#8	-10.5	-11.5	-12.0	-12.3	-12.3	-12.3	-12.2	-12.0	-11.8	-11.6	-11.4	-11.2
FB#9	-17.5	-12.5	-9.8	-8.5	-7.8	-7.5	-7.4	-7.6	-7.8	-8.2	-8.5	-8.9

Table 5-12: Annual Shoreline Change Rates for 2060 Beach Nourishment Project at SBEACH Model Reaches

SBEACH Reach	Shoreline Change Rates (ft/yr)											
	2060	2061	2062	2063	2064	2065	2066	2067	2068	2069	2070	2071
FB#1	-34.3	-29.7	-23.7	-18.5	-13.8	-6.5	3.6	7.7	7.6	5.7	3.2	1.3
FB#2	3.9	-1.1	-4.5	-6.1	-6.6	-6.1	-3.7	-0.7	1.3	2.1	2.0	1.6
FB#3	0.2	0.3	0.3	0.2	-0.1	-0.3	-0.5	-0.6	-0.6	-0.5	-0.4	-0.3
FB#4	-1.9	1.0	1.5	1.2	0.8	0.4	0.2	-0.1	-0.2	-0.4	-0.6	-0.7
FB#5	-4.3	-3.8	-3.3	-3.0	-2.7	-2.5	-2.4	-2.3	-2.2	-2.2	-2.2	-2.2
FB#6	4.3	0.8	-1.1	-2.3	-3.0	-3.5	-3.9	-4.1	-4.3	-4.4	-4.5	-4.6
FB#7	-21.0	-17.1	-15.2	-13.9	-13.0	-12.3	-11.8	-11.4	-11.1	-10.8	-10.5	-10.2
FB#8	-10.5	-11.6	-12.2	-12.5	-12.6	-12.5	-12.4	-12.2	-12.0	-11.7	-11.5	-11.3
FB#9	-17.2	-12.3	-9.8	-8.5	-7.9	-7.6	-7.6	-7.8	-8.1	-8.5	-8.9	-9.3

Table 5-13: Total 12 Years Average Shoreline Change Rates of Future with Project Scenarios at SBEACH Reaches

SBEACH Reach	Average Shoreline Change Rates (ft/yr)			
	Jan2024 Fill	Jan2036 Fill	Jan2048 Fill	Jan2060 Fill
FB#1	-5.9	-6.7	-6.1	-8.1
FB#2	-1.1	0.1	0.2	-1.5
FB#3	2.7	0.4	0.2	-0.2
FB#4	-6.3	-0.4	0.0	0.1
FB#5	-2.8	-2.9	-2.8	-2.8
FB#6	-2.8	-2.9	-3.0	-2.6
FB#7	-13.0	-13.1	-13.1	-13.2
FB#8	-11.7	-11.7	-11.7	-11.9
FB#9	-9.3	-9.3	-9.3	-9.5

Table 5-14: Last 8 Years Average Shoreline Change Rates of Future with Project Scenarios at SBEACH Reaches

SBEACH Reach	Average Shoreline Change Rates (ft/yr)			
	Jan2024 Fill	Jan2036 Fill	Jan2048 Fill	Jan2060 Fill
FB#1	3.0	3.0	3.3	1.1
FB#2	1.2	0.9	1.1	-1.3
FB#3	1.4	0.2	0.1	-0.4
FB#4	-1.9	-0.3	-0.1	-0.1
FB#5	-3.1	-2.6	-2.4	-2.3
FB#6	-4.2	-4.4	-4.4	-4.0
FB#7	-11.3	-11.4	-11.4	-11.4
FB#8	-11.8	-11.8	-11.8	-12.0
FB#9	-7.9	-8.0	-8.0	-8.2

It was noted that:

a) the shoreline change rate at Beach-fx model reach 8 is higher for the January 2024 project, and

b) the shoreline change rates at Beach-fx model reach 12 are higher for all four future beach nourishment projects.

These are discussed as below:

a) Higher shoreline change rate at Beach-fx model reach 8

The shoreline model for 2024 beach nourishment project was developed based on the December 2018 shoreline and the proposed January 2024 beach nourishment project. There is a significant shoreline orientation difference at Beach-fx model reach 8 compared to the adjacent shoreline orientations (see Figure 5-3). This shoreline orientation at Beach-fx model reach 8 will increase the longshore sediment transport rates along Beach-fx model reach 8. Therefore, the annual shoreline change rates for the January 2024 beach nourishment will be higher at the Beach-fx model reach 8 until the shoreline transitions to a more stable equilibrium orientation.

Figure 5-3: Location of Beach-fx Model Reach 8, and Shoreline on January 10, 2019

b) Higher shoreline change rates at Beach-fx model reach 12

The Beach-fx model reach 12 includes transect 2838 while transect 2040 is located in Beach-fx model reach 13. Figure 5-4 and Figure 5-5 illustrate the USACE's initial beach fill design at transects 2838 and 2840. After USACE reviewed the initial beach fill design, USACE recommended to modify the beach fill design to provide more beach fill volumes at transects 2838 and 2840 (see Figure 5-6 and Figure 5-7). The modified beach fill design changed the shoreline orientation at Beach-fx model reach 12, and thus increased shoreline change rates along Beach-fx model reach 12 as the shoreline transitions to a more stable equilibrium orientation.

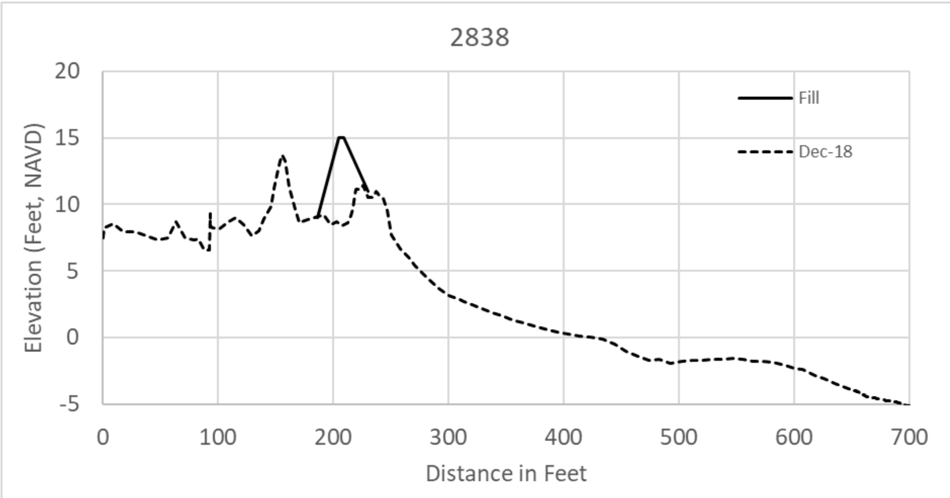

Figure 5-4: Initial Beach Fill Design at Transect 2838

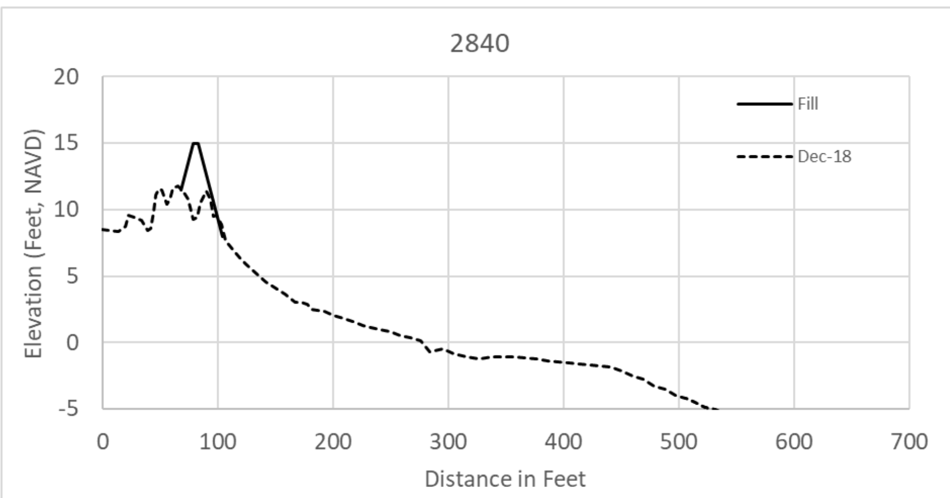

Figure 5-5: Initial Beach Fill Design at Transect 2840

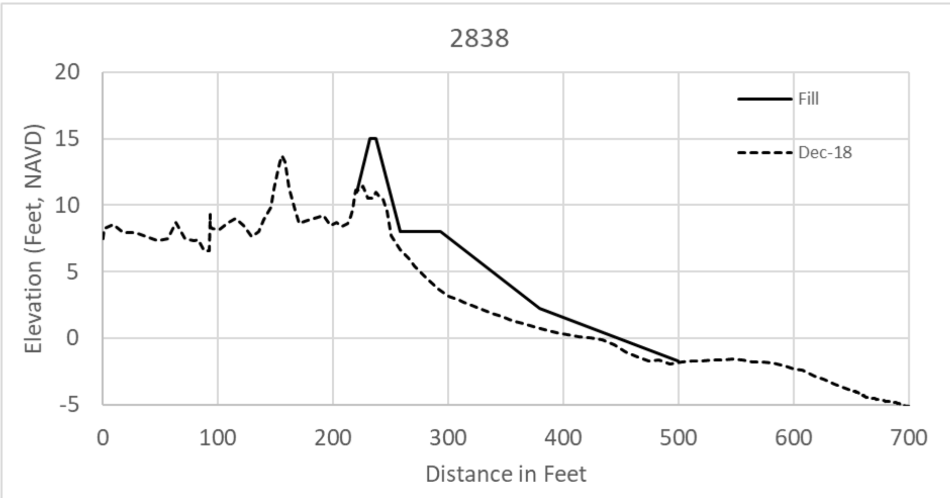

Figure 5-6: Modified Beach Fill Design at Transect 2838

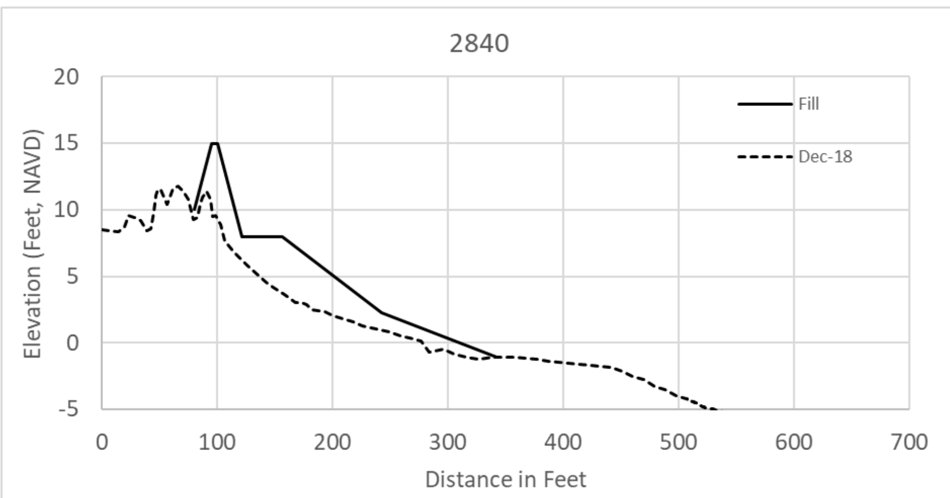

Figure 5-7: Modified Beach Fill Design at Transect 2840

6. CONCLUSION

A spectral wave model was developed to transform a time series of waves from offshore to nearshore at the Folly Beach project site. The spectral wave model was calibrated and validated using Oceanweather hindcast wave data at station 10452, USACE WIS wave data at station 63348, and NOAA buoy 41004 wave data. A 12-year time series of offshore wave data from

2008 to 2019 was transformed to nearshore at the project site using the calibrated spectral wave model.

A GenCade shoreline model was developed to simulate Folly Beach shoreline change rates. The spectral wave model simulated time series nearshore wave conditions were utilized as input forces for the GenCade shoreline model. The shoreline model was calibrated and validated using the historical measured shoreline locations for three different periods. The calibrated shoreline model was applied to simulate long-term Folly Beach shoreline change rates for the USACE proposed four future beach nourishment projects in January 2024, January 2036, January 2048, and January 2060. The shoreline model simulated shoreline change rates presented in Table 5-3 through Table 5-14 at each reach were recommended for the USACE Beach-fx economic model and SBEACH storm profile model.

7. References

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Sub-Appendix D:

Folly Beach

Offshore Borrow Area Analysis

February 2020

Effects of four potential borrow areas on wave propagation at Folly Beach, South Carolina

by S.C. Dillon

INTRODUCTION: The USACE Wilmington District (SAW) requested assistance in conducting a wave assessment for four proposed borrow areas near Folly Beach, South Carolina. Folly Beach (32°39'18.65"N 79°56'25.32"W) is located southwest of Charleston harbor near Charleston, South Carolina. The excavation of these sites will cause changes in the nearshore bathymetry, which will affect the wave transformation in the area. An assessment of the effects these borrow areas have on the nearshore wave propagation will help SAW evaluate each potential borrow area site. To complete this assessment, the STeady-state WAVE (STWAVE) model (Smith et al. 2001, Massey et al. 2011), which is a phase-averaged spectral model for wave generation, propagation and transformation, was used to simulate wave transformation in the Folly Beach area.

STWAVE: STWAVE is a steady-state spectral wave model for nearshore wave generation, propagation, transformation, and dissipation. STWAVE numerically solves the steady-state conservation of spectral wave action along backward-traced wave rays:

$$(C_g)_i \frac{\partial}{\partial x_i} \frac{C C_g \cos \alpha E(\sigma,\theta)}{\sigma} = \sum \frac{S}{\sigma} \qquad (1)$$

Where *i* is tensor notation for *x*- and *y*- components, C_g is group celerity, θ is wave direction, *C* is wave celerity, σ is wave angular frequency, *E* is wave energy density, and *S* is energy source and sink terms. Source and sink mechanisms include surf-zone wave breaking, wind input, wave-wave interaction, whitecapping, and bottom friction. STWAVE is formulated on a Cartesian grid, with the x-axis oriented in the cross shore direction (I) and the y-axis oriented alongshore (J), generally parallel with the shoreline. Angles are measured counterclockwise from the grid x-axis.

GRID DEVELOPMENT: In order to capture the effects of each borrow area, four STWAVE grids were developed, which extended alongshore from Sullivan's Island to Kiawah Island and seaward to a depth of 90 ft. (25 m). The Cartesian grid resolution of all four grids was approximately 164 ft. (50 m) and is comprised of 643 cells in the cross-shore direction (I) and 817 cells in the alongshore direction (J). The projection of the grid is in State Plane South Carolina (FIPS 3900), with a vertical datum relative to NAD83 (meters). The properties of all four STWAVE domains are provided in Table 1.

Table 1. STWAVE Grid Properties

Horizontal Projection	Vertical Projection	Grid Origin (x,y) [m]	Azimuth [deg]	Δx/Δy [ft]	Number of Cells	
					I	J
State Plane South Carolina FIPS 3900	NAD83	(742990.0, 83070.0)	125.2	164	643	817

The outlines of the potential borrow area sites are shown in Figure 1. The four borrow areas include Stono Inlet, which is the most westward borrow area sites and is located near Stono Inlet. This area is proposed to be excavated to a depth of -41ft. The Central site is located adjacent to the Stono Inlet sites and offshore of Folly Beach. The Central borrow area site is proposed to be excavated to a depth of -35ft in the upper right quadrant and to a depth of -40ft in the lower left quadrant of the area. The Lighthouse borrow area is located between the Central site and Lighthouse Inlet, and is proposed to be excavated to a depth of -22ft. Finally, the last borrow area, Seaward, is located seaward of South Carolina's offshore territory and offshore from the Lighthouse Inlet site and is proposed to be excavated to a depth of -45 ft. The topography and bathymetry data to populate the STWAVE domain were obtained from the South Carolina Storm Surge Study - FEMA grid. The bathymetry was modified for each proposed borrow area site by deepening each site to the proposed dredge depth as described above and shown in Figure 1. Depictions of the modified depths as represented in the STWAVE domains are included in Appendix A.

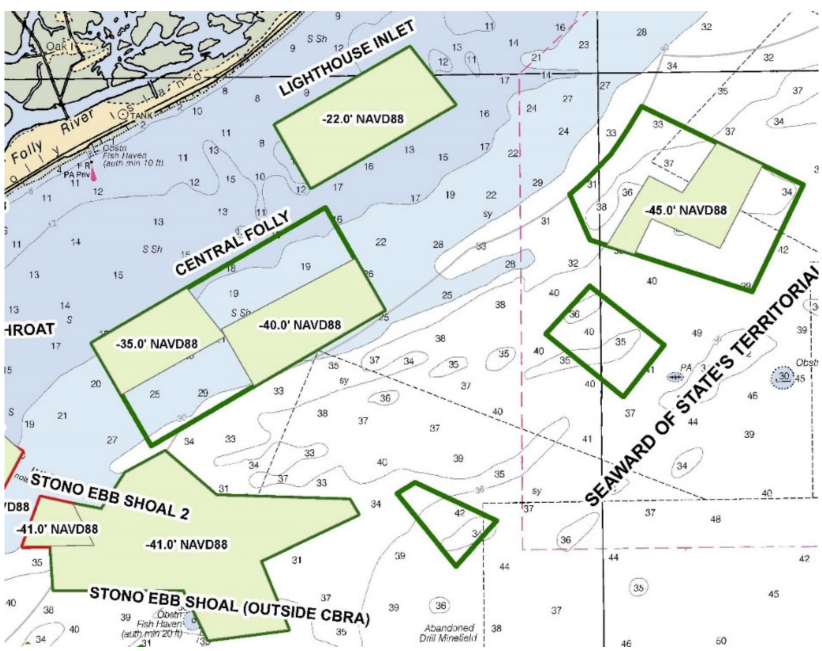


Figure 1: Dredge depths (shown in light green) of the four potential borrow areas offshore of Folly Beach, SC.

OFFSHORE BOUNDARY SPECTRA: To determine the boundary forcing conditions for STWAVE a wave assessment was conducted for the Folly Beach area, to capture the mean monthly, maximum monthly and extreme events using the wave data from the Wave Information Study (WIS) hindcast. The hindcast data provides a record of 37 years, from 1980 to 2017. Stations 63350 and 63348 were chosen as the primary stations of interest due to their close proximity to Folly Beach.

A comparison of the mean monthly wave heights and the max monthly wave heights for all four seasons for both stations is included in Appendix B. These histograms show that the mean monthly wave height, across all seasons, is less than 6.5 ft. (2 m) with the most frequent mean monthly wave height at approximately 3.6 ft. (1.1m). The histograms also show that the most frequent max monthly wave height is approximately 8.5 ft. (2.6 m).

A comparison of the mean monthly wave periods and the max monthly wave periods, show that the average max monthly wave period is approximately 9 seconds and the average mean monthly wave period is approximately 8.4 seconds.

The extreme plots for both stations are shown in Appendix C. For station 63348, the extreme wave heights range from 17.7 to 20.17 ft. (5.38 to 6.15 m) for the top 10 events with peak wave periods ranging from 12-18 seconds. For station 63350, the top 10 extreme events contained wave heights ranging from 17.2 to 18.7 ft. (5.25 to 5.70 m) with peak periods of 14 to 18 seconds.

Onshore propagation for the area is between 60 and 225 degrees.

In order to encompass the climate of the area nine conditions were identified for inclusion as boundary forcing for STWAVE. The chosen conditions, shown in Table 2, included a mean monthly condition composed of a wave height of 3.6 ft. (1.1 m) with an 8.4-sec. period, and a max monthly condition with an 8.5 ft. (2.6 m) wave height and 9-sec. period. Also chosen was the highest extreme event, which occurred for WIS station 63348, and had a 20.3 ft. (6.2 m) wave height and an 18-sec. period. Both the mean and maximum monthly condition spectra were simulated with a mean direction ranging from 60 to 225 degrees, for a total of four directions, and the extreme event was simulated at 97 degrees (as occurred in the extreme event).

The resolved spectra for each condition was represented by 35 frequency bands, ranging from 0.37 Hz (2.7 sec) to 0.03 Hz (33.3 sec), and 72 angle bands, from an angle of 0 degrees to 355 degrees with respect to the x-axis. Frequency and angular resolution were 0.01 Hz and 5 degrees, respectively.

Table 2: *STWAVE Conditions for Simulation*

	Wave Height (ft)	**Period (sec)**	**Direction (deg)**	**Number of STWAVE Simulations:**
Mean Monthly	3.6	8.4	60°, 115°, 170°, 225°	4
Max Monthly	8.5	9	60°, 115°, 170°, 225°	4
Extreme Event	20.3	18	97°	1
Total number of STWAVE simulations per borrow area:				**9**

MODEL EXECUTION: Each STWAVE simulation conducted used the full-plane mode of STWAVE to allow for wave generation and transformation in a 360-degree plane. The full-plane version of STWAVE uses an iterative solution process that requires user-defined convergence criteria to signal a suitable solution. Boundary spectra information is propagated from the boundary throughout the domain and iteratively executes until it reaches a convergent state. The convergence criteria includes the maximum number of iterations to perform per time-step, the relative difference in significant wave height between iterations, and the minimum percent of cells that must satisfy the convergence criteria (i.e., have values less than the relative difference.) Convergence parameters were selected based on a previous study by Massey et al. (2011) in which the sensitivity of the solution to the final convergence criteria was examined. The relative difference and minimum percent of cells were set as (0.1, 100.0) and (0.1, 99.8) for the initial and final iterations, respectively. STWAVE was set up with parallel in-space execution whereby each computational grid was divided into different partitions (in both the x- and y-direction), with each partition executing on a different computer processor. The number of partitions in the x direction was 13, while the number of partitions in the y direction was 17. The maximum number of initial and final iterations was set to a value of 20 iterations, higher than the largest partition size.

Thirteen locations were identified within the STWAVE grid to save significant wave height, peak period, mean period and mean wave direction from each of the 9 wave conditions simulated. The x y coordinates of these locations are included in Table 3 and depicted in Figure 2.

Table 3: Location of Save Points inside STWAVE domain

	x [m]	y [m]		x [m]	y [m]
pt1	713942	94557.7	**pt7**	709691	91561.9
pt2	713410	94183.2	**pt8**	708922	91264.5
pt3	712732	93583	**pt9**	708088	90798.9
pt4	712259	93126.8	**pt10**	707172	90275.7
pt5	711686	92723.5	**pt11**	706571	89913.3
pt6	710742	92241.2	**pt12**	705917	88412
			pt13	703363	85266

Figure 2: Save point locations

RESULTS: Three types of figures were generated for each borrow area and boundary condition: a spectral wave height plot (Appendix D), a spectral wave height difference plot (Appendix E), and a plot depicting the significant wave height, mean period, peak period, and STWAVE wave angle, at each save point for each condition (Appendix F).

Mean Monthly Condition: The mean monthly wave condition, 3.6 ft. wave height and 8.4 sec. period, showed, as expected, the least effects caused by the borrow areas. Due to the low energy of this condition, the differences in depth caused by the excavation of the borrow areas will be minimal compared to the higher energy events. Overall, each borrow area site increased and decreased the wave heights on average ~0.5 ft. in the area of each site, with increases occurring on the perimeter and decreases in the center. The greatest impact was observed at the Central borrow area in the 60° wave direction, with decreases at the borrow area and toward shore of ~1.0 ft and slight increases of ~0.5-1.0 ft. in the perimeter, as shown in figure 3. The decrease most significantly affected the eastern shores of Folly Beach, while the increases affected Kiawah Island and the central shores of Folly Beach. The Seaward borrow area site had the least effects on the wave height. The results showed only slight increases and decreases of less than 0.5ft, which were widely distributed from the borrow area site.

Mean Monthly Condition
Central Folly Borrow Area:

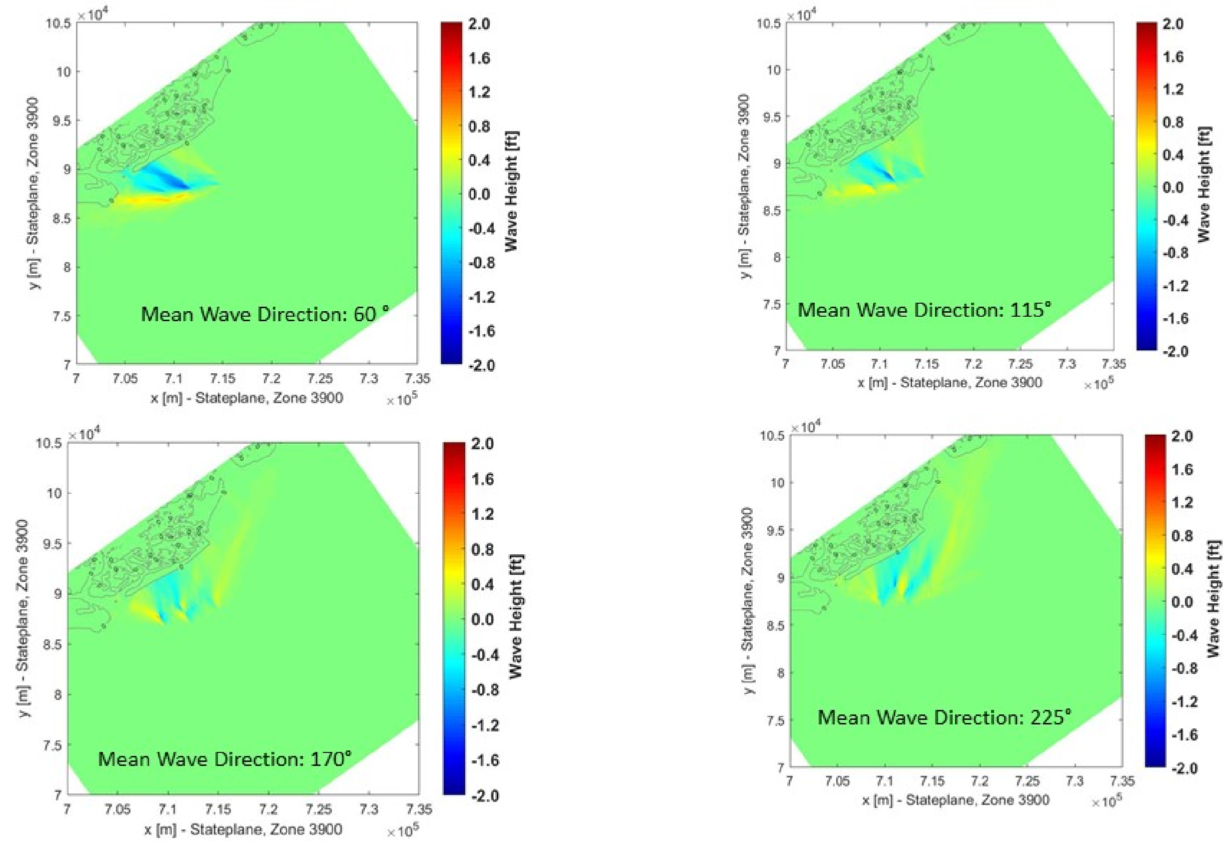


Figure 3: Mean monthly condition at the Central borrow area. Warm tones indicate increases in wave height when compared to the no borrow are simulations and cool tones represent decreases in wave height.

The save point locations provide similar results as shown in the above difference plot with most increases and decreases being ~0.5 ft. for all wave directions and borrow area sites. Under the 60° wave direction, the greatest decreases were observed by the Central borrow area site from points 9-12. Under the 115 ° wave direction, the Lighthouse borrow area showed decreases at save points 3-5 and Central showed decreases at points 7-10. Both Lighthouse and Central gave a slight increase at point 6. The 170° wave direction showed decreases from the Lighthouse borrow area at points 1-3 and the Central borrow area showed decreases at points 6-8, with increases at points 10. Finally, under the 225° wave direction, the Lighthouse borrow area showed increases at points 4 and 5 and decreases at points 1 and 2. While the Central borrow area, showed decreases at points 4-7 and an increase at point 9.

Max Monthly Condition: The max monthly condition, 8.5 ft. wave height and 9 sec. period, showed greater effects on the wave heights due to the presence of the borrow area sites when compared to the mean monthly condition. Overall, the locations of the increases and decreases were similar to what was observed under the mean monthly condition, but the magnitude and extent of the difference is greater. Under the max monthly condition, on average increases and decreases of ~2 ft. were observed due to the borrow area sites. The Central borrow area site showed the greatest effects on the wave heights, shown in Figure 4. The Central borrow area site showed decreases of ~4.0 ft. in the 60 ° wave direction, with increases of ~3.0 ft. in the perimeter, and increases and decreases of 1.0-2.0 ft. in the other wave directions. The Seaward borrow area showed the smallest effects on the wave heights with diffuse differences of less than 1.0 ft.

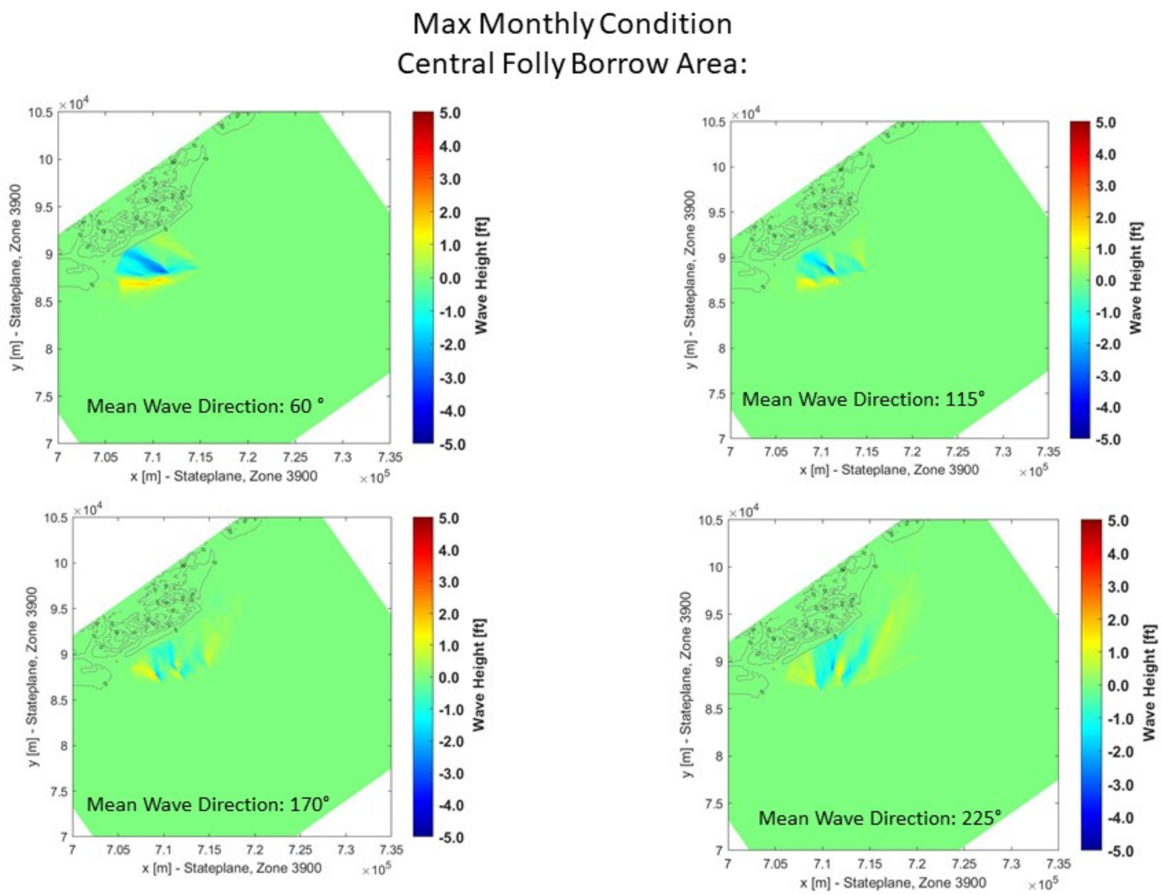


Figure 4: Max Monthly Condition at the Central borrow area site. Warm tones indicate increases in wave height when compared to the no borrow are simulations and cool tones represent decreases in wave height.

The save point locations show differences only under the most oblique wave angles (60° and 225°). Under the 60° wave direction, decreases at points 4 and 5 for the Lighthouse borrow area and decreases at points 9 and 10 for the Central borrow area occur. As well as, under the 225° wave direction, the Central borrow area shows decreases at points 4, 5, and 7 with increases at points 9 and 10.

Extreme Event Condition: The extreme event condition, 20.3 ft. wave height, 18 sec. period and 97° mean direction, showed the greatest effects due to the borrow area sites. However, for most of the borrow area sites, these effects were isolated at the borrow area and had very little effect on the coastline, as shown in Figure 5. Under the extreme condition, all of the borrow area sites show increases in the wave height of ~1.4 ft. or greater. However, the Central borrow area site shows the greatest increase of up to ~5.0 ft. and the Stono Inlet borrow area shows the widest distribution of wave height increases (~1.5 ft.). The deepening of these area compared to their surrounding allow the waves to propagate further inshore before breaking, which is most likely the cause for the observed increases under this condition.

Due to the isolation of the effects at the borrow area sites and their lack of propagation onshore, there were no differences observed in the save point location plots for wave height.

SUMMARY: The effects of the four borrow areas were investigated offshore of Folly Beach, SC using the STWAVE nearshore model. Nine identified conditions were selected to represent the mean monthly, max monthly and extreme event in the area, based on the 39-year record at the offshore WIS stations 63348 and 63350. The mean monthly condition showed the lowest impacts on wave heights due to the borrow areas with decreases of ~0.5 ft. in the borrow area and increases in ~0.5 ft. in the perimeters. The Seaward borrow area showed the least amount of effects under this condition and the Central borrow area showed the greatest effects. Under the max monthly condition, the borrow area sites showed increases and decreases in the wave heights of ~1.5 ft. with a decrease of up to ~4.0 ft. at the Central borrow area site. The Seaward borrow area site showed the least effects under this condition as well. The extreme event condition showed the greatest decreases and increases due to the borrow areas, but did not extend shoreward. The Central borrow area showed the greatest increases under the extreme event; while the Seaward borrow area continued to have the least effect on wave heights.

REFERENCES:

Massey, T.C., M.E. Anderson, J.M. Smith, J. Gomez, and R. Jones. 2011. *STWAVE: Steady-state spectral wave model user's manual for STWAVE, version 6.0*. ERDC/CHL SR-11-1. U.S. Army Engineering Research and Development Center, Vicksburg, MS.

Smith, J. M., A. R. Sherlock, and D. T. Resio. 2001. STWAVE: Steady-state spectral wave model, user's guide for STWAVE version 3.0, ERDC/CHL SR-01-01, US Army Engineer Research and Development Center, Vicksburg, MS, 80 pp.

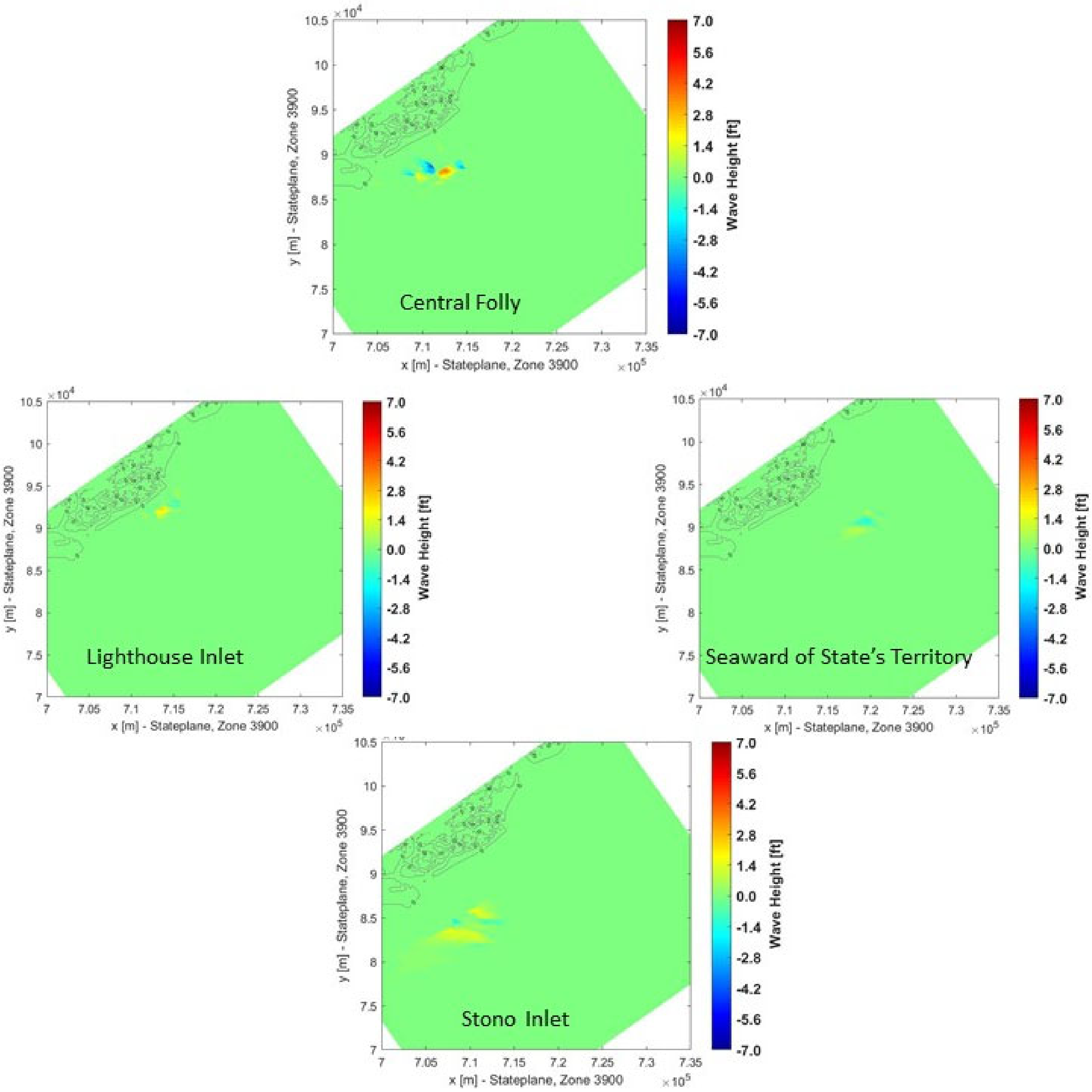
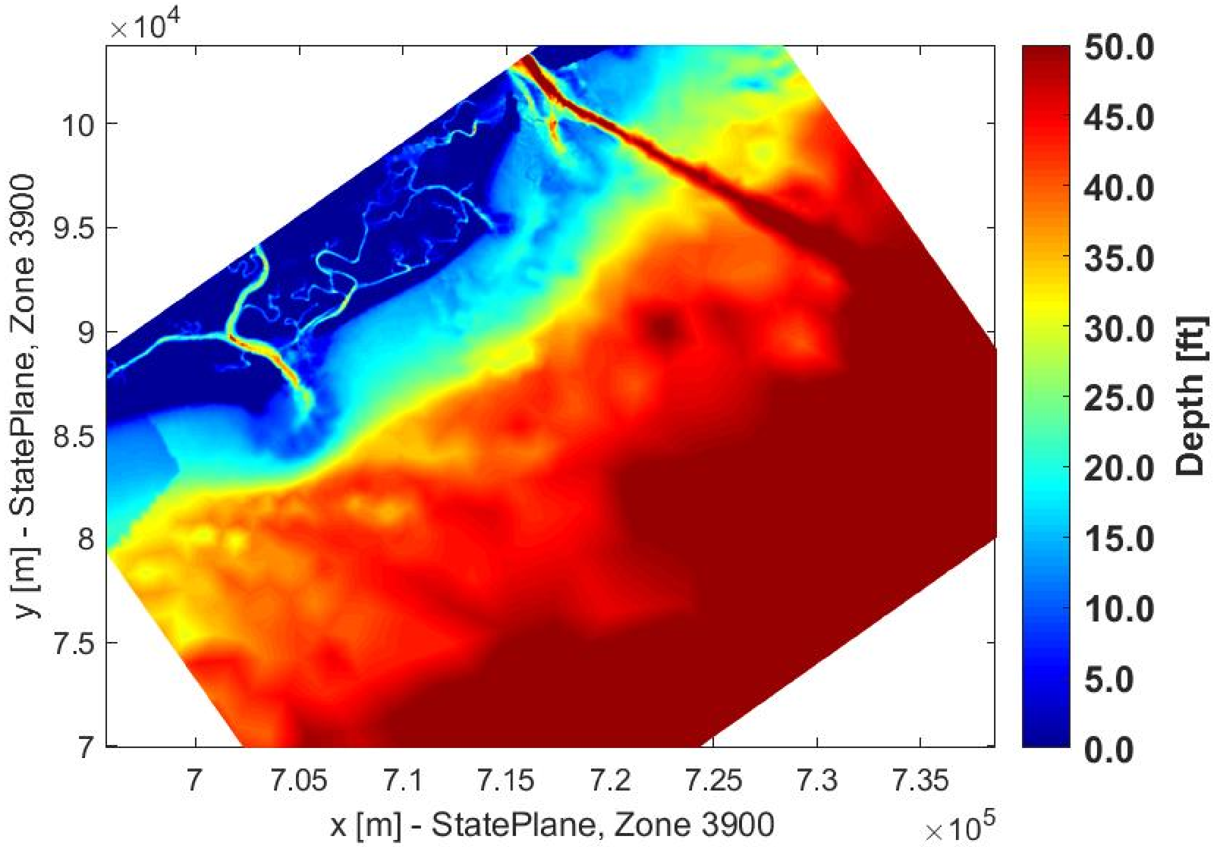


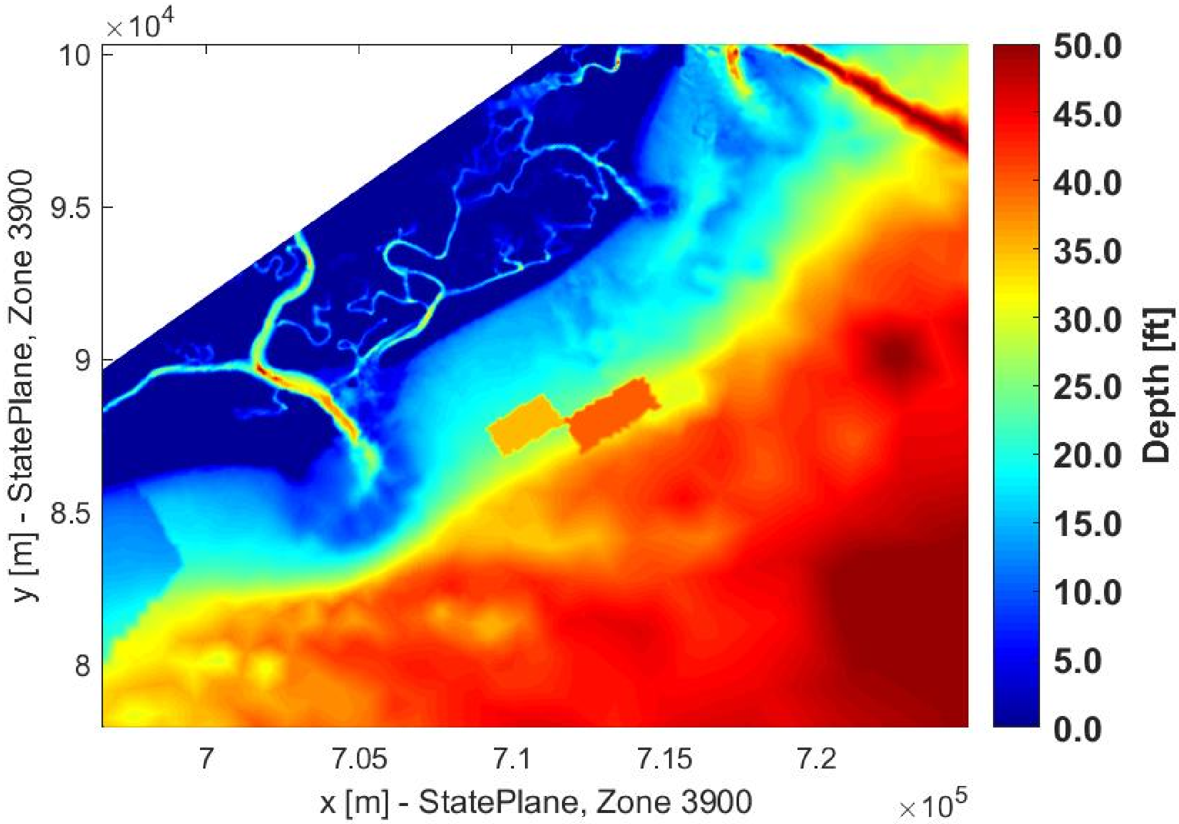
Figure 5: Extreme Event Condition at all four borrow areas. Warm tones indicate increases in wave height when compared to the no borrow are simulations and cool tones represent decreases in wave height.

Appendix A— Depth Changes in Grid

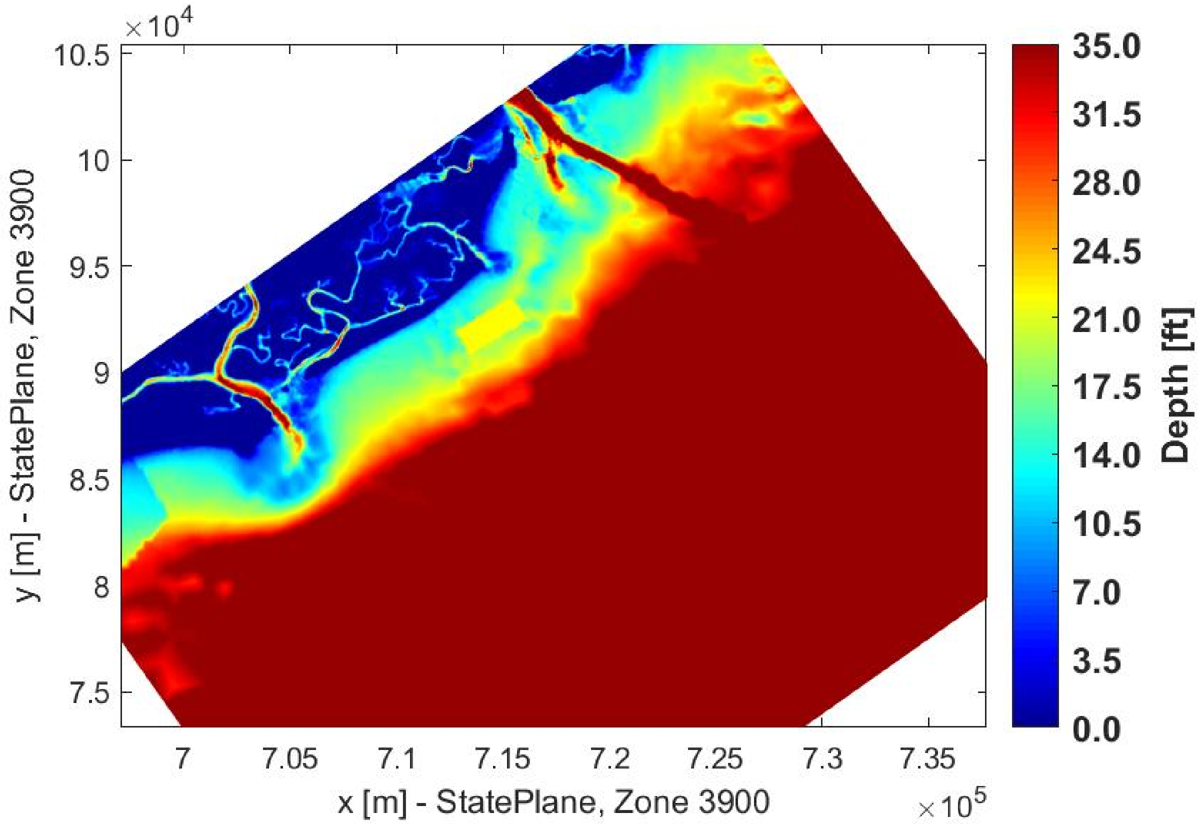
Base Condition Bathymetry



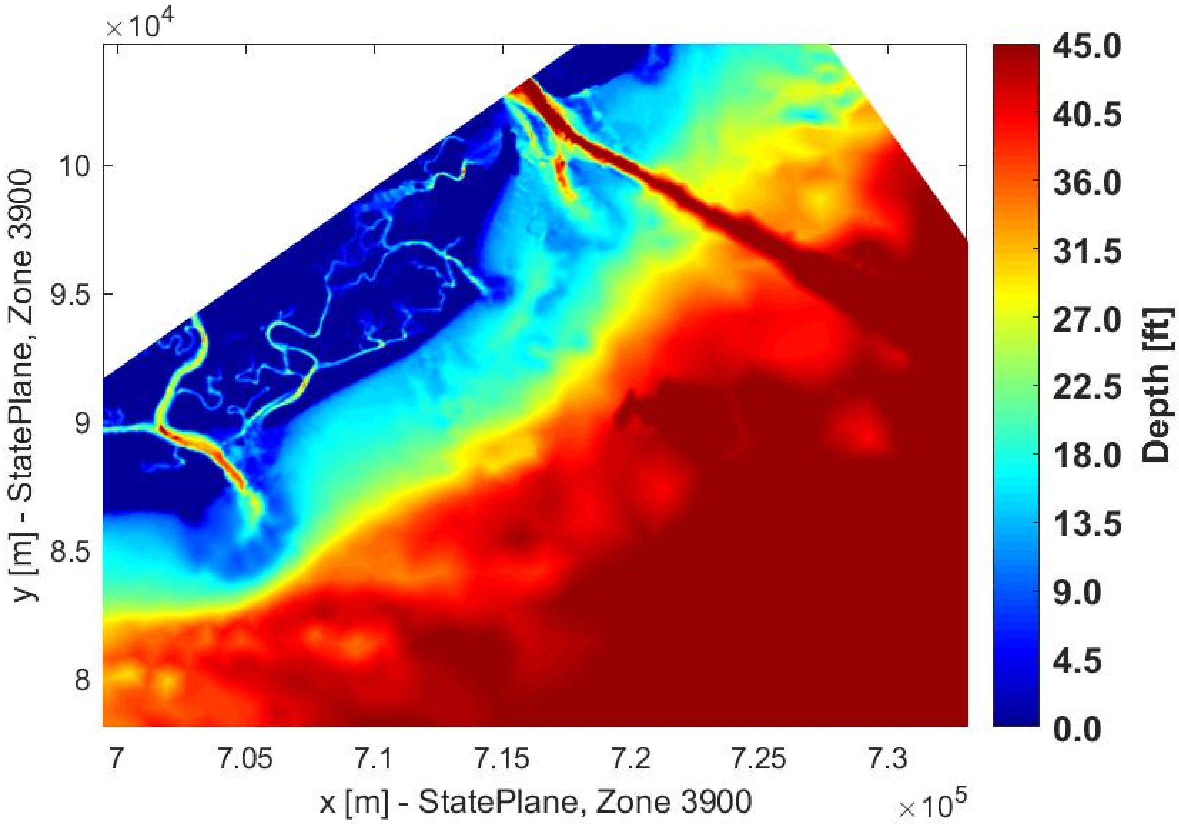
Central *Borrow Area modified Bathymetry*

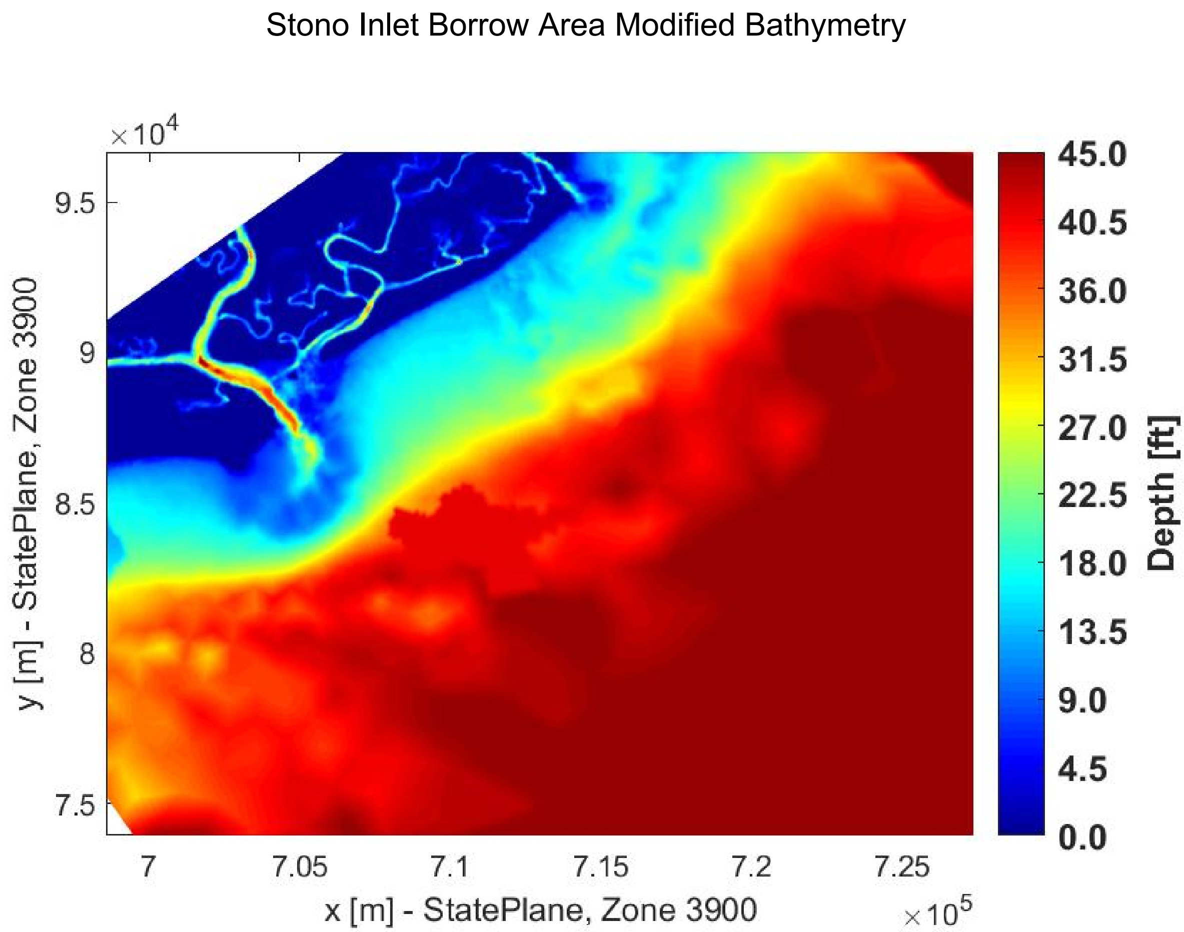


Lighthouse Borrow Area Modified Bathymetry

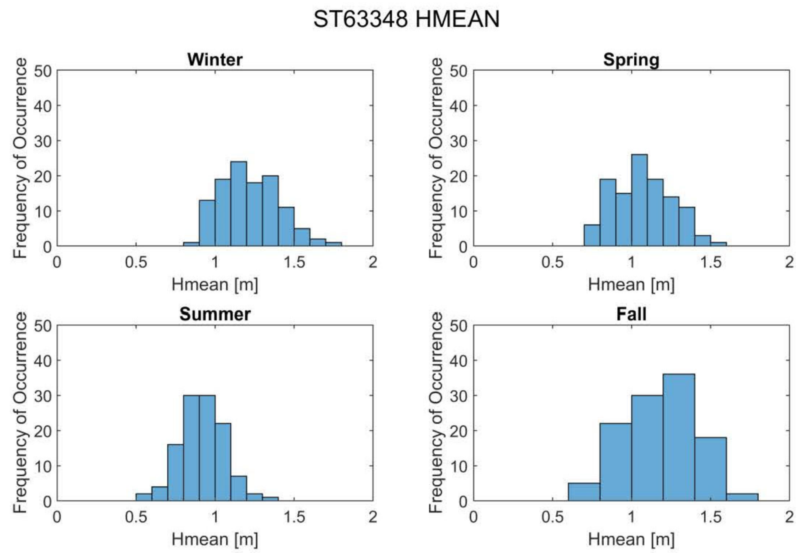


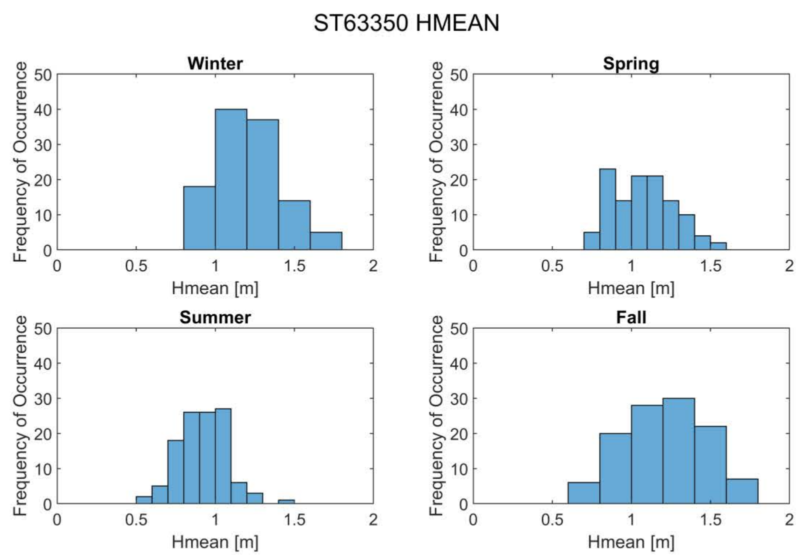
Seaward Borrow Area Modified Bathymetry



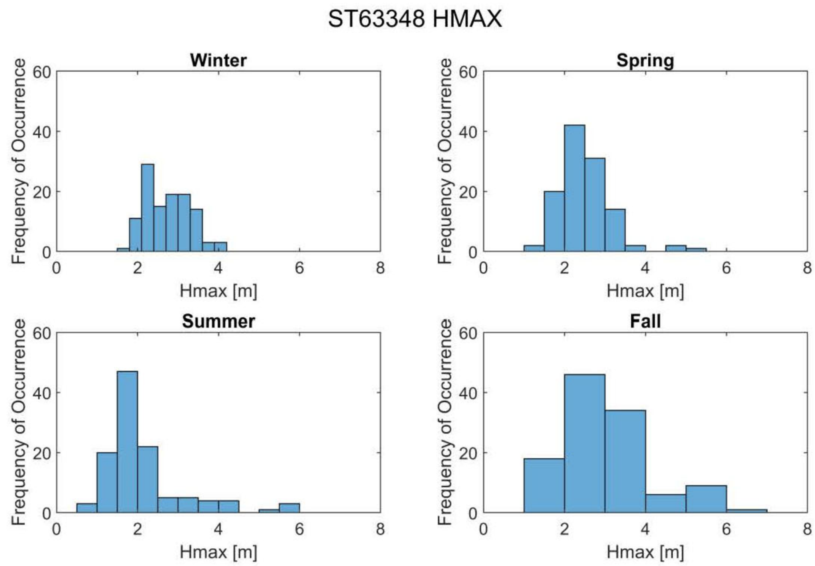

Stono Inlet Borrow Area Modified Bathymetry
×10^4
9.5
9
8.5
8
7.5
y [m] - StatePlane, Zone 3900
7
7.05
7.1
7.15
7.2
7.25
x [m] - StatePlane, Zone 3900
×10^5
45.0
40.5
36.0
31.5
27.0
22.5
18.0
13.5
9.0
4.5
0.0
Depth [ft]

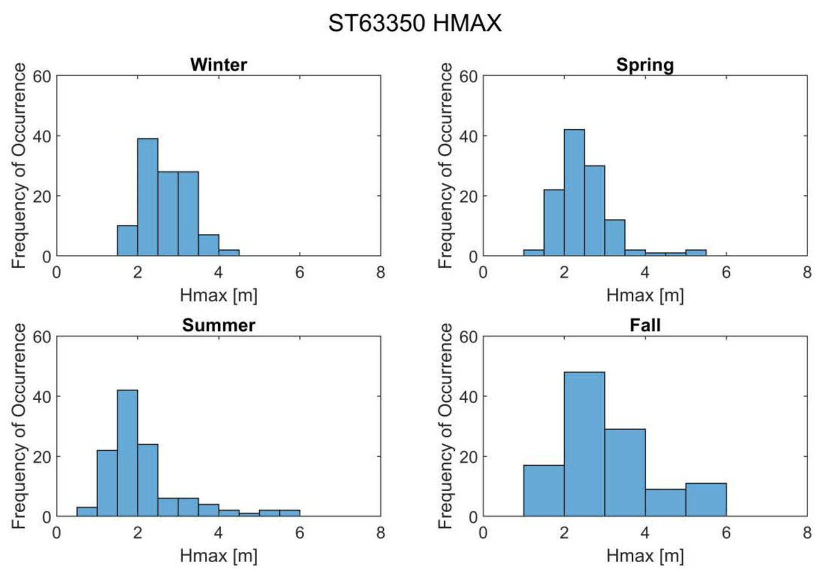
Appendix B—Seasonal Wave Height Statistics





The frequency of occurrence for the mean monthly wave conditions from 1980 to 2017

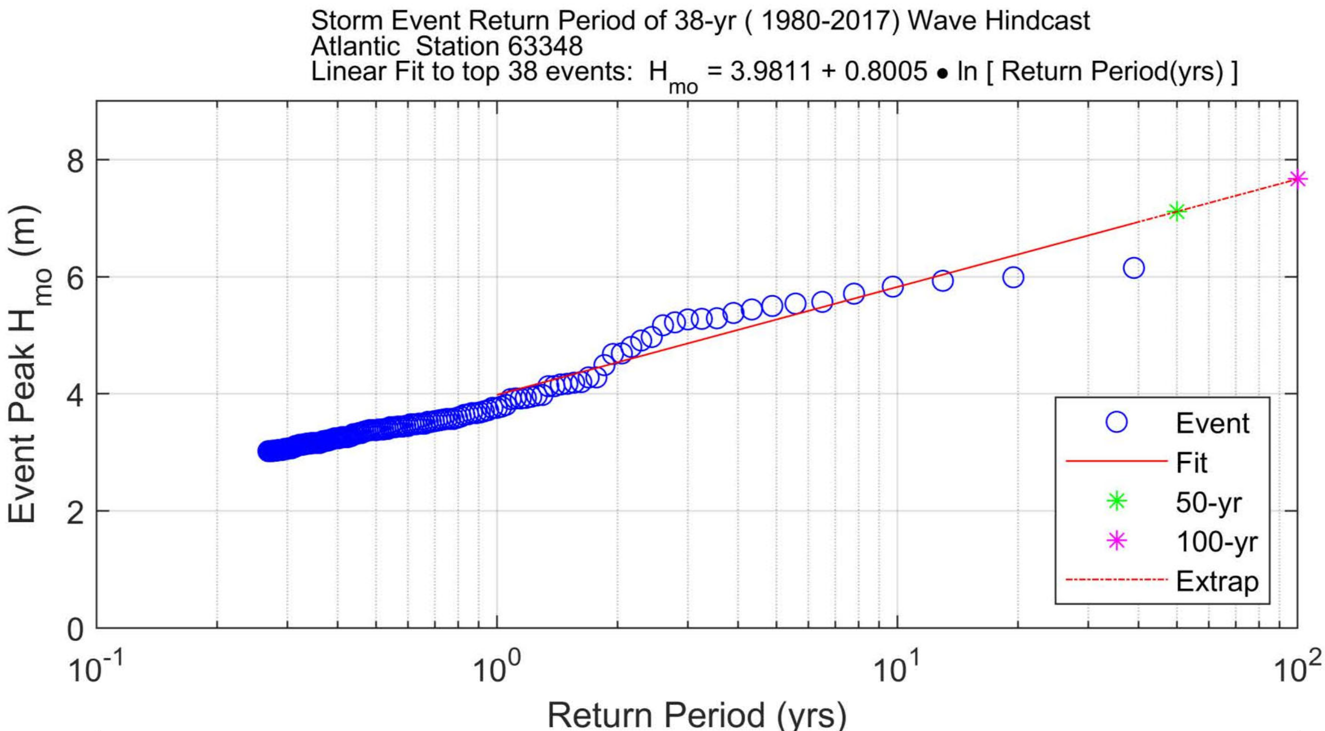




The frequency of occurrence for the max monthly wave conditions from 1980 to 2017

.

Appendix C—Extremal Analysis
Extremal Analysis WIS Station 63348

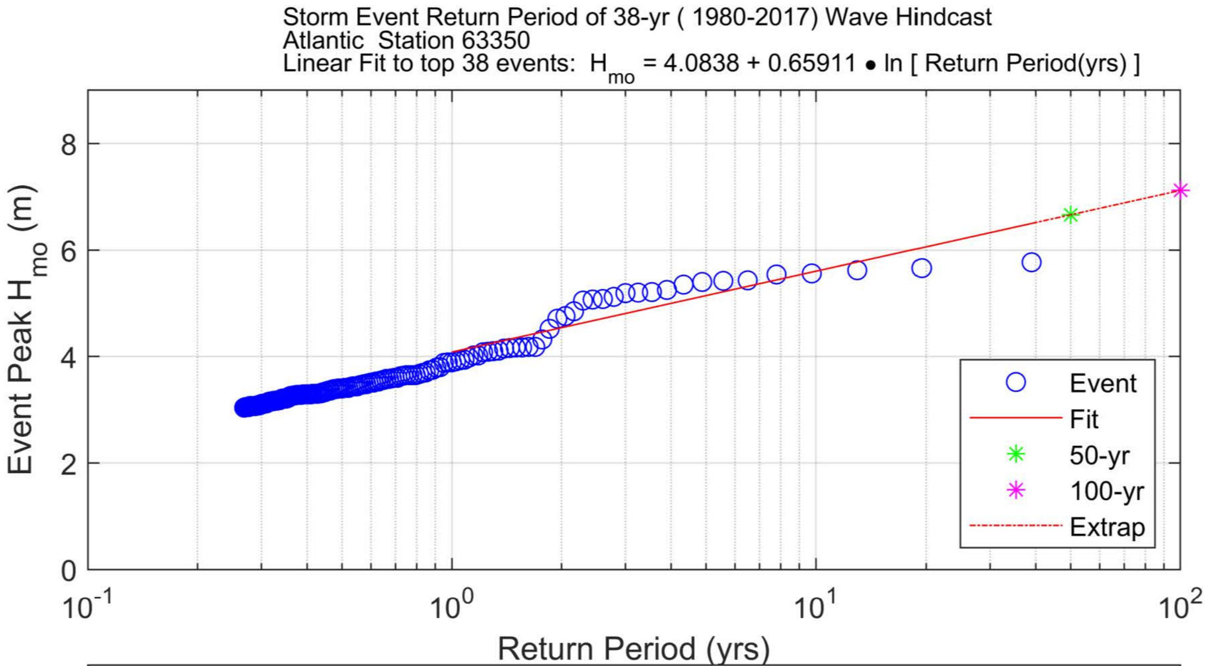


Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}	Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}
1	1989/09/22 02:00	6.15	18.25	97.0	6	1996/09/05 18:00	5.57	15.74	117.0
2	2008/09/06 00:00	5.99	15.69	128.0	7	1996/07/11 19:00	5.54	15.25	131.0
3	1999/09/15 19:00	5.93	16.09	130.0	8	1985/09/26 17:00	5.50	17.17	125.0
4	2011/08/26 19:00	5.83	16.18	130.0	9	2010/09/02 22:00	5.44	17.05	125.0
5	1998/08/26 08:00	5.71	16.48	119.0	10	2016/10/08 10:00	5.38	12.90	148.0

An event is defined as any period when $H_{mo} > 3.00$m θ_{wind} is direction that waves are arriving from

Extremal Analysis WIS Station 63350



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}	Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}
1	1989/09/22 00:00	5.77	18.07	114.0	6	1985/09/26 17:00	5.43	17.19	129.0
2	2008/09/06 00:00	5.66	15.69	128.0	7	2010/09/02 23:00	5.42	17.04	127.0
3	1999/09/15 18:00	5.62	16.06	131.0	8	1996/09/05 18:00	5.40	15.77	122.0
4	1998/08/26 09:00	5.56	16.75	122.0	9	1996/07/11 20:00	5.35	15.28	132.0
5	2011/08/26 19:00	5.54	16.15	133.0	10	2012/10/27 14:00	5.25	14.93	122.0

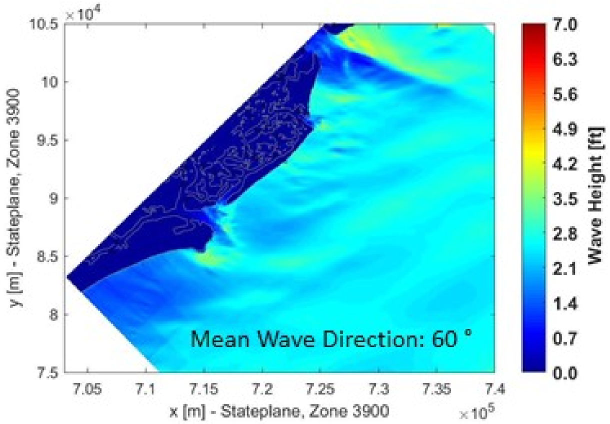
An event is defined as any period when H_{mo} > 3.00m θ_{wind} is direction that waves are arriving from

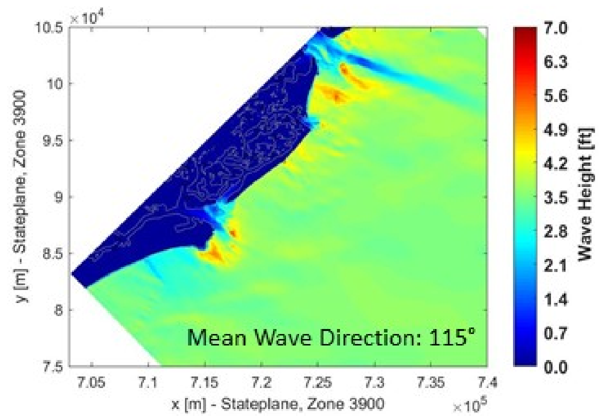
.

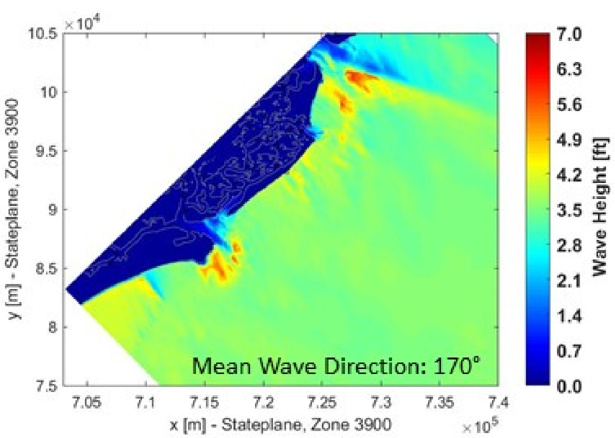
Appendix D—Spectral Wave Height Plots

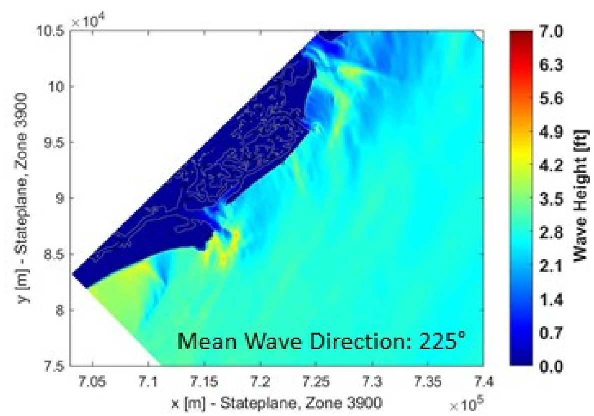
Mean Monthly Condition

Base:

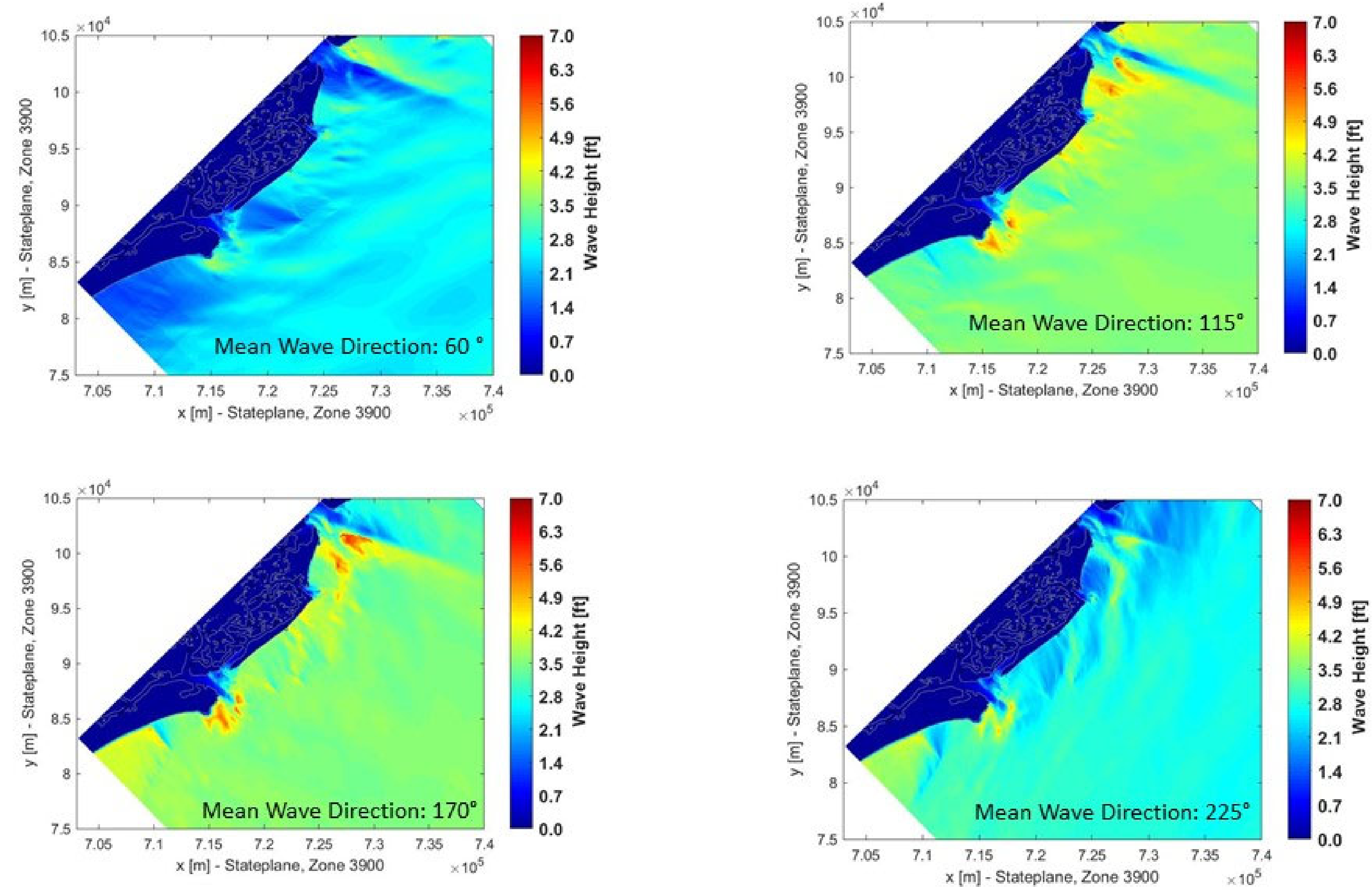




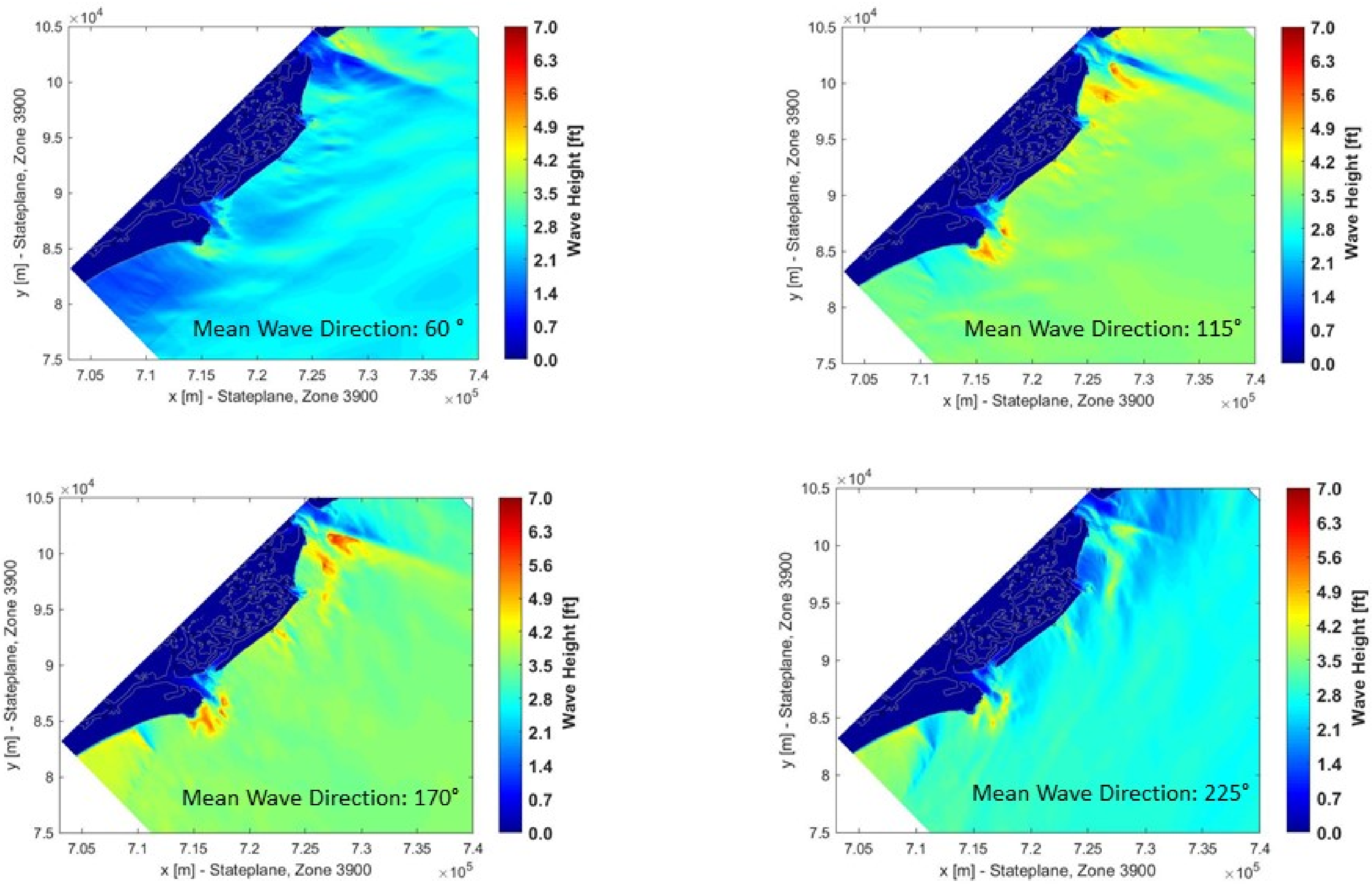




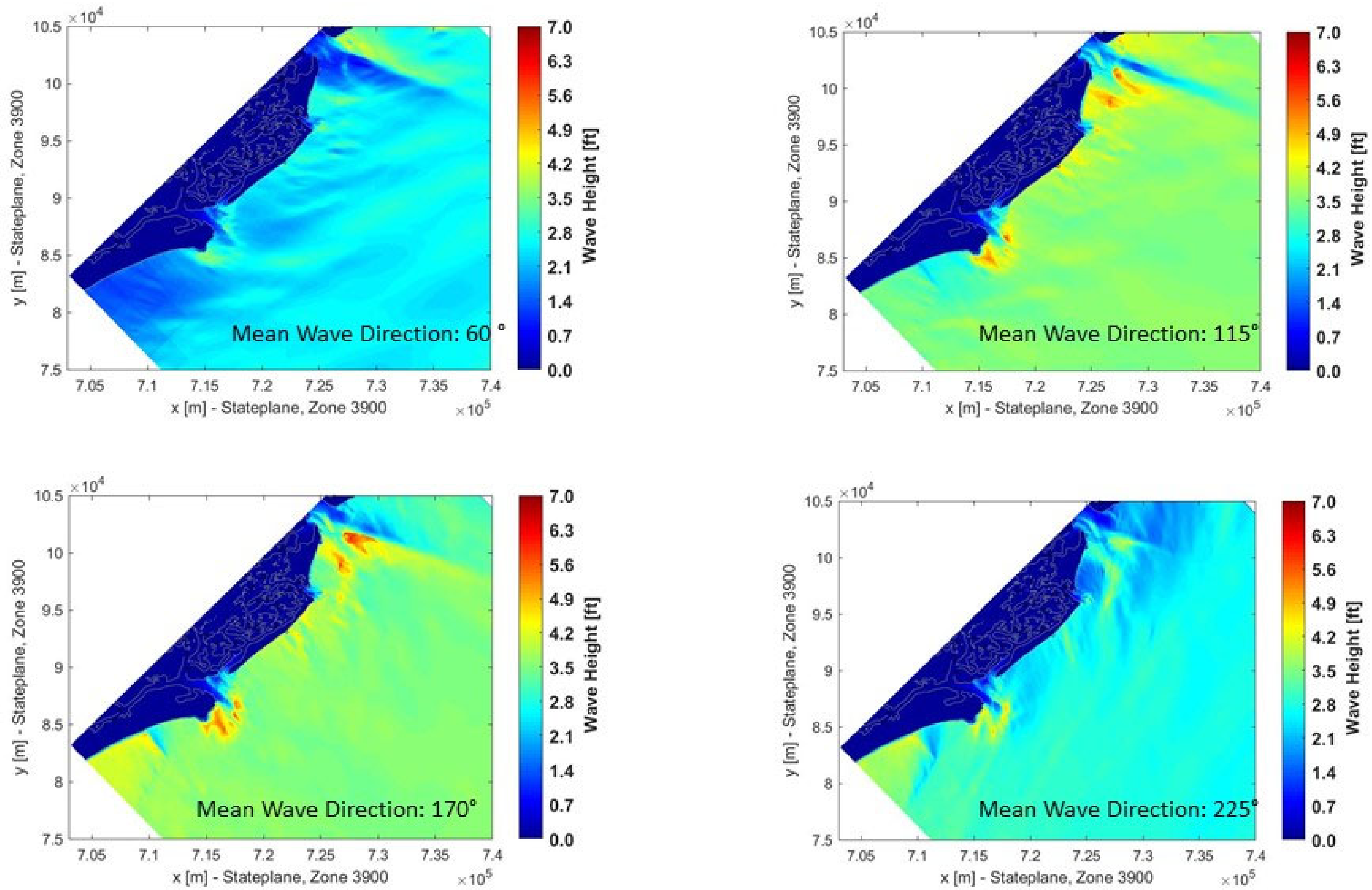
Mean Monthly Condition
Central Folly Borrow Area:



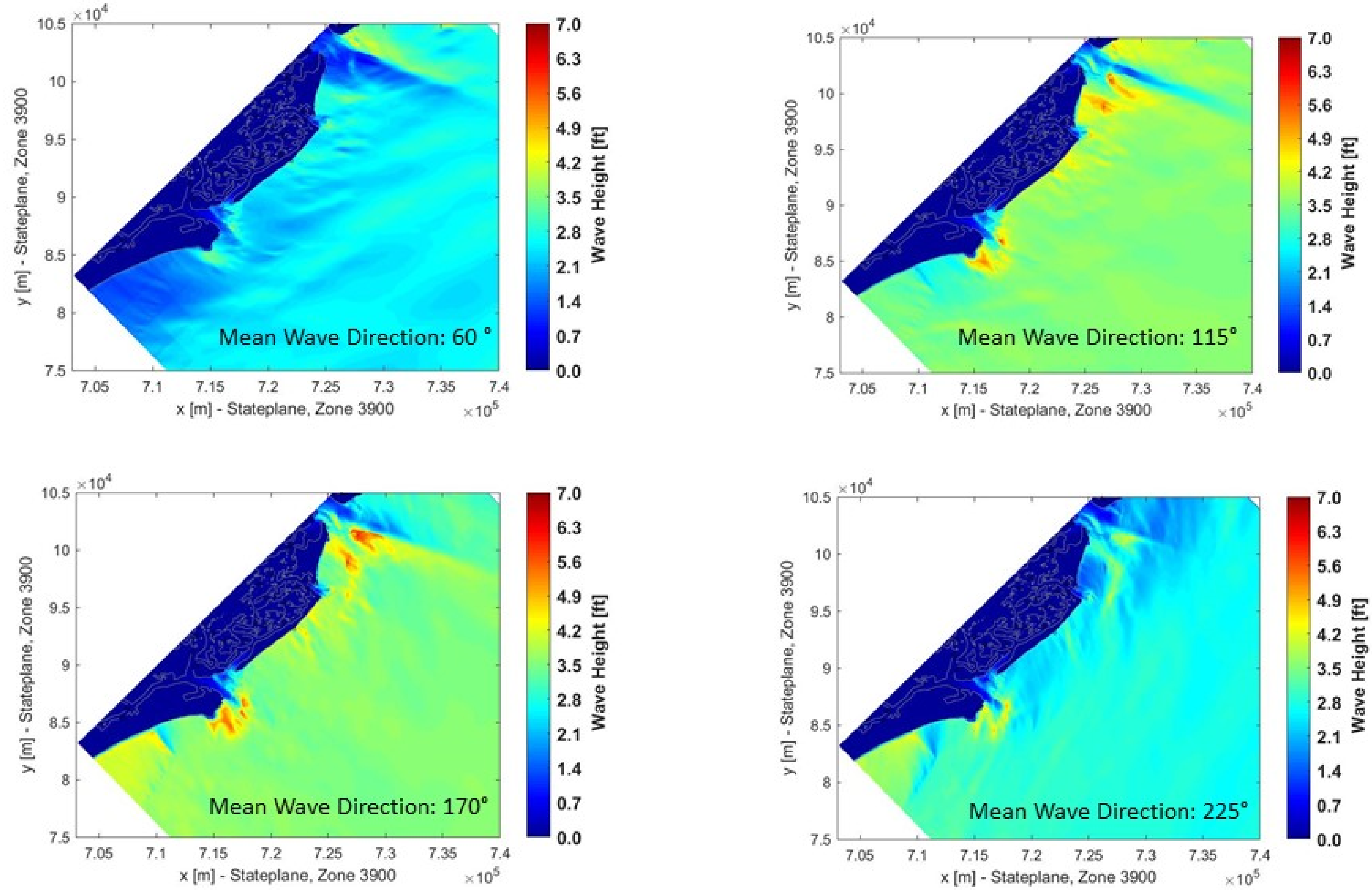
Mean Monthly Condition
Lighthouse Inlet Borrow Area:



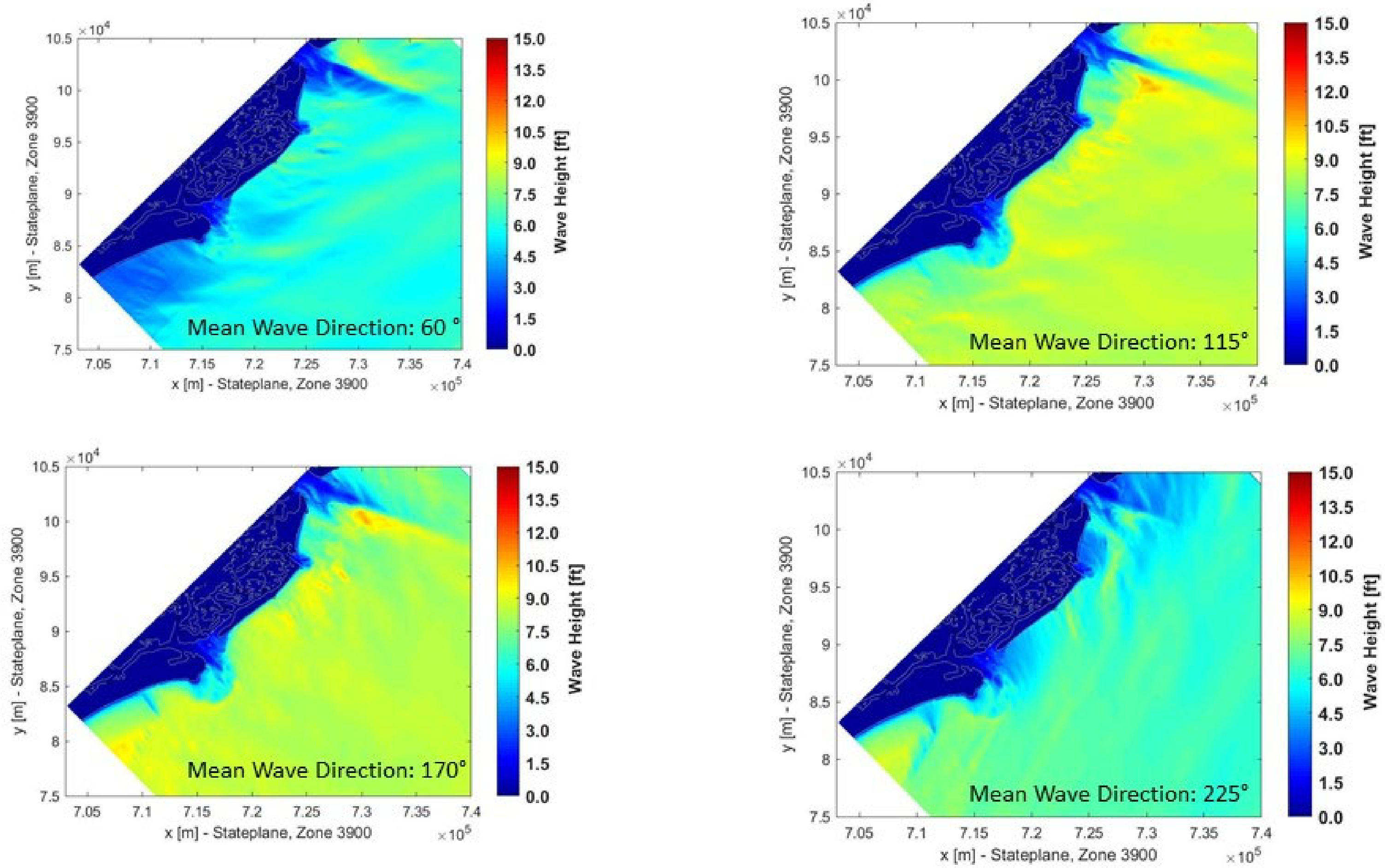
Mean Monthly Condition
Seaward of State's Territory Borrow Area:



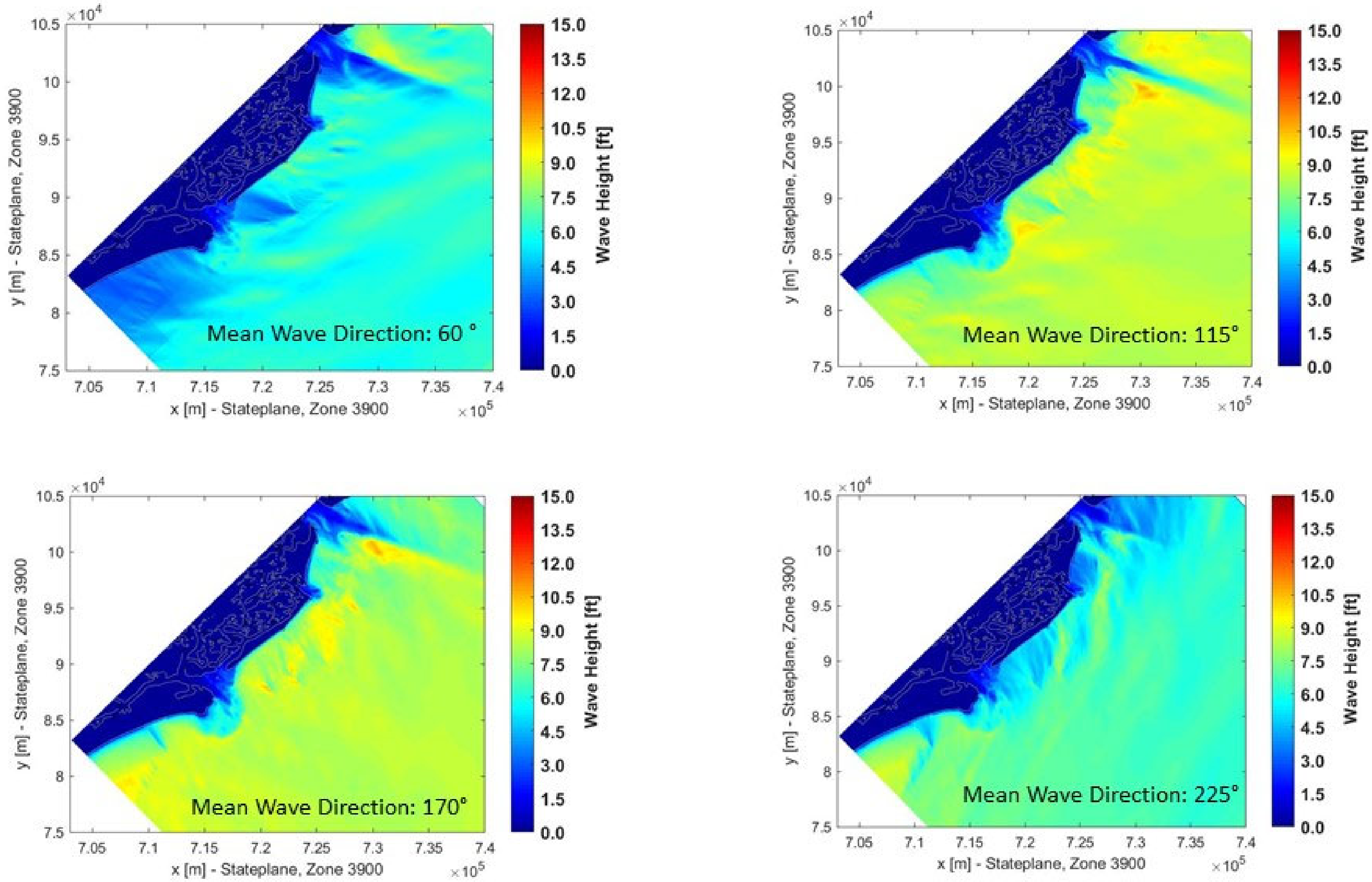
Mean Monthly Condition
Stono Inlet Borrow Area:



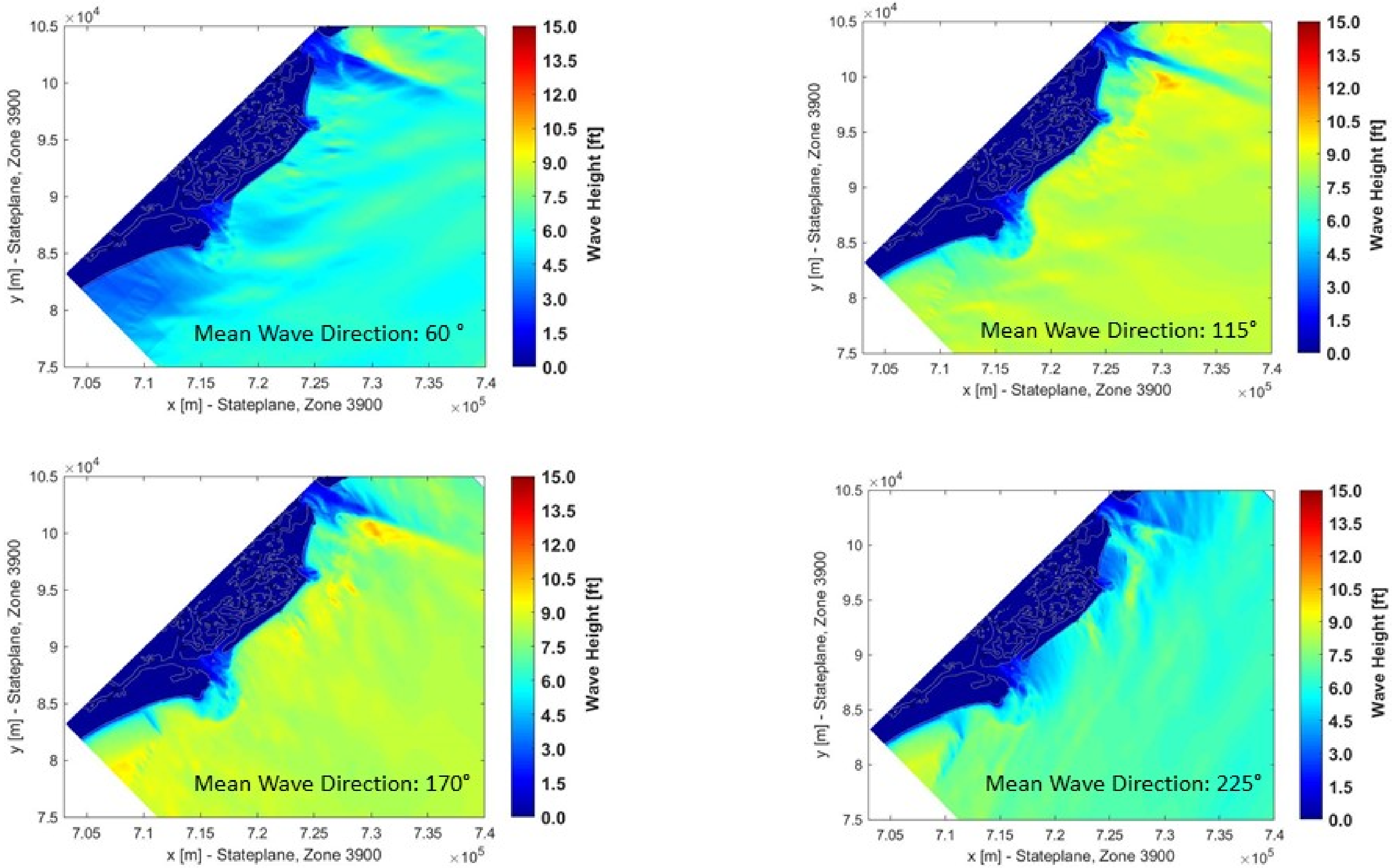
Max Monthly Condition
Base:



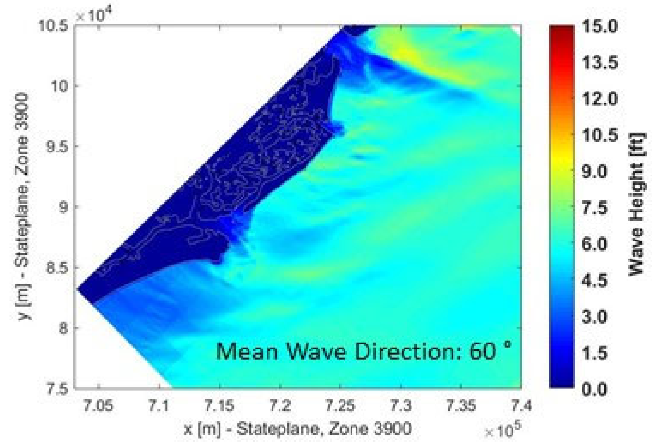
Max Monthly Condition
Central Folly Borrow Area:

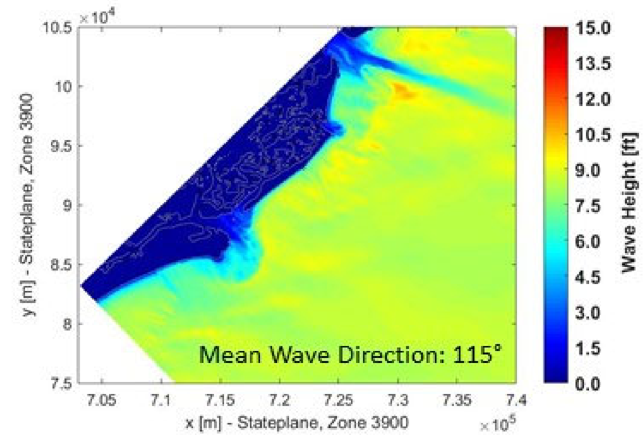


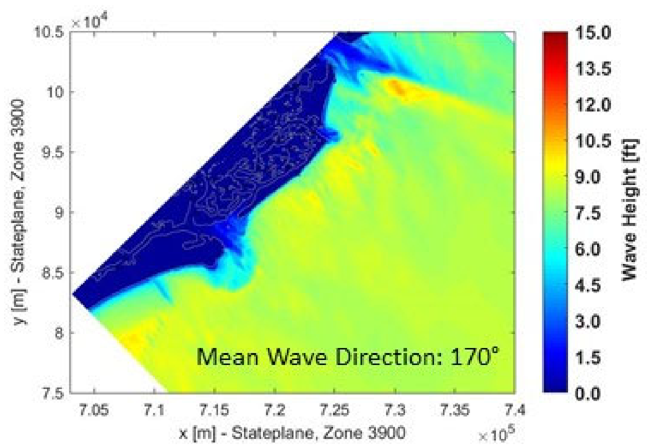
Max Monthly Condition
Lighthouse Inlet Borrow Area:

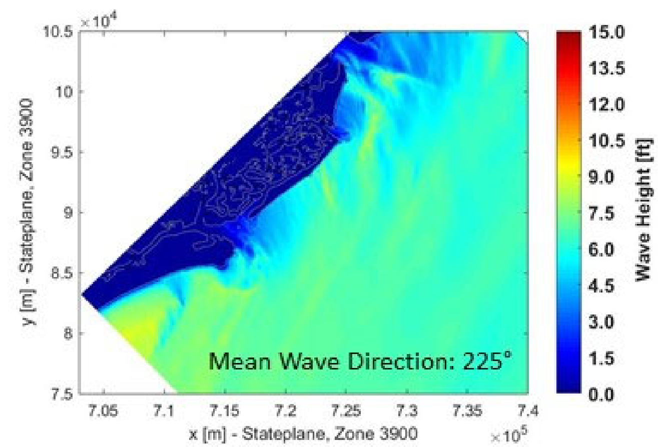


Max Monthly Condition
Seaward of State's Territory Borrow Area:

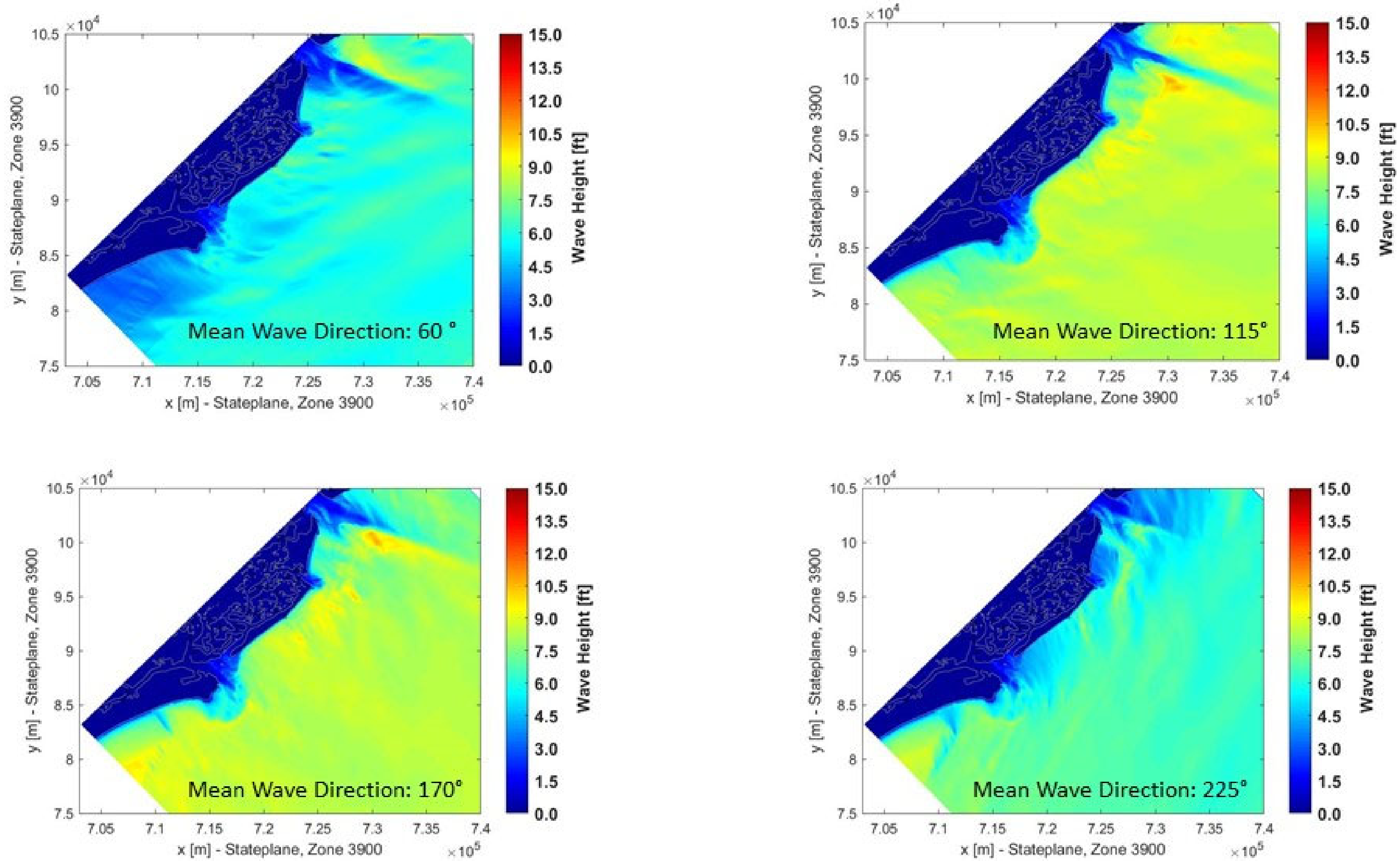




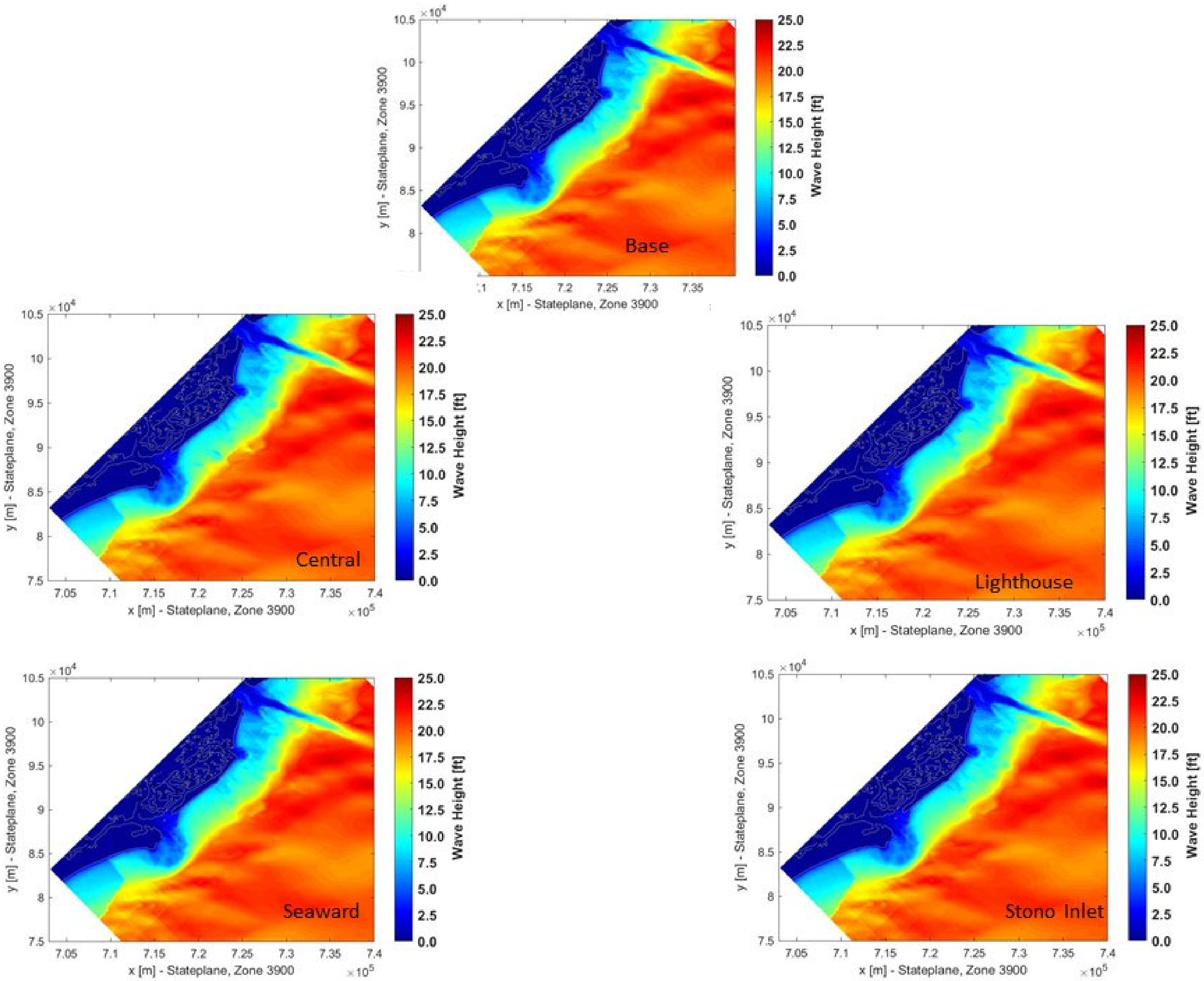




Max Monthly Condition
Stono Inlet Borrow Area:

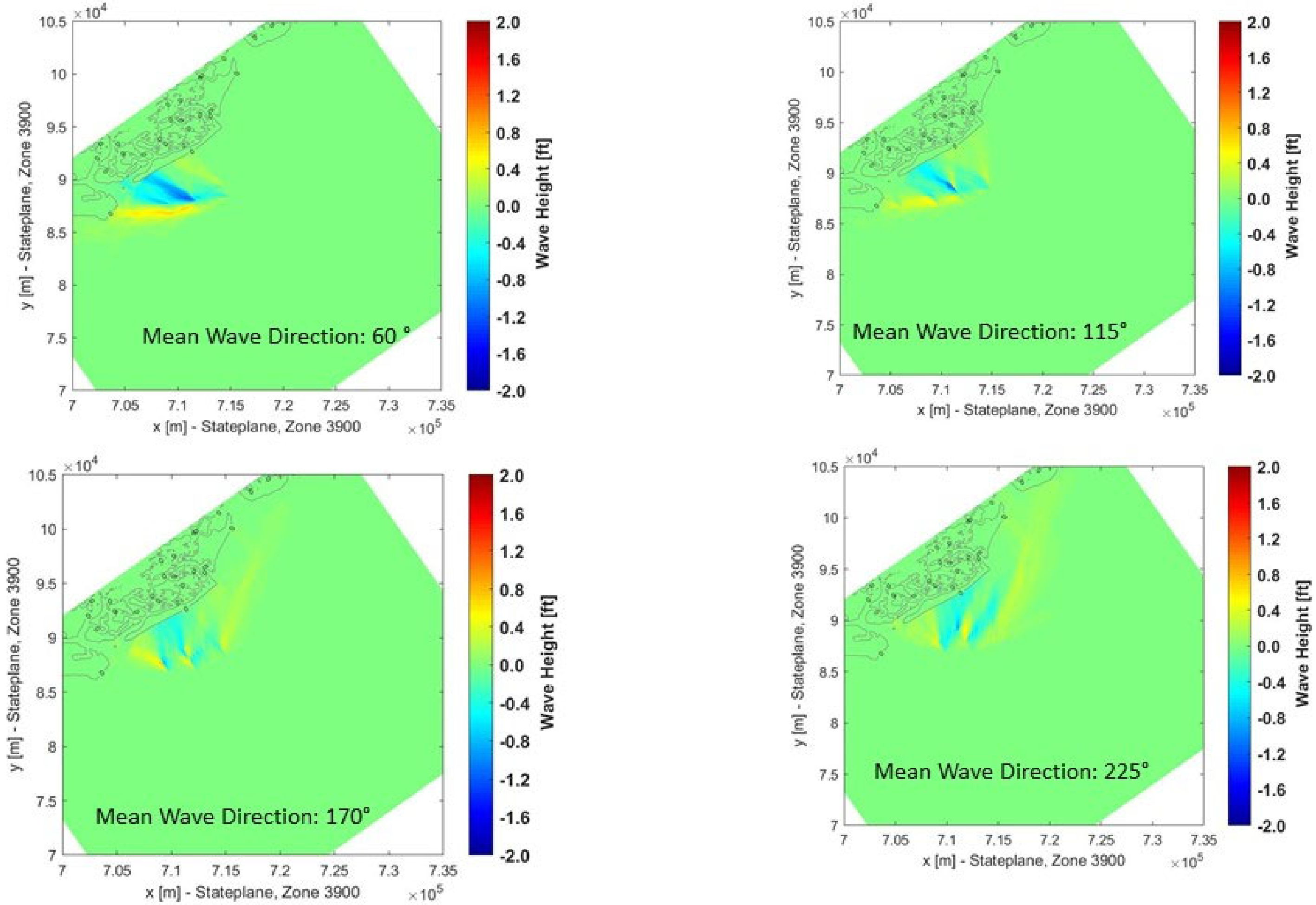


Extreme Event Condition:

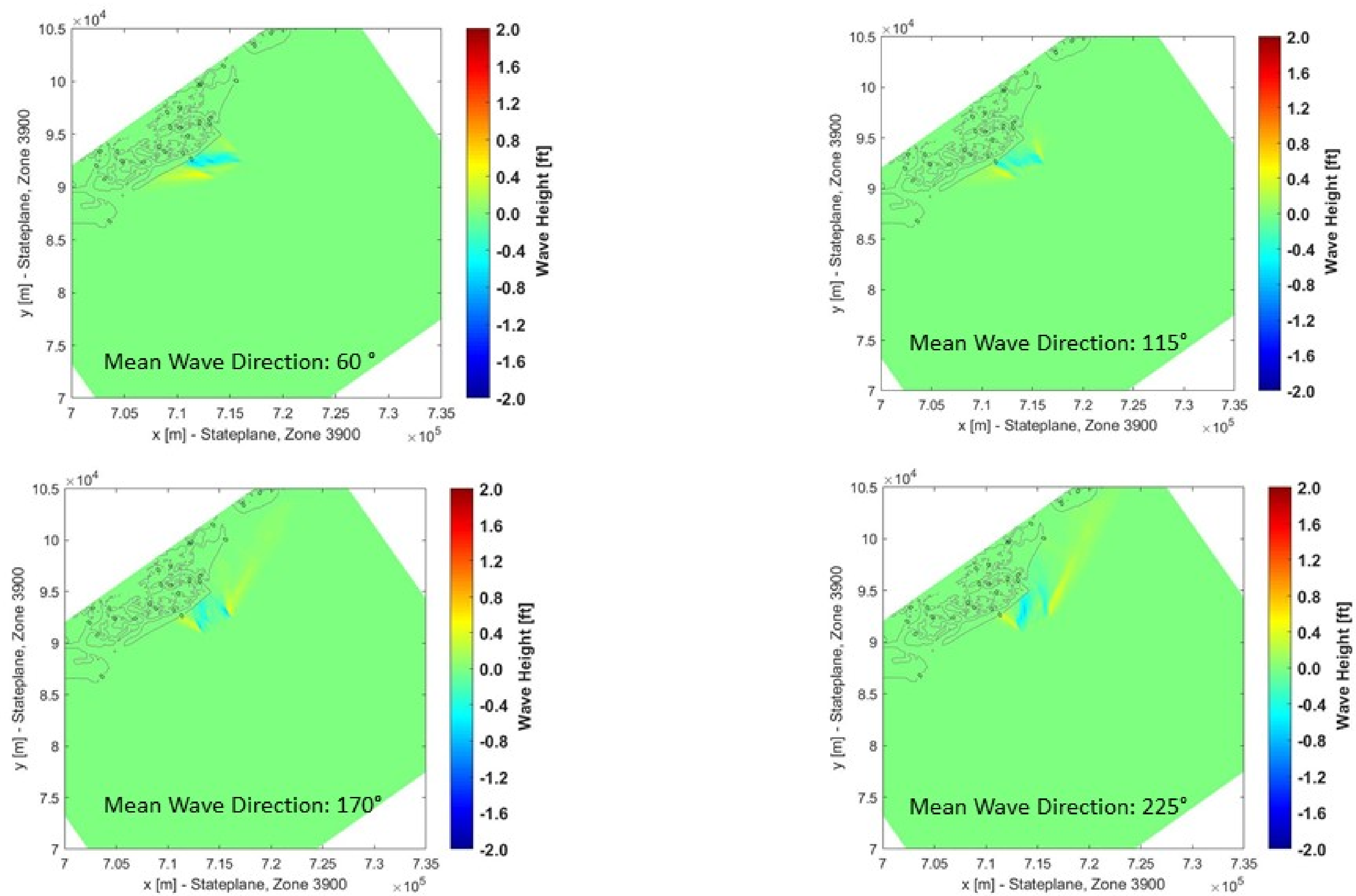


Appendix E—Difference Wave Height Plots

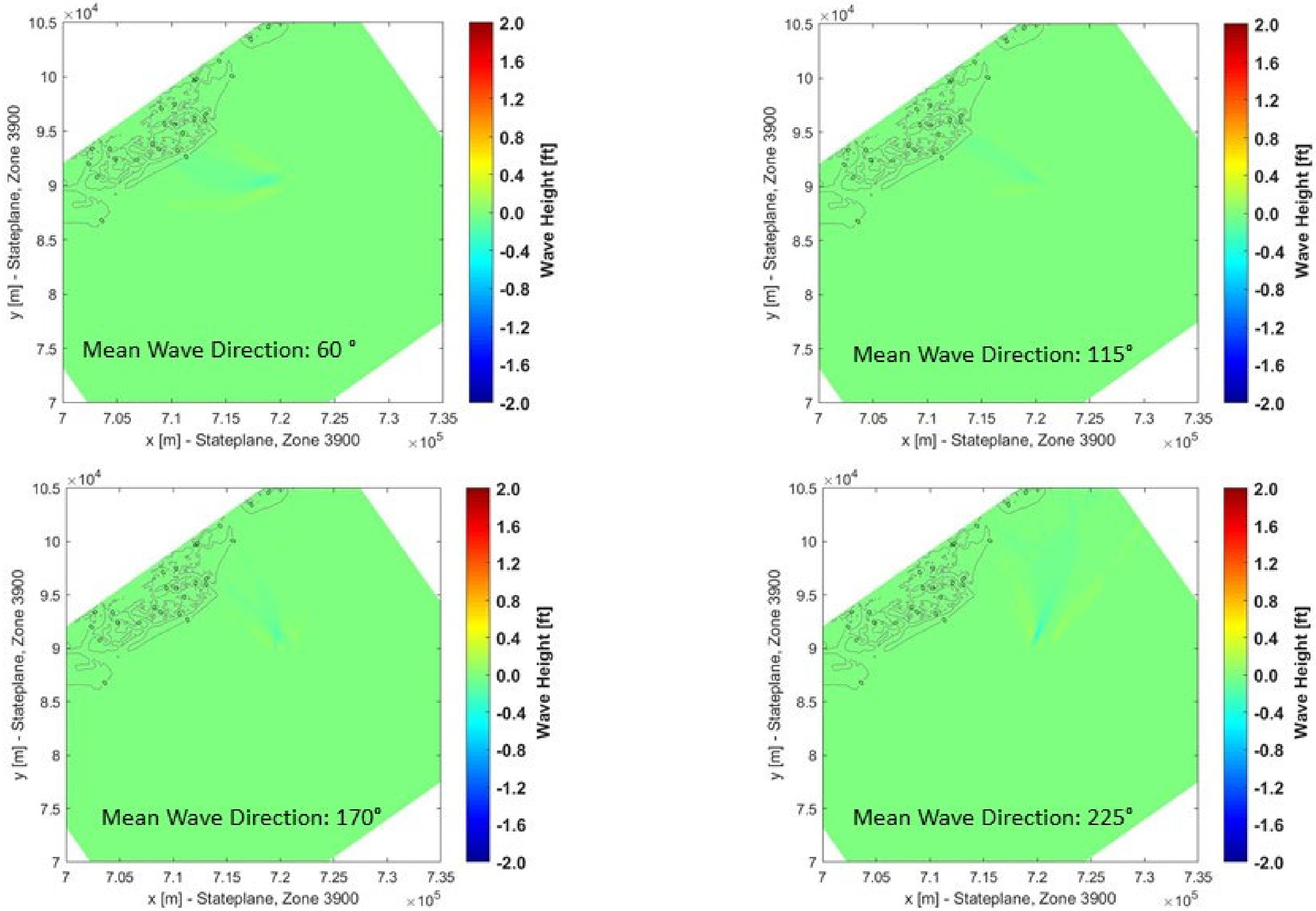
Mean Monthly Condition
Central Folly Borrow Area:



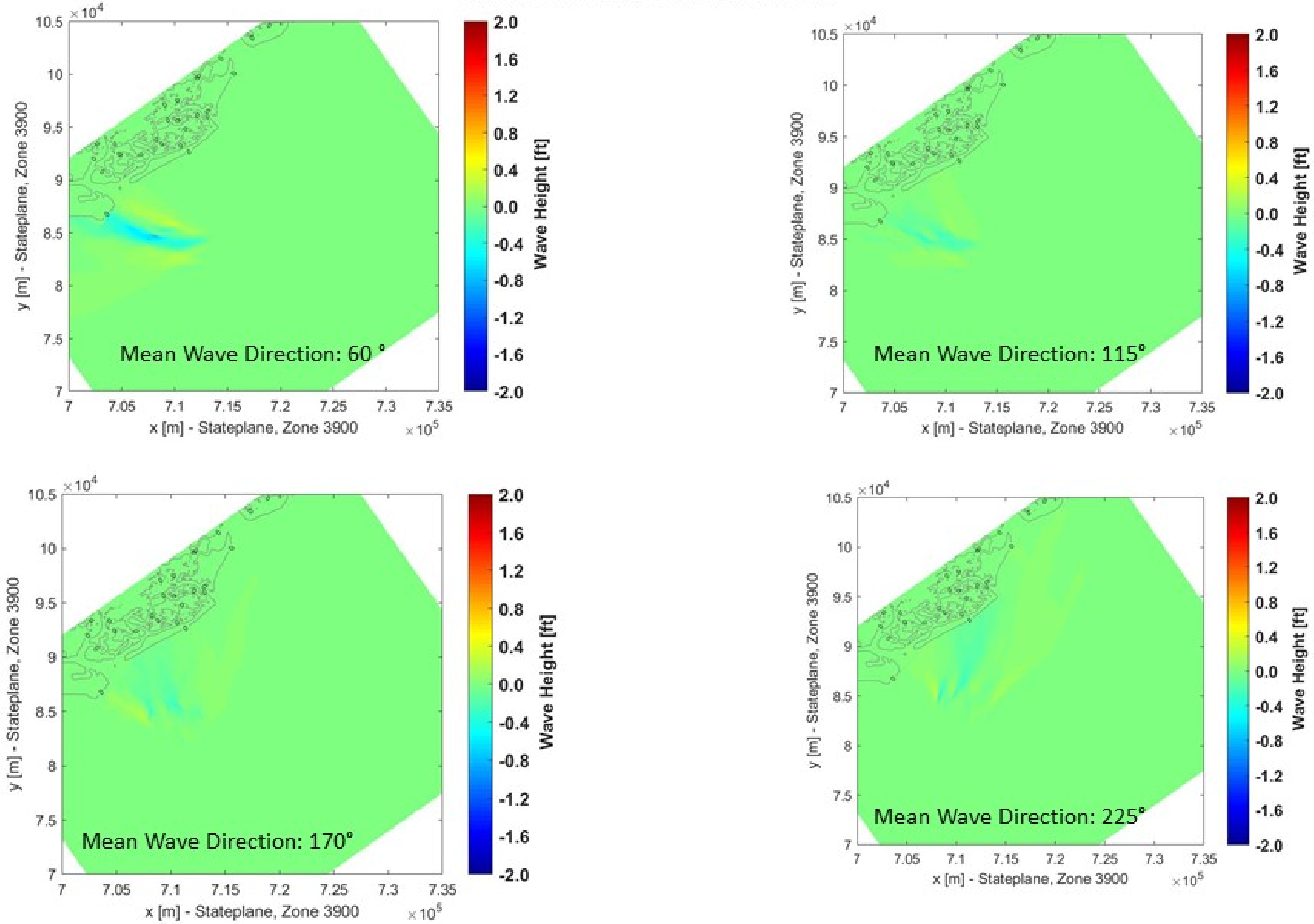
Mean Monthly Condition
Lighthouse Inlet Borrow Area:



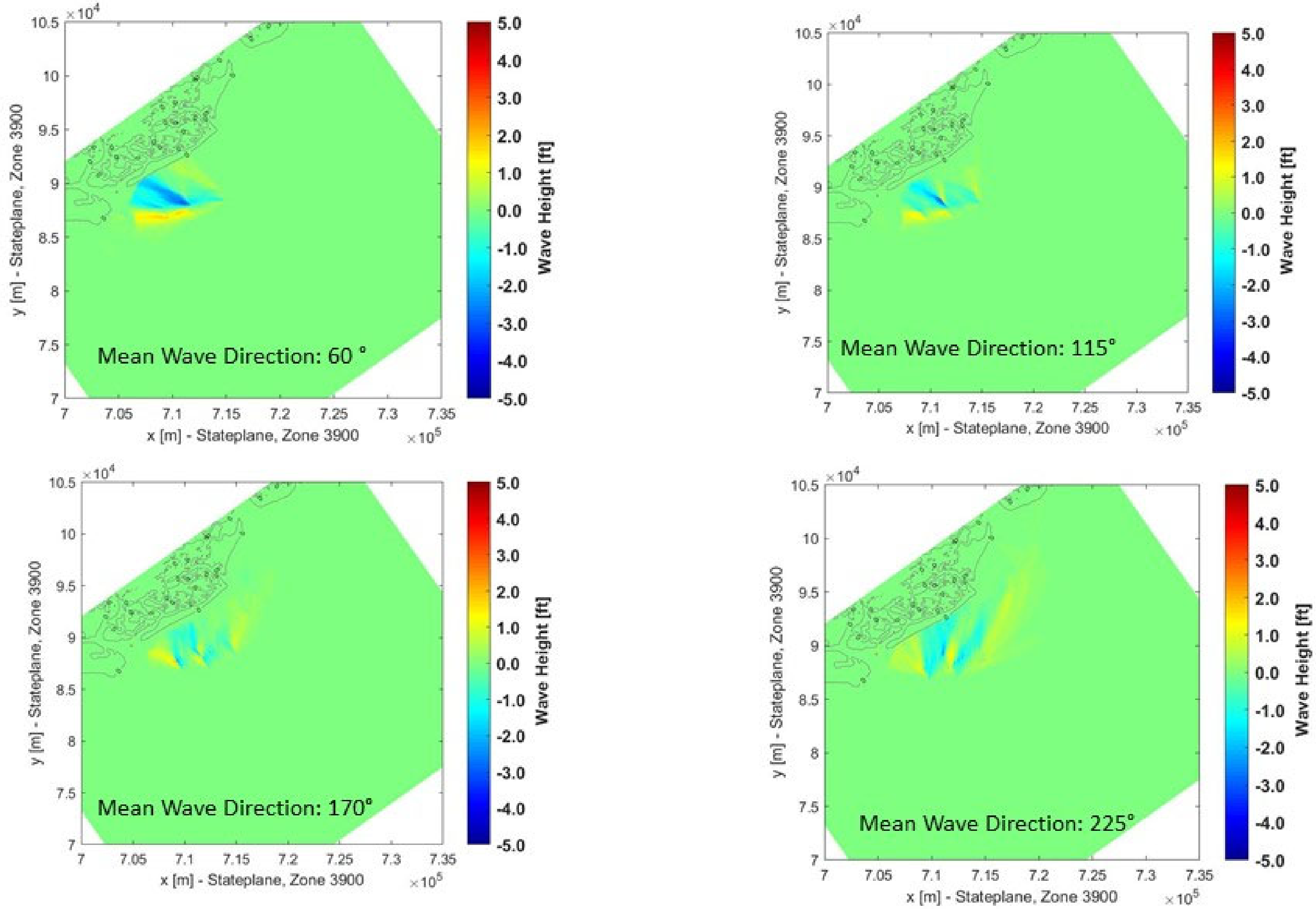
Mean Monthly Condition
Seaward of State's Territory Borrow Area:



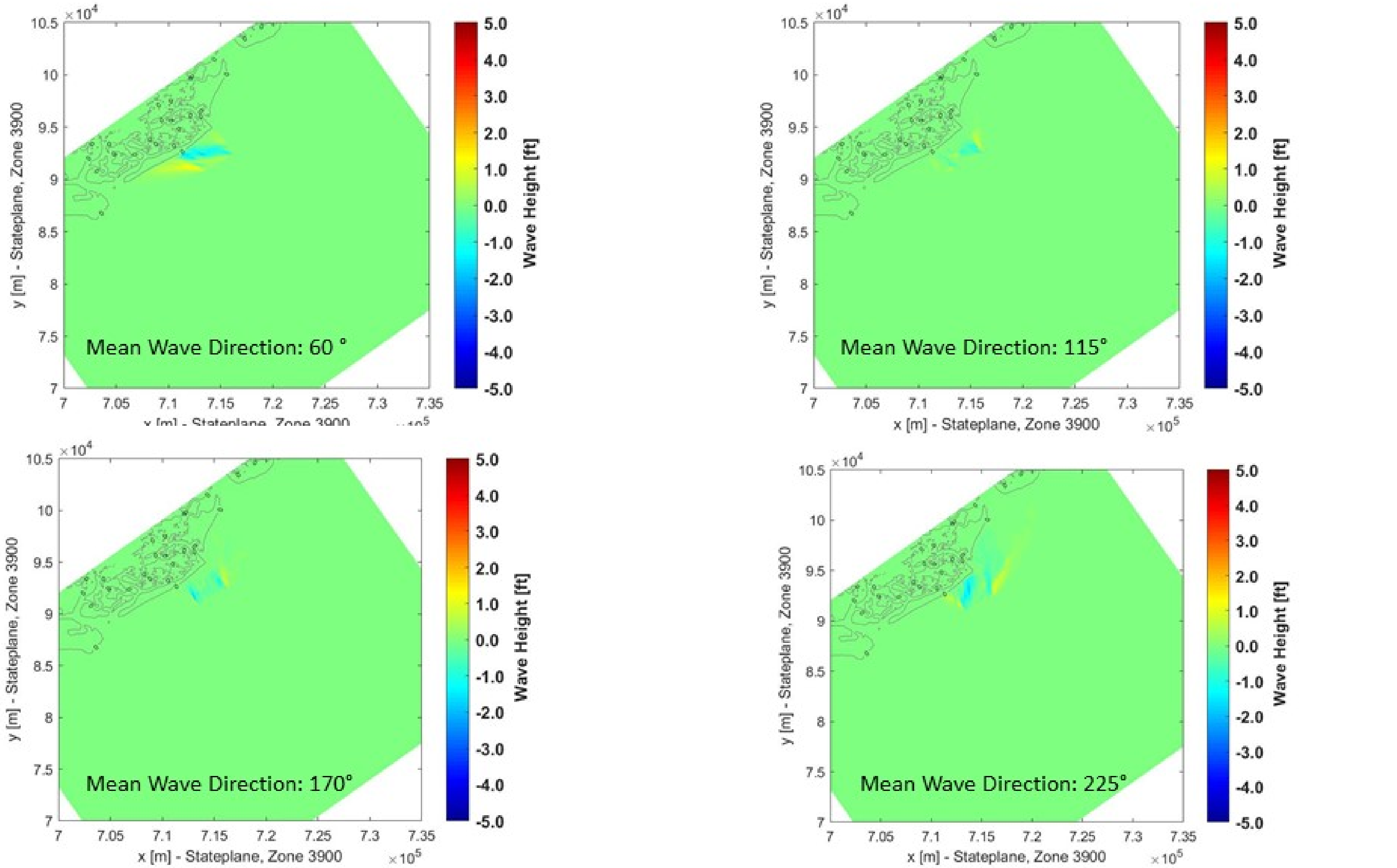
Mean Monthly Condition
Stono Inlet Borrow Area:



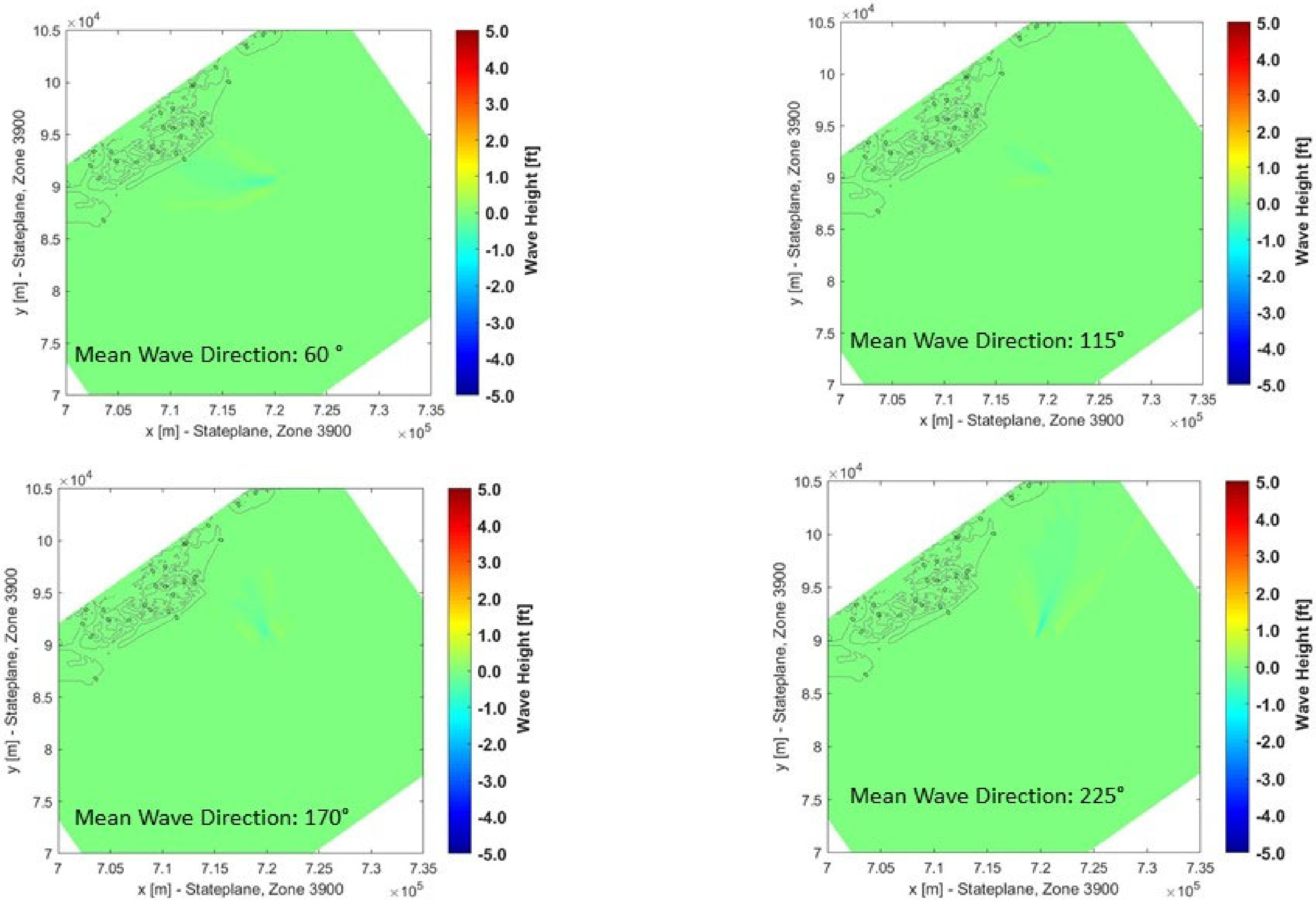
Max Monthly Condition
Central Folly Borrow Area:



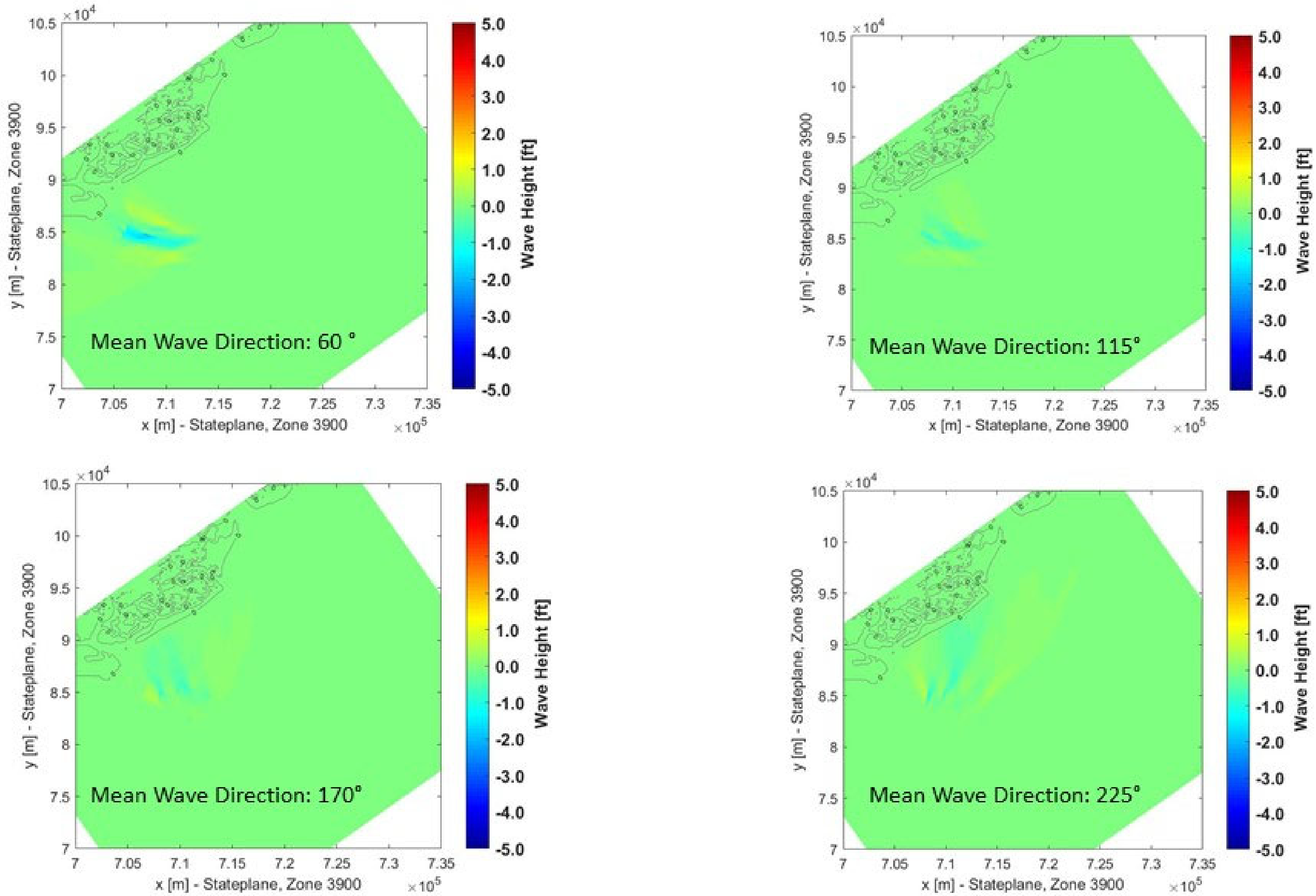
Max Monthly Condition
Lighthouse Inlet Borrow Area:



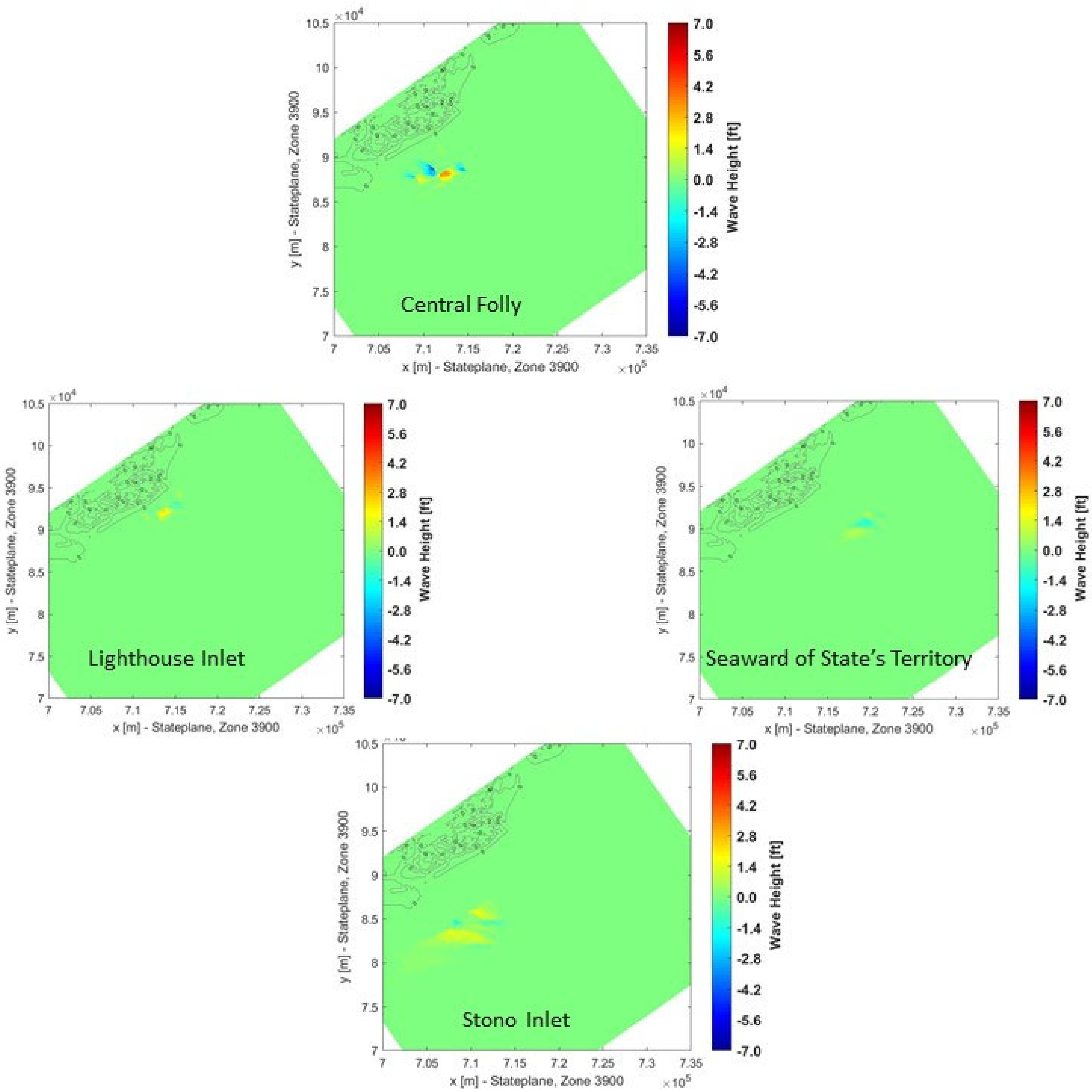
Max Monthly Condition
Seaward of State's Territory Borrow Area:



Max Monthly Condition
Stono Inlet Borrow Area:

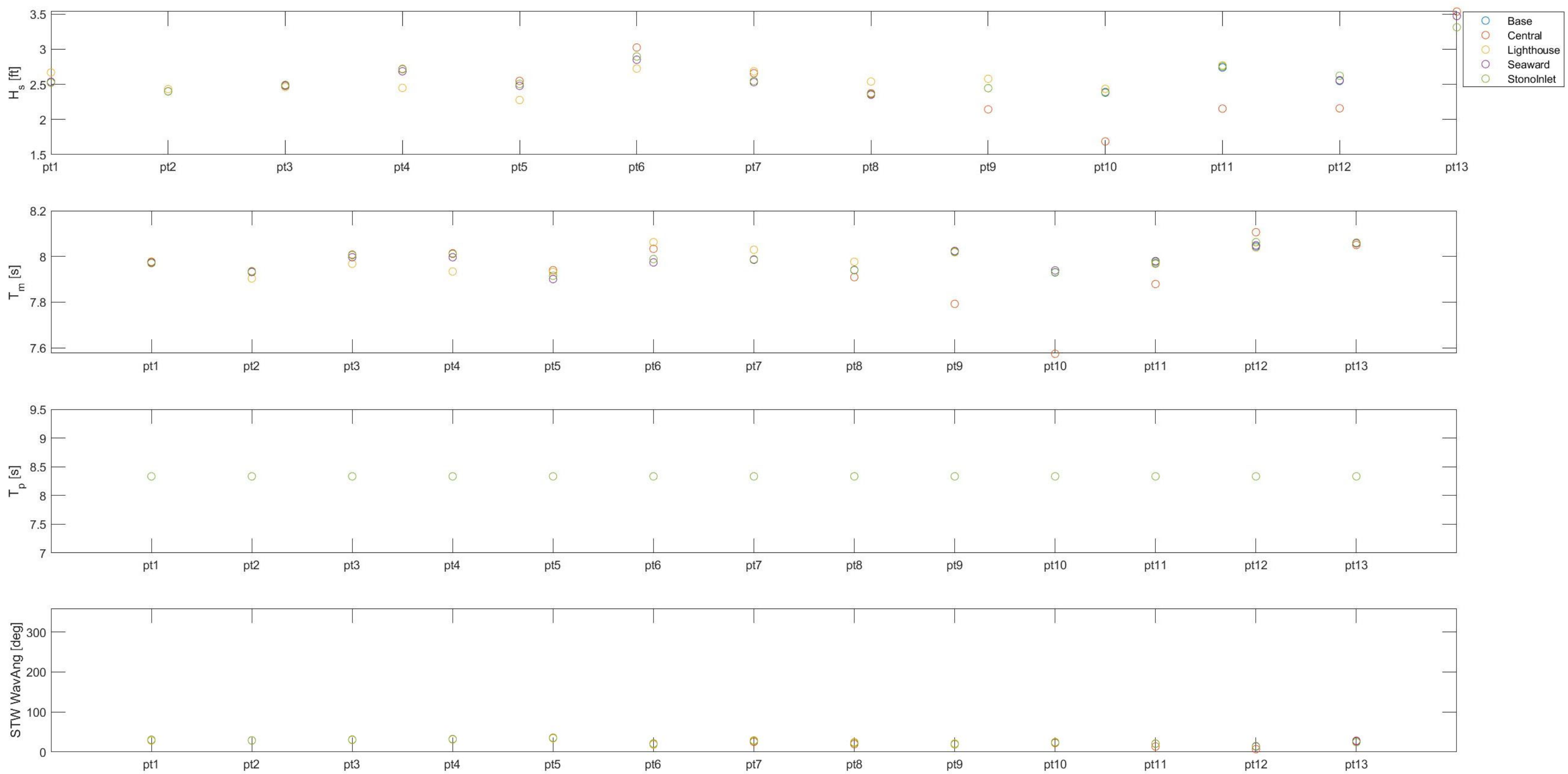


Extreme Event Condition

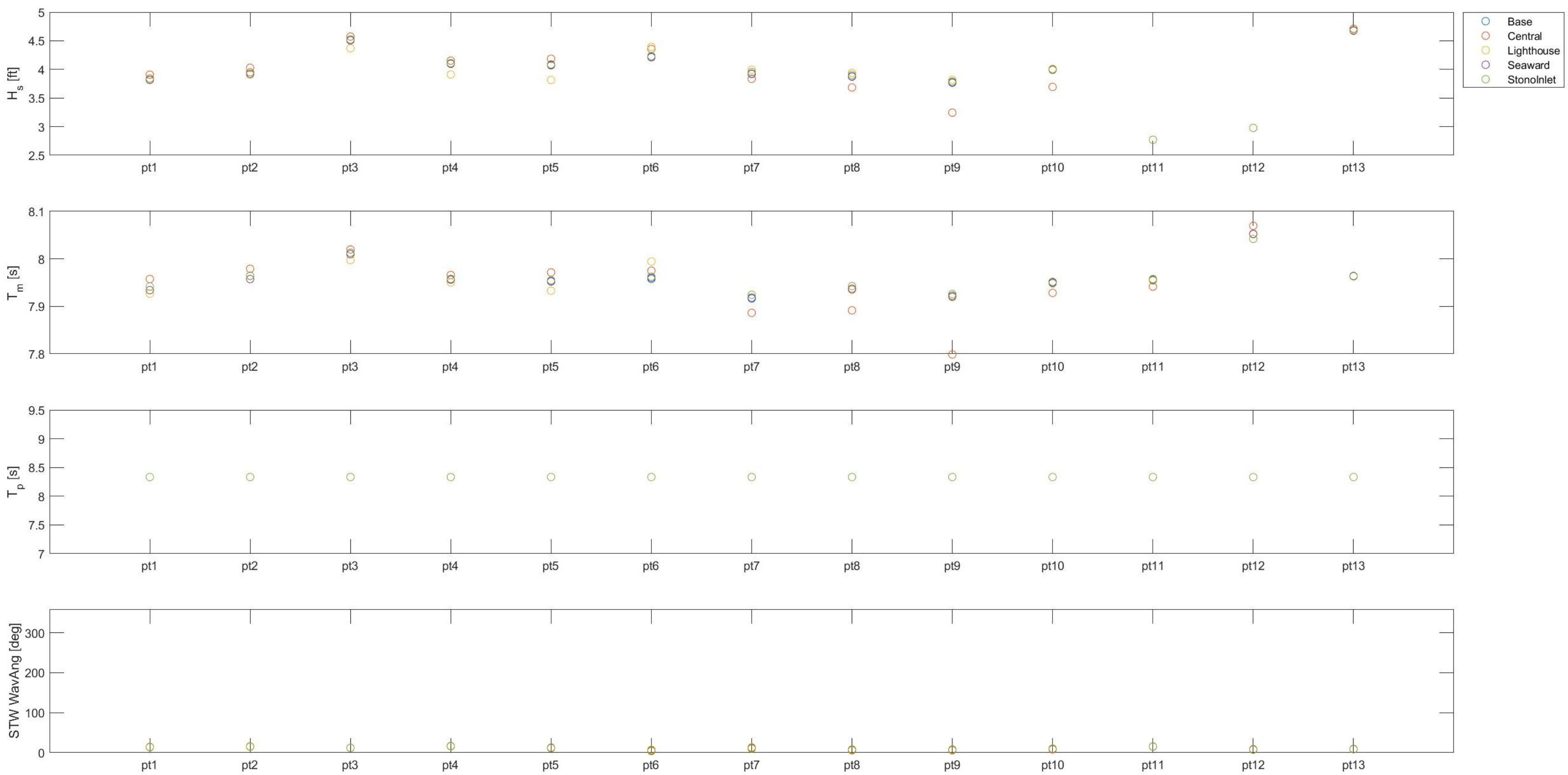


Appendix F—Save Point plots

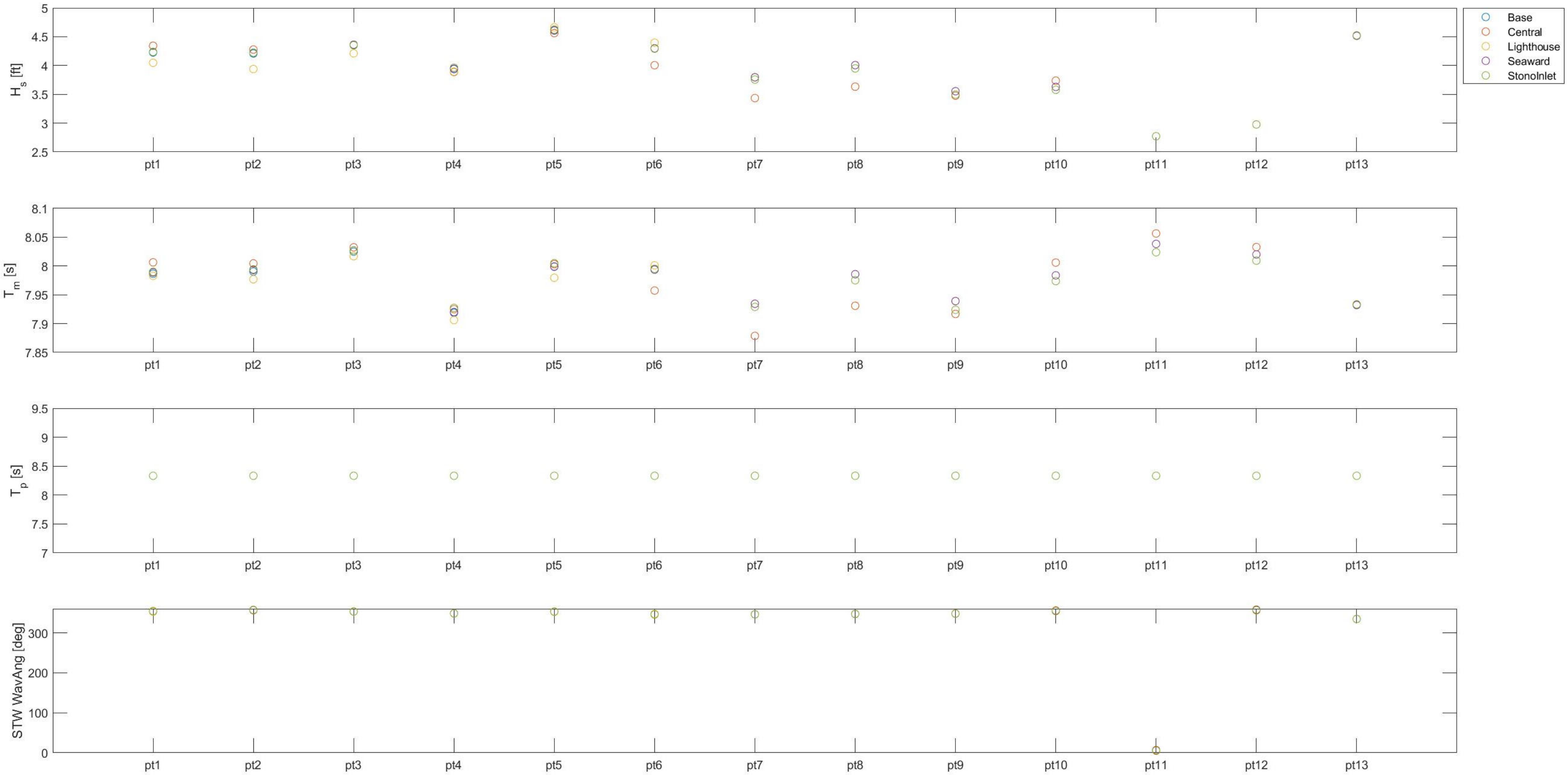
Mean Monthly- 60° Wave Direction



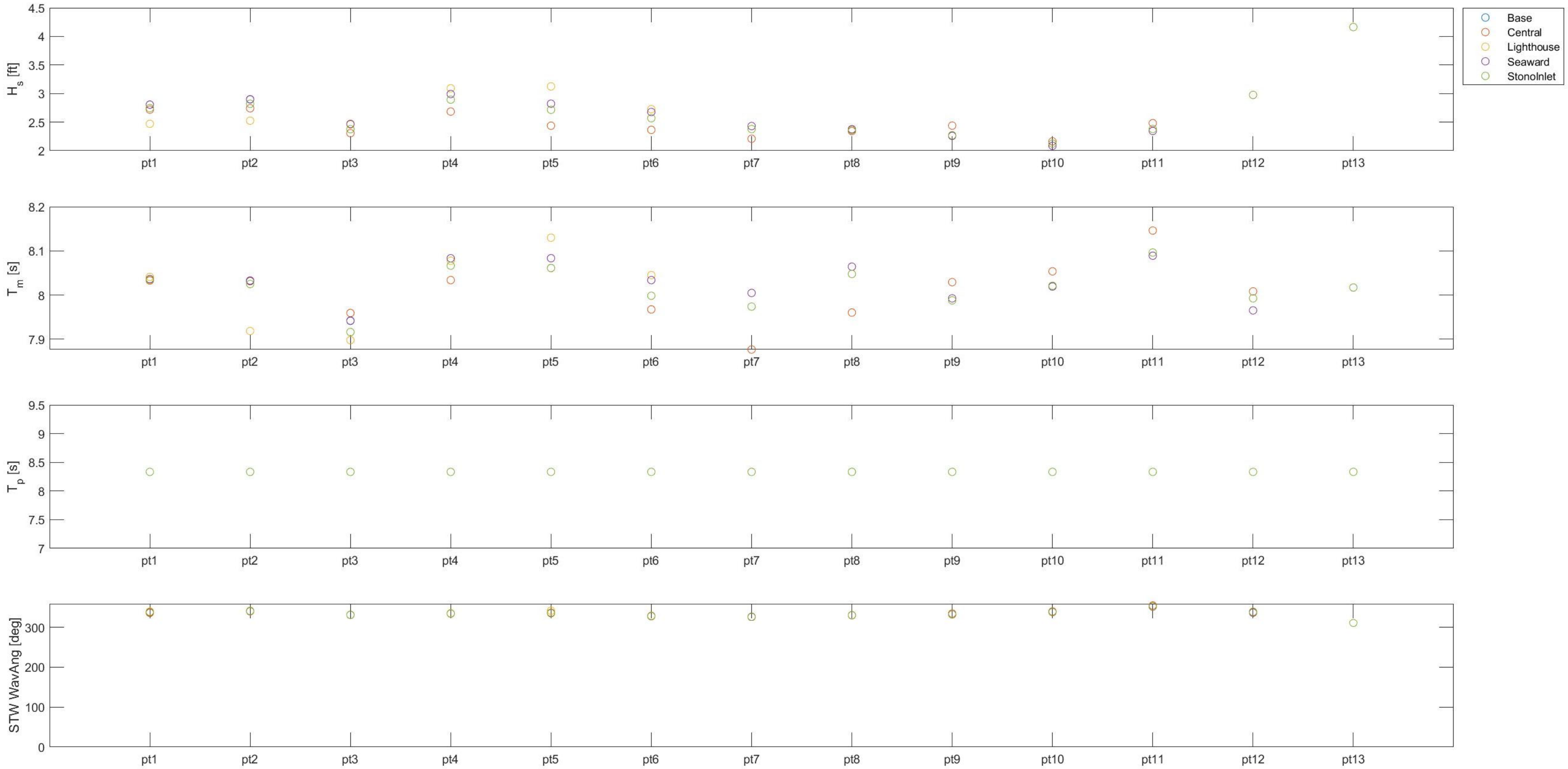
Mean Monthly- 115° Wave Direction



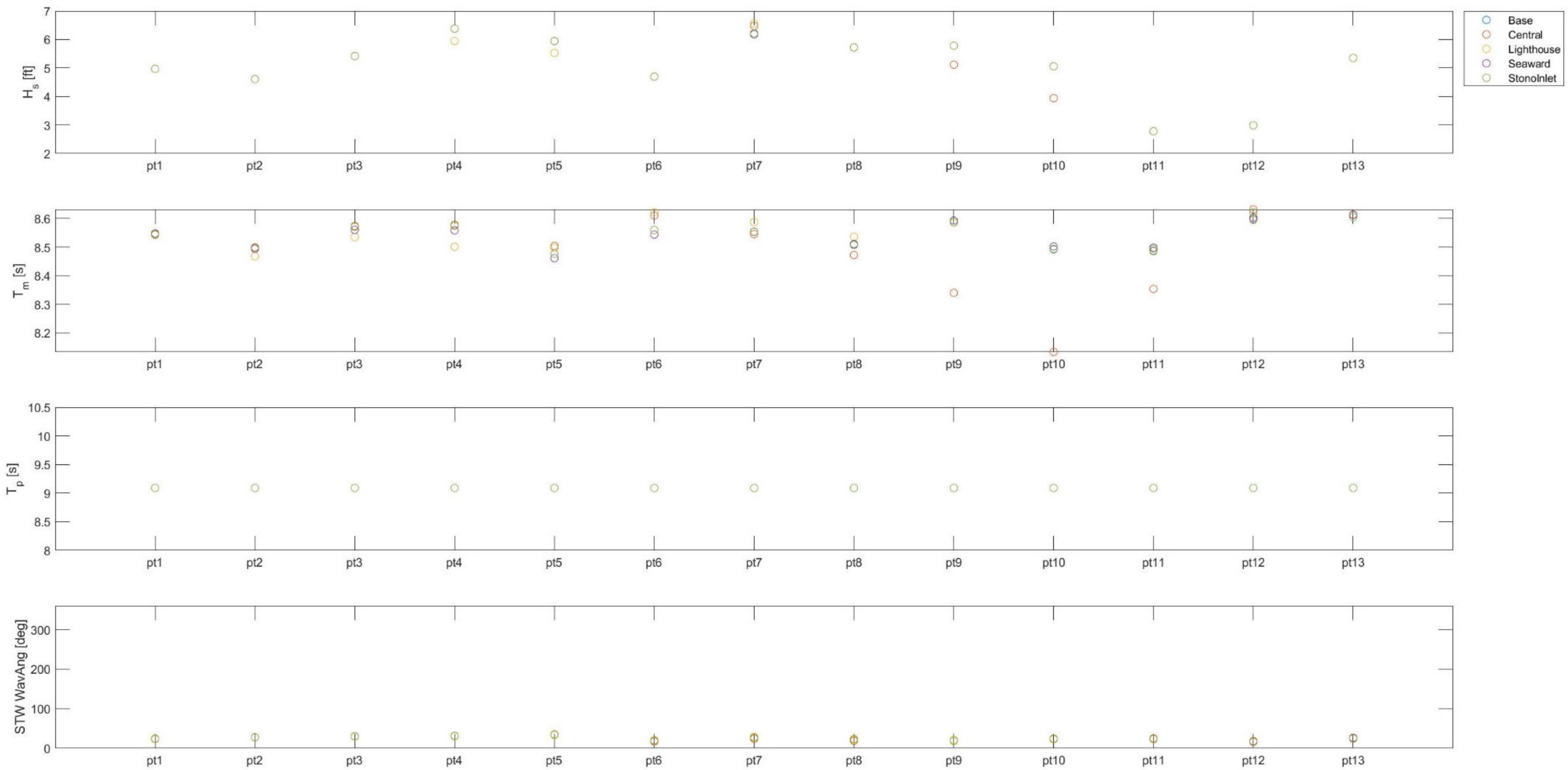
Mean Monthly- 170° Wave Direction



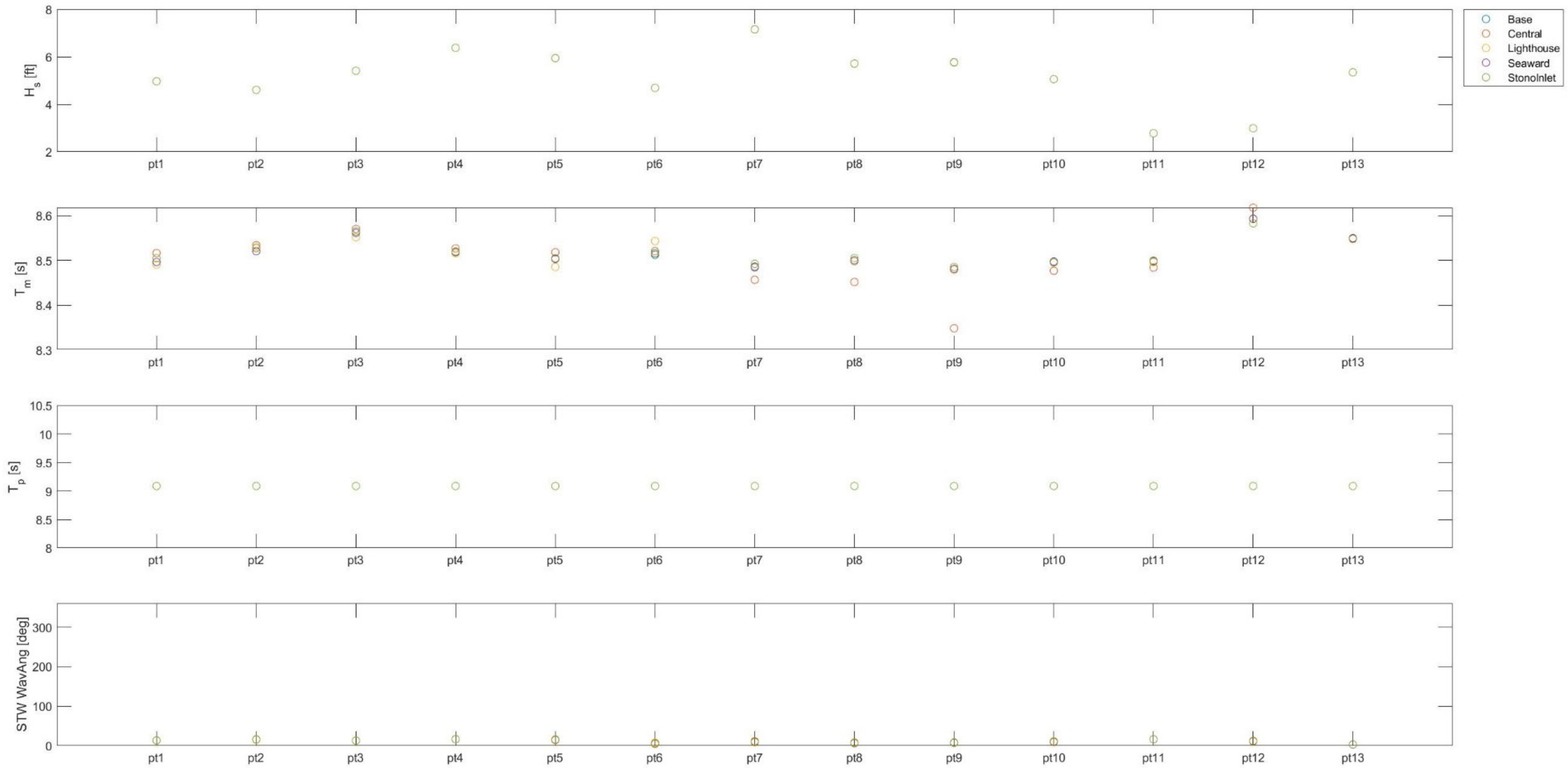
Mean Monthly- 225° Wave Direction



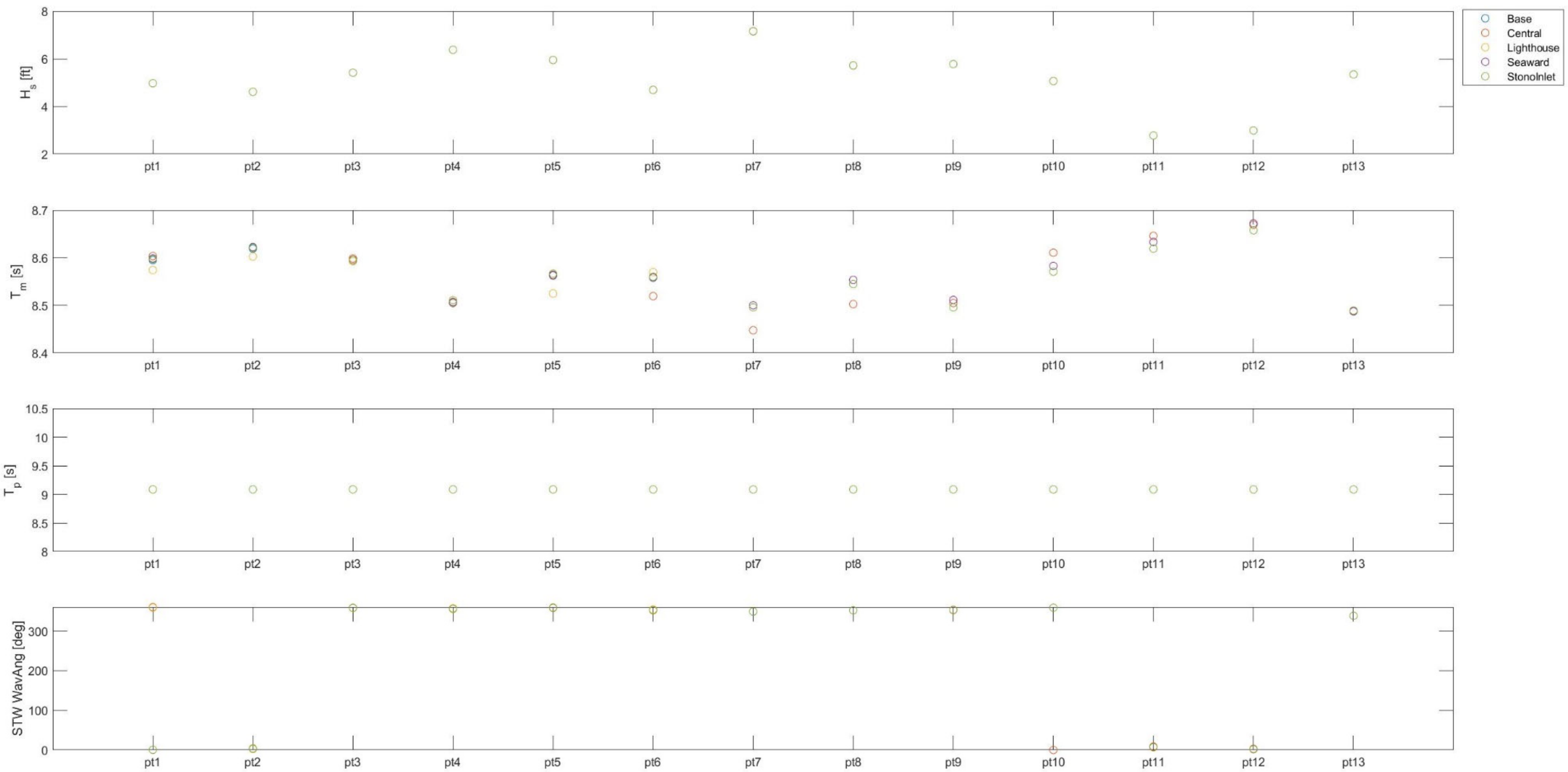
Max Monthly- 60° Wave Direction



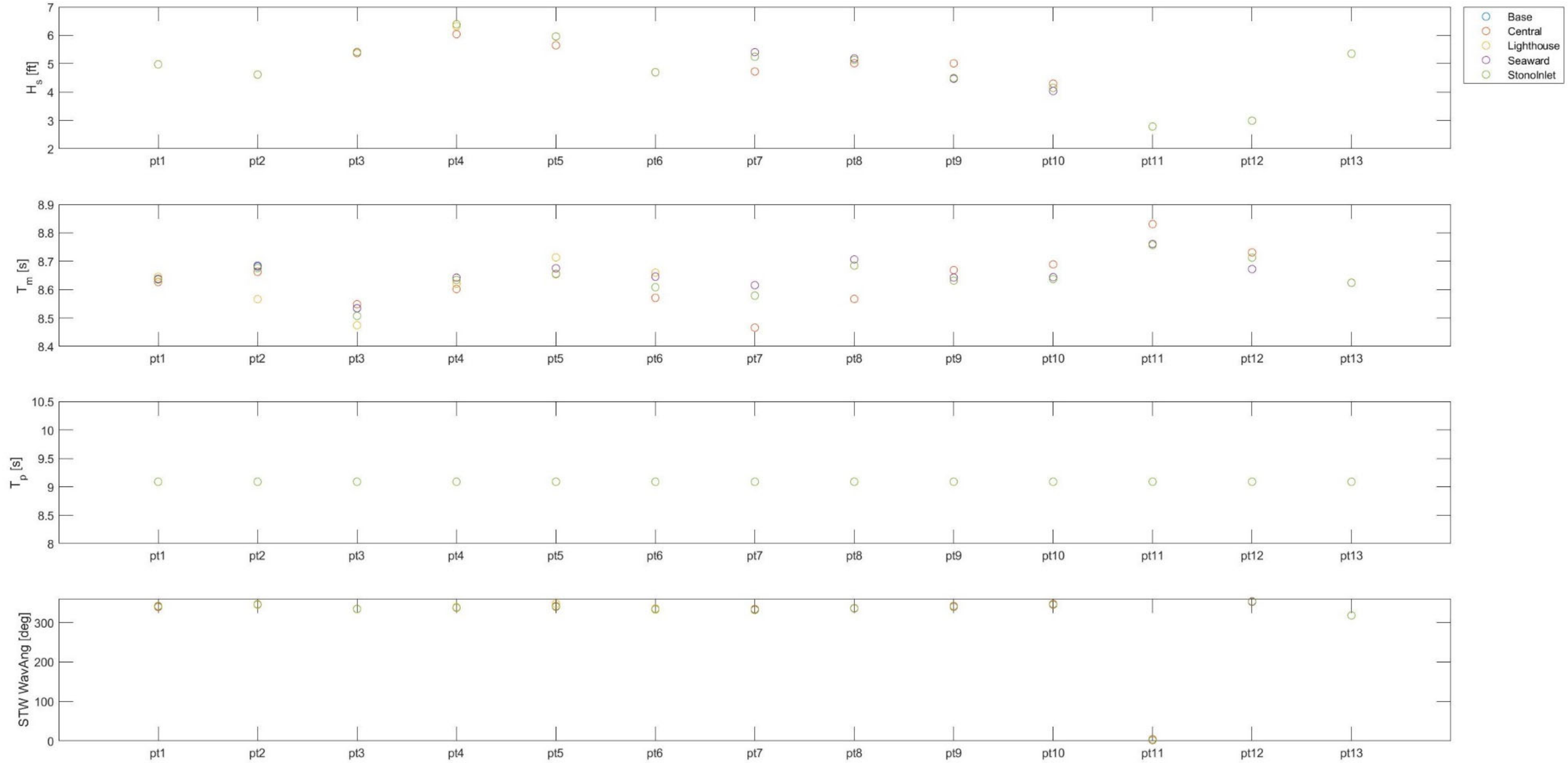
Max Monthly- 115° Wave Direction



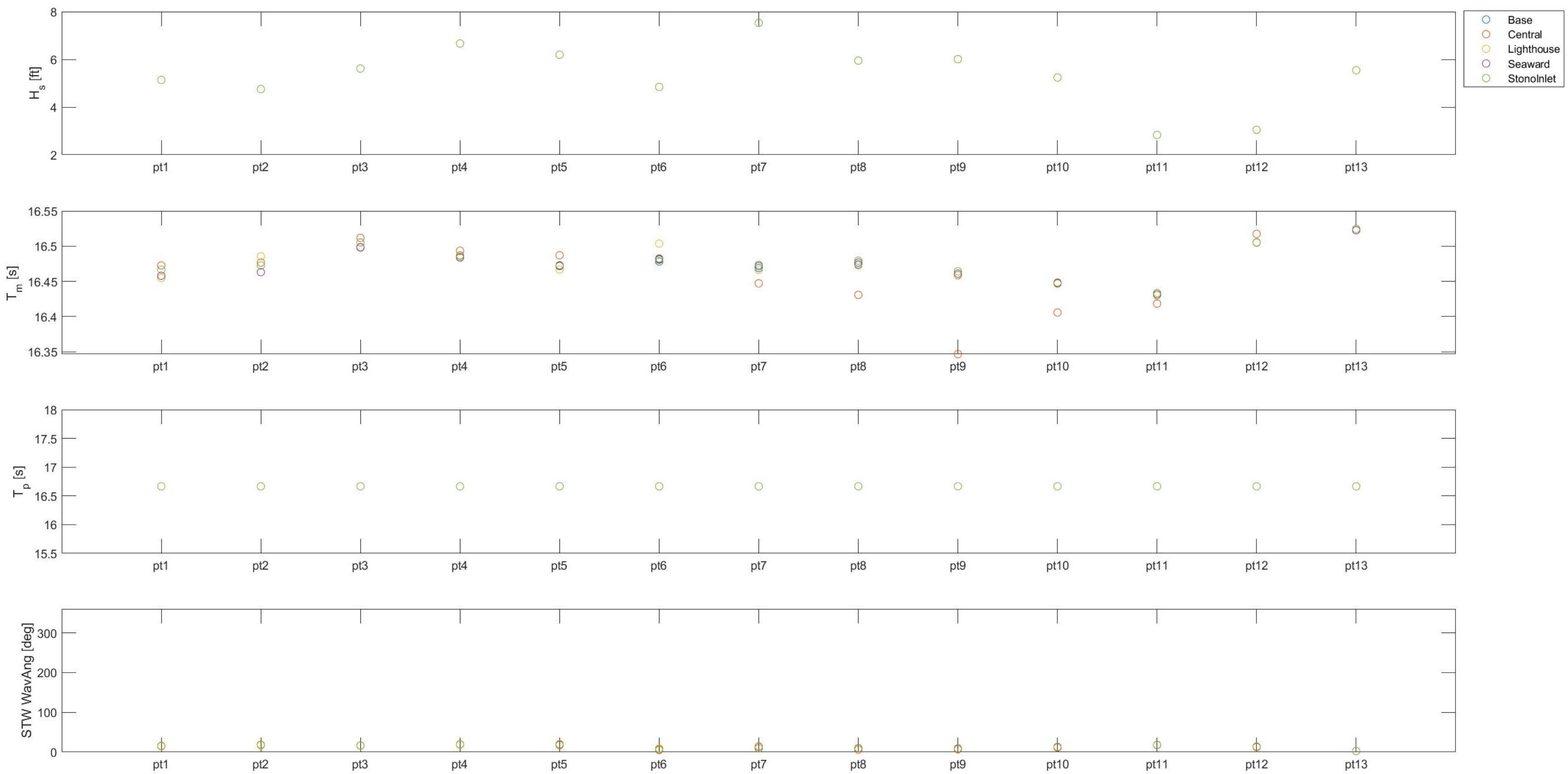
Max Monthly- 170° Wave Direction



Max Monthly- 225° Wave Direction



Extreme Even- 97° Wave Direction



Sub-Appendix E:

Folly Beach

Folly River Borrow Area Analysis

Folly River Borrow Area Refilling Rate.

A Folly River borrow area refilling rate was calculated for the May 2013 dredging.

Folly River surveys were collected in Q2 2014, Q4 2015, Q2 2017, and Q2 2018 on behalf of Charleston County Parks and Recreation Department (CCPRC). It is assumed that Q2 surveys were conducted in May and Q4 surveys were conducted in November. Material used in the 2103 Folly Beach nourishment is known to have been dredged from Folly River and was dredged in May of 2013, however no immediate post dredging survey was found so it is assumed that the 415,000 cy of material placed on the beach was removed immediately prior to the 2014 survey data in this analysis. The terminal groin was completed in June 2013 so this refilling rate is assumed to be appropriate for post groin construction refilling rates.

Using SMS 13.0.8, the borrow area was defined with 25 foot spaced points which was used to generate a Triangulated Irregular Network (TIN). Each of the surveys conducted was interpolated onto the TIN using inverse distance weighting creating a bottom surface depth. Figures 1-4 show the interpolated bottom surface depth TIN for surveys conducted from 2014 to 2018. In each image the full borrow area is shown in a magenta line, with the thicker black line overtop indicating the perimeter of the calculated TIN.

Figure 1: May 2014 bathymetric survey data with interpolated bottom surface TIN overlay.

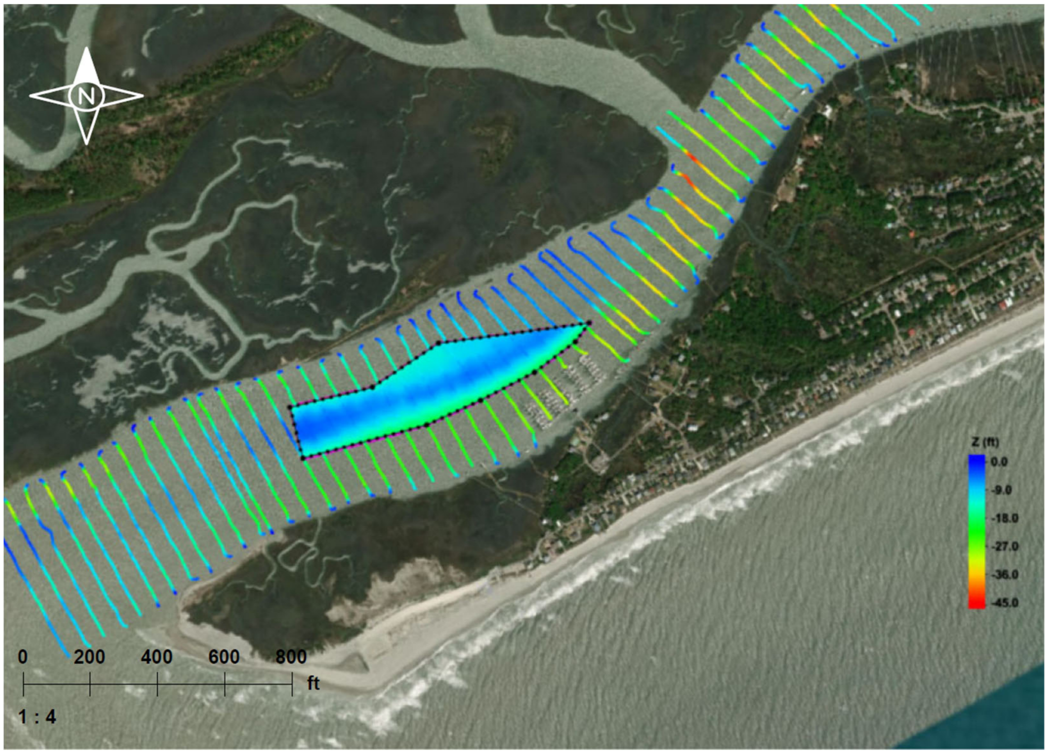


Figure 2: May 2015 bathymetric survey data with interpolated bottom surface TIN overlay.

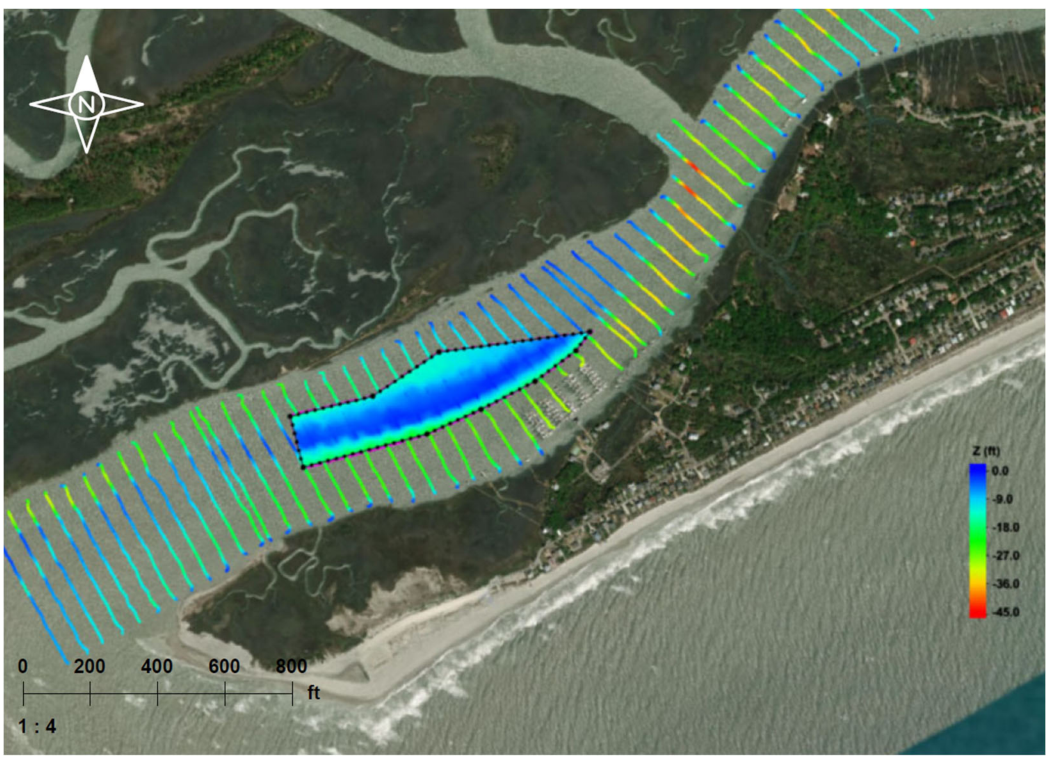


Figure 3: November 2017 bathymetric survey data with interpolated bottom surface TIN overlay.

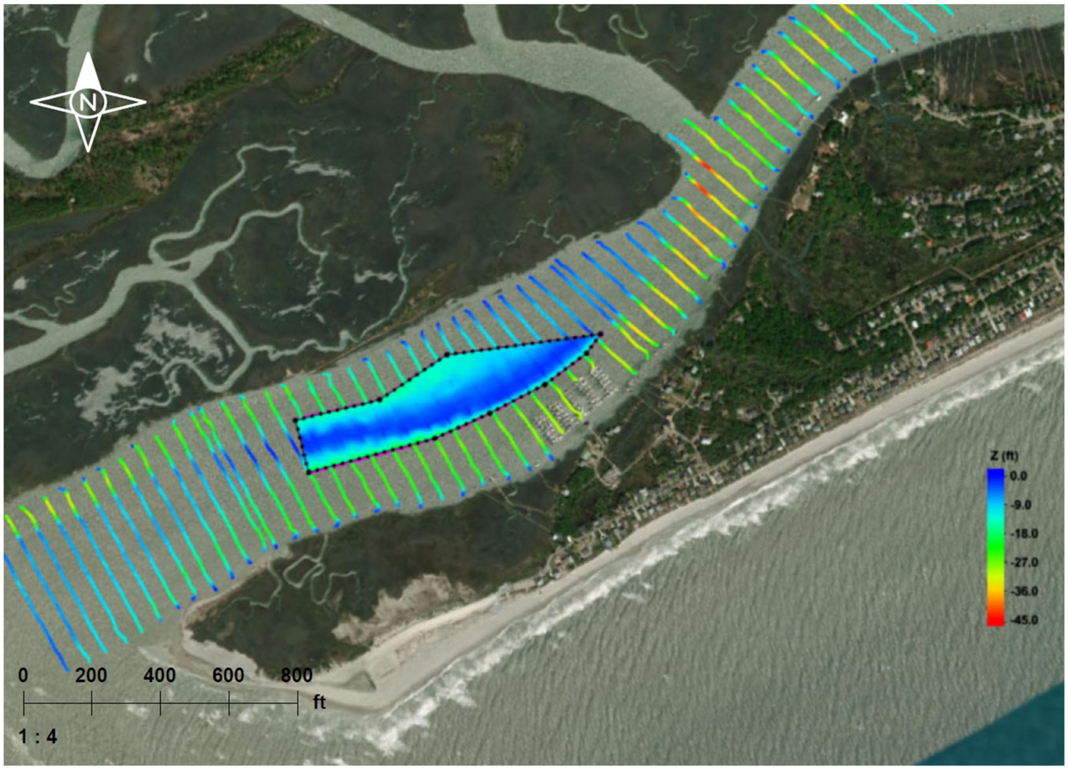


Figure 4: May 2018 bathymetric survey data with interpolated bottom surface TIN overlay.

Changes in the bottom surface elevation were compared from survey to survey to calculate volume change. Positive changes in water volume equate to sediment lost while negative changes in water volume equate to sediment gained. For each period between surveys the sediment volume deposited, sediment volume lost, net sediment volume change, and percent of original sediment loss replaced were calculated, shown in Table 1. Over the four year period the average refilling rate of this portion of the borrow area, assuming all material was taken from this area, was calculated to be 12.25%.

Table 1: Volume change analysis measuring sediment deposition within the inshore Folly River borrow area following dredging activities in May 2013.

	Time Periods Between Bathymetric Surveys				
	2014	2014-2015	2015-2017	2017-2018	Totals
Δt [yr]		1	2.5	0.5	4
Volume Deposited [cy]		116,698	106,538	51,350	274,586
Volume Lost [cy]		-16,478	-18,720	-35,959	-71,157
Net ΔV Calculated [cy]	-415,000**	100,220	87,818	15,391	203,429
% loss replaced		24.15%	21.16%	3.71%	49.02%

** Represents estimated volume of sediment removed based on sediment placed on the beach.

Volume of Sediment Still Missing [cy]:	211,571
Average % Original Loss Replaced/Year:	12.25%
Estimated Total Years to Replace:	8.2

Sub-Appendix F:

Folly Beach

Sediment Transport Modeling at Stono Inlet and Adjacent Beaches

(NOT FINAL PUBLISHED ERDC VERSION)

ERDC/CHL TR-21-XX

1Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Coastal Inlets Research Program

Sediment Transport Modeling at Stono Inlet and Adjacent Beaches, South Carolina

Honghai Li, Grace M. Maze, Kevin B. Conner, and John M. Hazelton

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Approved for public release; distribution is unlimited.

Sediment Transport Modeling at Stono Inlet and Adjacent Beaches, South Carolina

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Abstract

This report documents a numerical modeling investigation for dredged material from nearshore borrow areas and placed on Folly Beach adjacent to Stono Inlet, South Carolina. Historical and newly collected wave and hydrodynamic data around the inlet were assembled and analyzed. The datasets were used to calibrate and validate a coastal wave, hydrodynamic and sediment transport model, the Coastal Modeling System (CMS). Sediment transport and morphology changes within and around the immediate vicinity of the Stono Inlet estuarine system, including sand borrow areas and nearshore Folly Beach area, were evaluated. Results of model simulations show that sand removal in the borrow areas increases material backfilling, which is more significant in the nearshore than the offshore borrow areas. In the nearshore Folly Beach area, the dominant flow and sediment transport directions are from the northeast to the southwest. Net sediment gain occurs in the central and southwest sections while net sediment loss occurs in the northeast section of Folly Island. A storm and a one-year simulation developed for the study produce a similar pattern of morphology changes, and erosion and deposition around the borrow areas and the nearshore Folly Beach area.

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Figures and Tables

Figures

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List of Acronyms

2D	Two-Dimensional
3D	Three-Dimensional
ADCP	Acoustic Doppler Current Profiler
AWAC	Acoustic Wave and Current Profiler
CHL	Coastal & Hydraulics Laboratory
CIRP	Coastal Inlets Research Program
CMS	Coastal Modeling System
DEM	Digital Elevation Model
ERDC	Engineering Research and Development Center
ETS	Environmental Tracing Systems
GMT	Greenwich Mean Time
HW	High Water
HQUSACE	Headquarters U.S. Army Corps of Engineers
JALBTCX	Joint Airborne Lidar Bathymetry Technical Center of Expertise
LST	Local Standard Time
LW	Low Water
MLLW	Mean Lower Low Water
MSL	Mean Sea Level
NDBC	National Data Buoy Center
NET	Non-Equilibrium Sediment Transport
NGDC	National Geophysical Data Center
NOAA	National Oceanic and Atmospheric Administration
NRMSE	Normalized Root-Mean-Square Error
NS	Nearshore
ODMDS	Ocean Dredged Material Disposal Site
OS	Offshore
QA	Quality Assurance
QC	Quality Control
R	Correlation Coefficient
RM	River Mile
RMSE	Root-Mean-Square Error
SAW	USACE Wilmington District
SMS	Surface-water Modeling System
USACE	U.S. Army Corps of Engineers
WSE	Water Surface Elevation

Preface

A numerical modeling study and a field data collection program were conducted to investigate sediment transport around Stono Inlet and Folly Beach, South Carolina. This study was performed by the Coastal and Hydraulics Laboratory (CHL) of the US Army Corps Engineer Research and Development Center (ERDC) at the request of the U.S. Army Engineer District, Wilmington (SAW). The study started with the preparation of historical data assembly followed by numerical model development and modeling simulations of waves, currents, and sediment transport. The modeling domain includes the entire Stono Inlet estuarine system and the offshore area.

The Coastal Modeling System (CMS), an integrated coastal modeling system composed of a spectral wave model and a 2-D depth-averaged hydrodynamic and sediment transport model was applied to calculate sediment movement under the combined influence of waves and current. The field data collection program was completed by Field Data Collection & Analysis Branch of CHL. This report provides the details of these tasks and the results and major findings of the study.

The Coastal Inlets Research Program (CIRP) conducted this study with the funding from the Wilmington District. The CIRP is administered for Headquarters by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS, under the Navigation Program of HQUSACE. Michael E. Ott is HQUSACE Navigation Business Line Manager overseeing the CIRP. Charles E. Wiggins, CHL, is the ERDC Technical Director for Navigation. Dr. Tanya M. Beck, CHL, is the CIRP Program Manager. The study was conducted by Dr. Honghai Li of Coastal Engineering Branch (CEB), Kevin B. Conner, Dr. Grace. M. Maze, and John M. Hazelton of the Wilmington District. This work was conducted under the general administrative supervision of Lauren M. Dunkin, Chief of CEB, and Dr. Jackie S. Pettway, Chief of the Navigation Division. Funding for the study was provided by the Wilmington District. Mr. Jeffrey R. Eckstein and Dr. Ty V. Wamsley were the Deputy and Director of CHL during this study period, respectively.

At the time of publication of this report, COL Teresa A. Schlosser was Commander and Executive Director of ERDC. Dr. David W. Pittman was ERDC Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
Feet	0.3048	meters
Yards	0.9144	meters

1 Introduction

1.1 Study area

Stono Inlet lies about 10 miles (16.1 km) southwest of the Charleston Harbor, South Carolina. Folly Beach is located on Folly Island, which is about six miles long and has a maximum width of 2,800 ft (853 m) near the center and narrows to under 200 ft (61 m) wide on the northeast end. Folly Island is bounded by Lighthouse Inlet on the northeast and by Stono Inlet to the southwest. The tidally influenced Folly River is located behind the southwest end of the island. The Folly River Navigation Channel is a shallow-draft channel. The dredged channel stretches out from downstream of the river to Stono Inlet and further extends to the open ocean through inlet ebb shoals. Kiawah Island is on the southwest side of Stono Inlet and Stono River enters the inlet from the north (Figure 1-1).

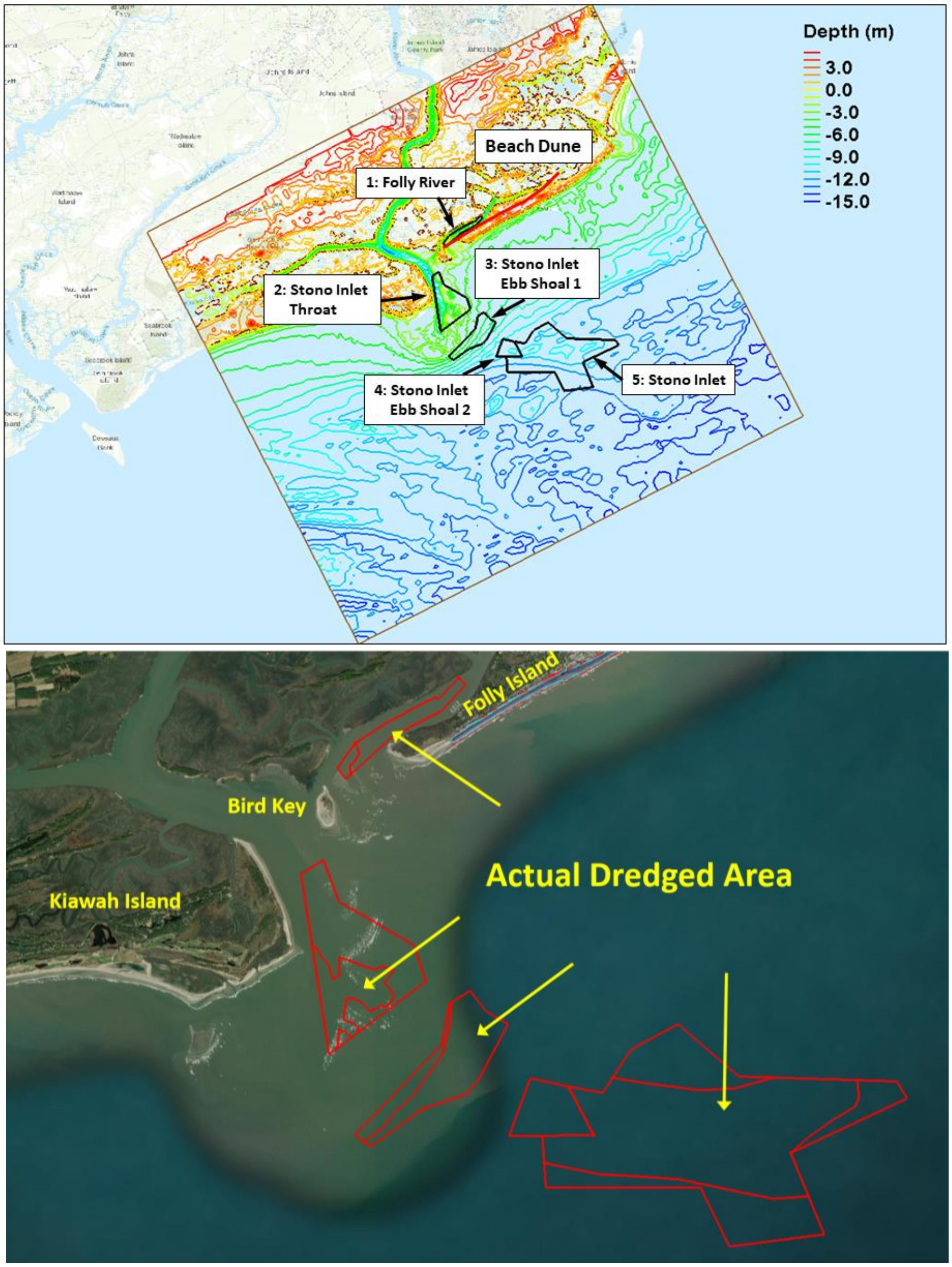

Figure 1-1. Stono Inlet, Folly Beach, and designated sand borrow areas (Folly River; Area I = Stono Inlet Throat; Area J = Stono Ebb Shoal 1; Area K = Stono Ebb Shoal 2; Area E = Stono Inlet). The yellow arrows indicate the actual dredged areas in the study.

Elevations on the island range from a low of 5 ft (1.5 m) to over 14 ft (4.3 m) NAVD88 along a remnant dune system that runs intermittently along the center of the island. The entire length of Folly Beach is experiencing shoreline recession with higher rates at the ends of the island and lower rates along the middle. The predominant longshore drift is toward the southwest. The mean grain diameter of the native beach is 0.17 mm. There are multiple groin fields along Folly Beach of varying effectiveness. In June 2013 a 745 ft (227 m) long steel sheet pile groin with armor stone toe protection was constructed on the southwest end of the island. The Folly Beach shoreline is protected by numerous concrete and timber sheet pile bulkheads, stone revetments, concrete rubble revetments and bulkheads with armor stone at the base. The structures are of various length, elevation, design, age, and construction quality.

The U.S. Army Corps of Engineers (USACE) Wilmington District (SAW) is evaluating continued Federal interest of Folly Beach, SC coastal storm risk management (CSRM) project. The project extends 28,890 feet (8,806 m) along beachfront of the City of Folly Beach. The past borrow sources for the nourishment project include offshore areas and the Folly River. The Folly River navigation channel has routinely been dredged since the 1970's. The first large-scale dredging of Folly River was in 1993 with 3.1 mcy (2.4 million m^3) removed from the river and 2.7 mcy (2.1 million m^3) placed along Folly Beach. The most recent project placed 1.2 mcy (0.9 million m^3) in 2018 with material from the Folly River.

1.2 Purpose of study

Dredge and placement activities significantly modify nearshore bathymetry and lead to sediment movement and redistribution. In order to assess the impact of borrow area selection on local morphologic changes and understand nearshore sediment transport in the area, the selection and design of borrow areas must be carefully examined for suitable sand material surrounding the littoral system. At the same time beach nourishment projects need to be carried out to mitigate shoreline erosion.

The purpose of the study is to evaluate the transport and morphologic changes for sediment material dredged from five borrow source areas as shown in Figure 1-1 and placed on Folly Beach adjacent to Stono Inlet. A coastal numerical model is applied to calculate waves, current, tide, sediment transport, and morphology change, and to perform sediment budget analysis within and around the immediate vicinity of Stono Inlet, Bird Key, Folly Island, and the eastern end of Kiawah Island. Sediment management alternatives on sand dredge and placement are developed, the effect of major forcing conditions (hydrodynamics, waves, and wind) on sediment movement is determined, and the impact of dredge/placement activities on sediment balance is investigated around Stono Inlet, the Folly River Navigation Channel, and Folly Beach.

1.3 Report Outline

This report is organized as follows. Chapter 2 introduces historical data that were applied to configure the numerical model and drive the numerical simulations, and the field data collection effort. Chapter 3 describes the methods for the numerical study. Chapter 4 presents the results of numerical modeling including the calibration and validation of the calculated waves and hydrodynamics to the field measurements. Chapter 5 summarizes the results of the study and provides conclusions regarding sediment transport around material borrow areas adjacent to the Stono Inlet estuarine system.

2 Data

A variety of physical and environmental data for the Stono Inlet study area were assembled and analyzed in the present study. Historical data available include open ocean, inlet channel and estuarine bathymetry, tidal variations, coastal wind and waves, and sediment composition.

2.1 Historical data

Bathymetry

Bathymetric data were compiled from a combination of ocean, beach, river surveys, and LIDAR data using Surface-water Modeling System (SMS 13.0) (Aquaveo 2020). Where merged datasets overlapped coverage, priority was given to the newest dataset. Folly River, Folly Beach, and channel surveys were conducted in Spring and Fall 2017, Fall 2018, and Spring 2019 separately. These surveys were collected as a part of the ongoing navigation projects from the USACE Charleston District (SAC). The other areas within the model domain were covered by the 1/9 arc-second Coastal Digital Elevation Model (DEM) dataset and the 3 arc-second Coastal Relief Model (CRM) dataset developed at NOAA's National Centers for Environmental Information (NOAA NCEI 2020). Shoreline data were used from the NOAA Continually Updates Shoreline Product (CUSP) (NOAA NGS 2011) and Google Earth images.

Figure 2-1 shows the spatial coverages and the depth (land elevation) contours of the merged datasets in the study area. Around Stono Inlet entrance channel and the material borrow areas, the latest conditional survey data were used to update areas of overlap with DEM/CRM data. The LIDAR surveys, DEM, and CRM datasets have more thorough coverage of land, coastal, and offshore areas with a high spatial resolution. Because of its uniform and dense data distribution, only the extent of the spatial coverage is shown in Figure 2-1(a). Using the datum information of NOAA tide gage #8665530 at Charleston, South Carolina (NOAA, 2020) all datasets were converted to local Mean Sea Level (MSL) and incorporated in numerical wave and flow models.

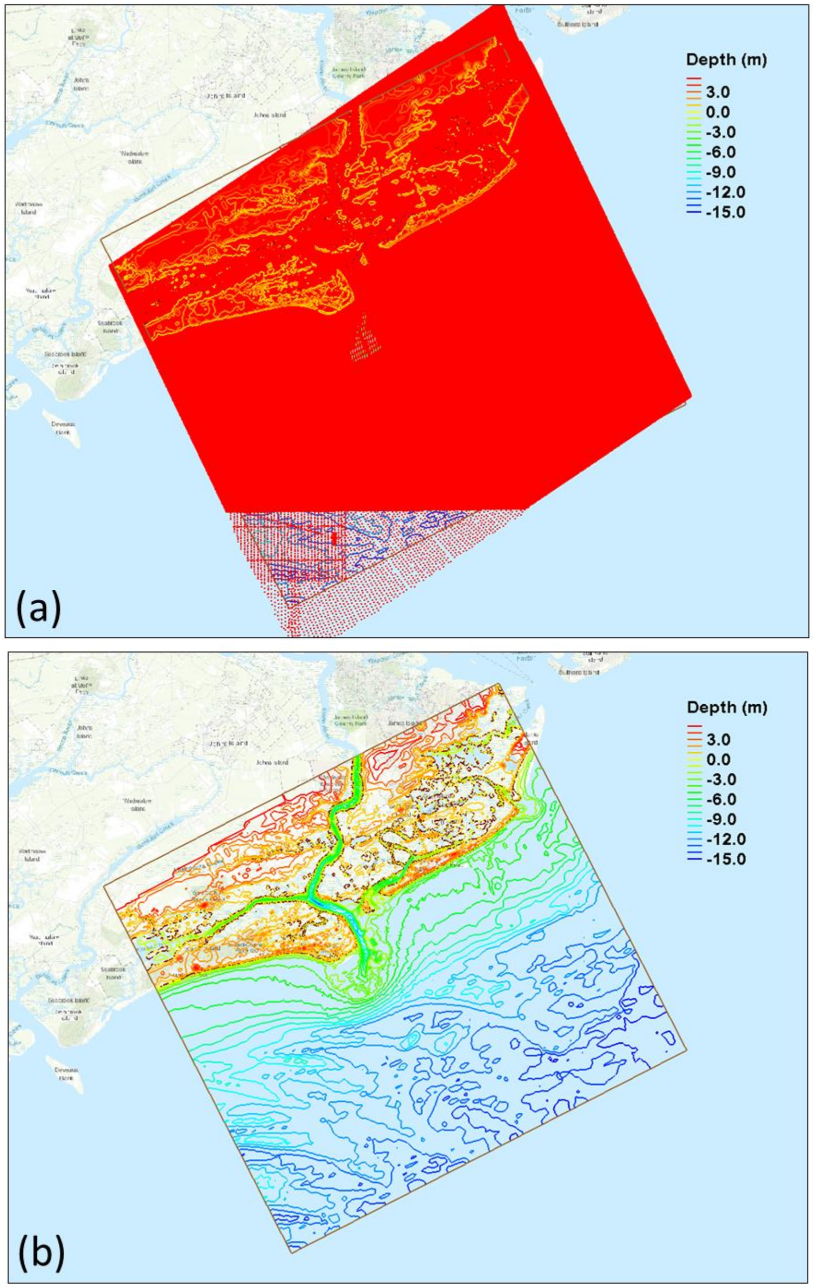

Figure 2-1. (a) NOAA DEM/CRM bathymetry and LIDAR data coverage, survey lines by USACE SAC. (b) Depth and land elevation contours of the merged dataset.

Tide

Water surface elevation (WSE) data were downloaded from NOAA Charleston tide gage #8665530 (NOAA, 2020). Figure 2-2 shows the location of the gage. A record of WSEs from 21 November to 31 December 2019 is plotted in Figure 2-3, which indicates distinguished spring and neap tidal ranges, and a mixed, predominately semi-diurnal tidal regime. The mean tidal range (mean high water – mean low water) is 1.59 m and the maximum tidal range (mean higher high water - mean lower low water) is 1.76 m.

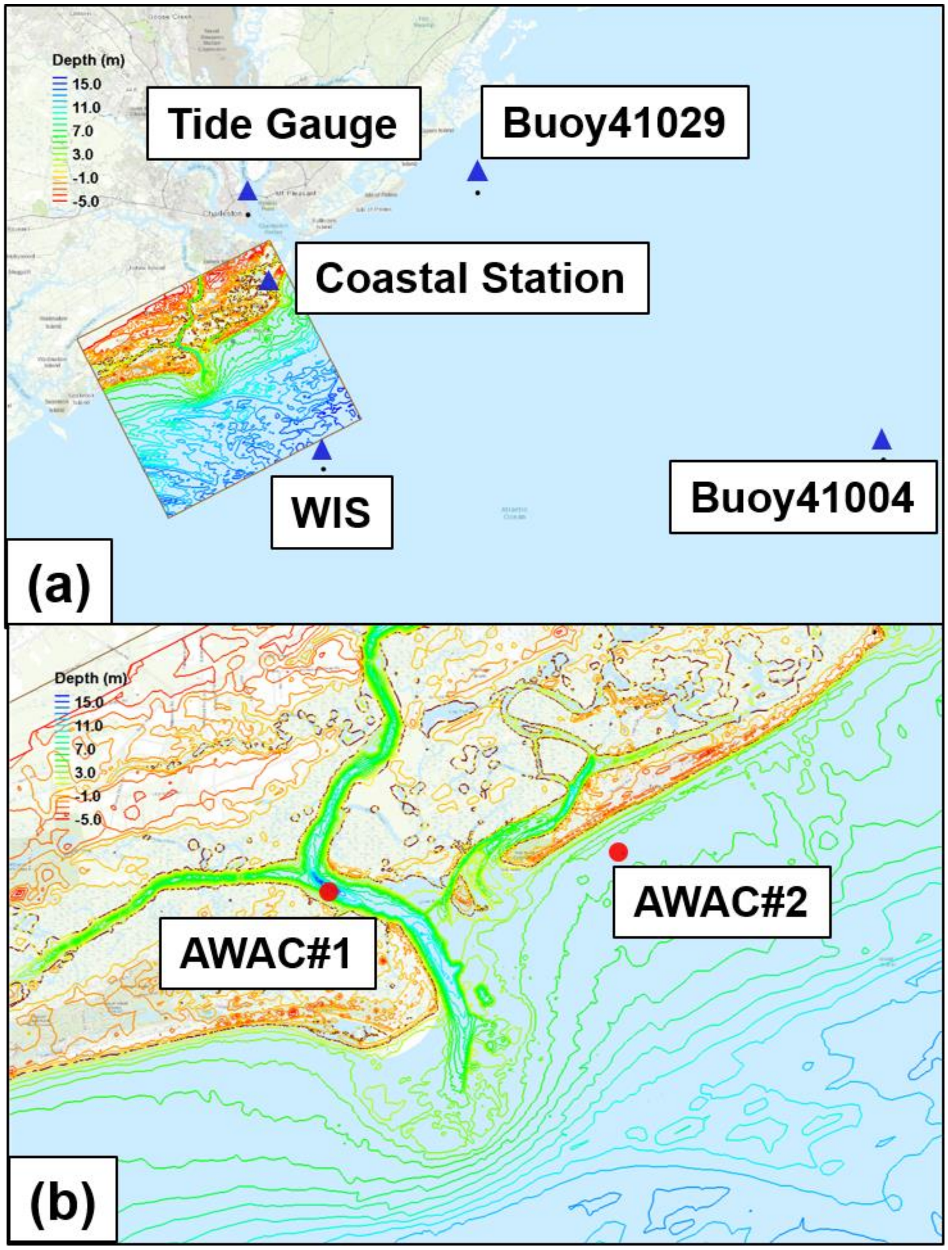

Figure 2-2. Locations of (a) NOAA tide gauge in Charleston Harbor, South Carolina (#8665530), WIS station, NDBC buoys, and (b) two AWAC gauges deployed around Stono Inlet.

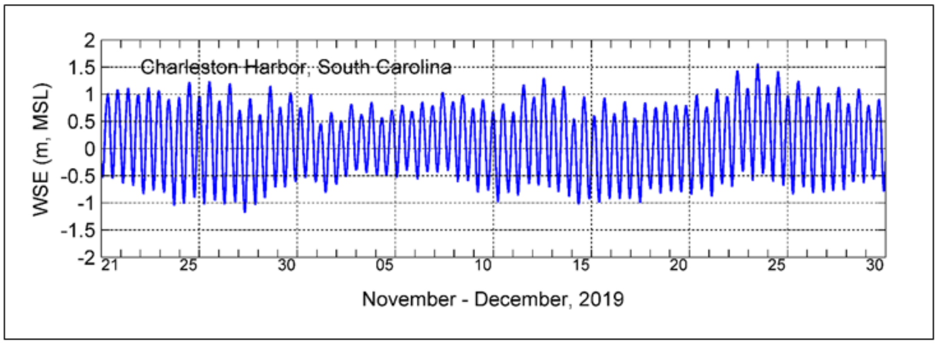

Figure 2-3. Water surface elevations measured at #8665530 from 21 November to 30 December 2019.

Wind and waves

Wind data were obtained from a National Data Buoy Center's (NDBC, 2020) land station, Station FBIS1, and a nearshore buoy, Buoy 41029, for different simulation periods. Station FBIS1 is located on the northern side of Folly Island and Buoy 41029 approximately 40 km northeast of Stono Inlet (Figure 2-2). Figure 2-4 shows a wind rose at Buoy 41029 using the data from 2015 to 2019. The 5-year dataset indicates two dominant shore parallel wind directions, southwesterly and northeasterly, in the region. Southwesterly wind occurred close to 37% of the time while northeasterly wind occurred approximately 30% of the time. On average, the northeasterly wind had a stronger speed. When wind blew from the northeast direction, about 10% of time wind speed reached 10 m/sec and above. The 5-year mean wind speed is around 5.7 m/sec.

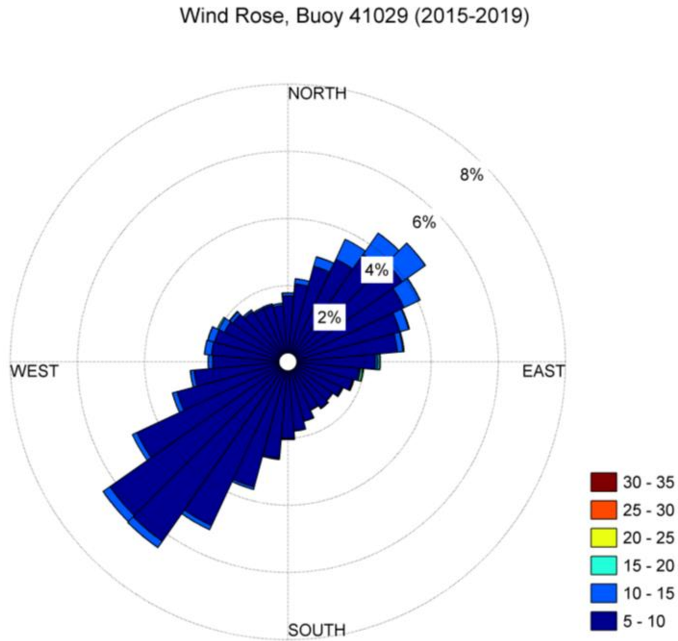

Figure 2-4. Wind rose at the NDBC buoy 41029 from 2015 to 2019.

Wave data were downloaded from a NDBC's offshore buoy 41004, located approximately 85 km east of Stono Inlet. When no wave data were available, U.S. Army Corps of Engineers' (USACE) hindcast wave field climatologies, Wave Information Studies (WIS), were used (wis.usace.army.mil, accessed 5 October 2020). WIS Station 63350 is the closest nearshore station to Stono Inlet (20 km, Figure 2-2) and provides wave parameters (wave height, wave period, and wave direction) for this modeling study. Figure 2-5 shows a wave rose at Buoy 41004 for the data from 2015 to 2019, which indicates that more than 50% of the time waves propagate from the southeast sector, close to the shore normal direction. The secondary dominant wave direction is southwest, approximately parallel to shoreline, which occurs close to 20% of the time. The region experiences a mild wave conditions year round and only about 11% of the time significant wave heights are above 2 m, propagating from the two dominant directions. The 5-year mean significant wave height is around 1.3 m and the peak wave height is between 5 and 6 m, usually corresponding to tropical or extratropical storms.

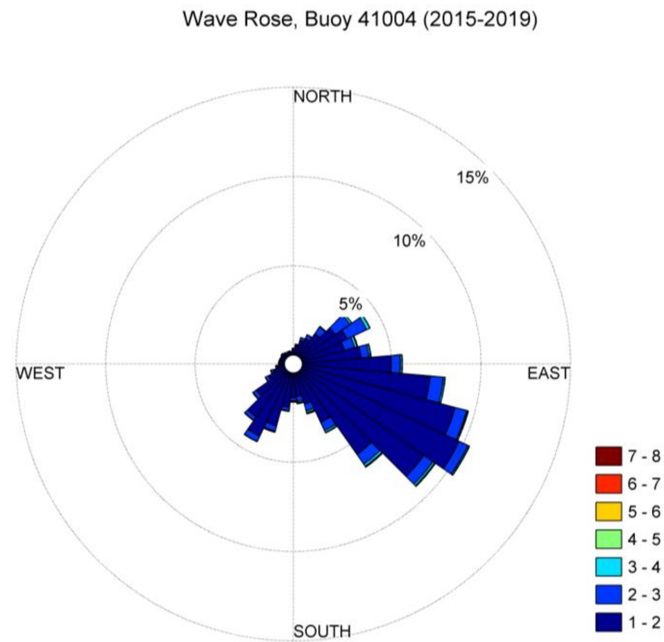

Figure 2-5. Wave rose at the NDBC buoy 41004 from 2015 to 2019.

Figure 2-4 and Figure 2-5 show that the study area is characterized by a high-energy wave climate and experiences energetic wave conditions during the passages of extra-tropical storms in the winter. Because of the pattern changes in monthly mean waves and wind, the sediment transport pattern in this region is also expected to change during the study period from September 2015 to March 2016.

Sediment

Sediment data were compiled from vibracore samples taken from 2002-2015 within Folly River, Stono Inlet, and offshore potential borrow areas. Grab samples from Folly Beach were collected in 1994 and 1998. Mean grain size, D50, was available at sample locations as shown in Figure 2-6. In the nearshore areas around the inlet and the material borrow sites the average D50 is 0.17 mm and in the offshore area the average D50 value is 0.25 mm.

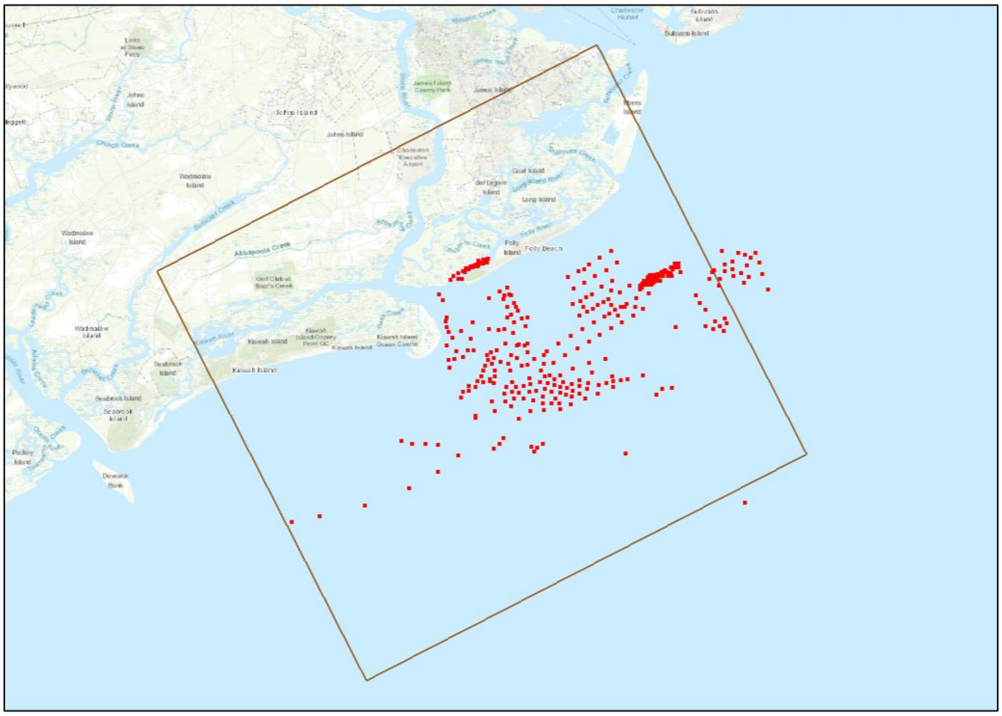

Figure 2-6. Location of sediment grab samples.

2.2 Field data collection

For this numerical modeling study two upward-looking acoustic wave and current profilers (AWACs) were deployed. AWAC#1 was located in the Stono River, off the deepest part of the river channel at a water depth of 9.1 m, and AWAC#2 was close to Folly Beach in the nearshore open area at a water depth of 5.1 m (Figure 2-7). The deployment was originally planned for one-month but lasted over four months starting on 22 November 2019 and ending on 11 April 2020. Severe weather conditions and instrument burials resulted in the unexpected delayed recovery of the AWACs.

Waves, current, and water surface elevation were measured at each AWAC location. It was noted that the acoustic transceivers had been buried repeatedly after 27 December 2019. The AWAC data measured between 22 November and 25 December 2019 were analyzed and used for model calibration and validation.

AWAC#1 was located in the river channel and the along-channel current component was much greater than the cross-channel component. AWAC#2 sat in the open ocean area nearshore and received strong wave impact. Therefore, the longshore current was much greater than the cross-shore current. Figure 2-7 shows the along-channel and the longshore current measured at AWAC#1 and AWAC#2, respectively. The current variations in Figure 2-7a illustrate that the along-channel current at AWAC#1 ranges from -1.0 m/s (ebb current) to +0.7 m/s (flood current) and the averaged current speed id -0.1 m/s, indicating an ebb-dominated estuary. Figure 2-7b displays a low-frequency variation of longshore current at AWAC#2. Overlapping the low-frequency signal is the tidal signal. Clearly, the tidal current at AWAC#2 is much weaker compared with that at AWAC#1 and tidal amplitude has a range of 0.2-0.4 m.

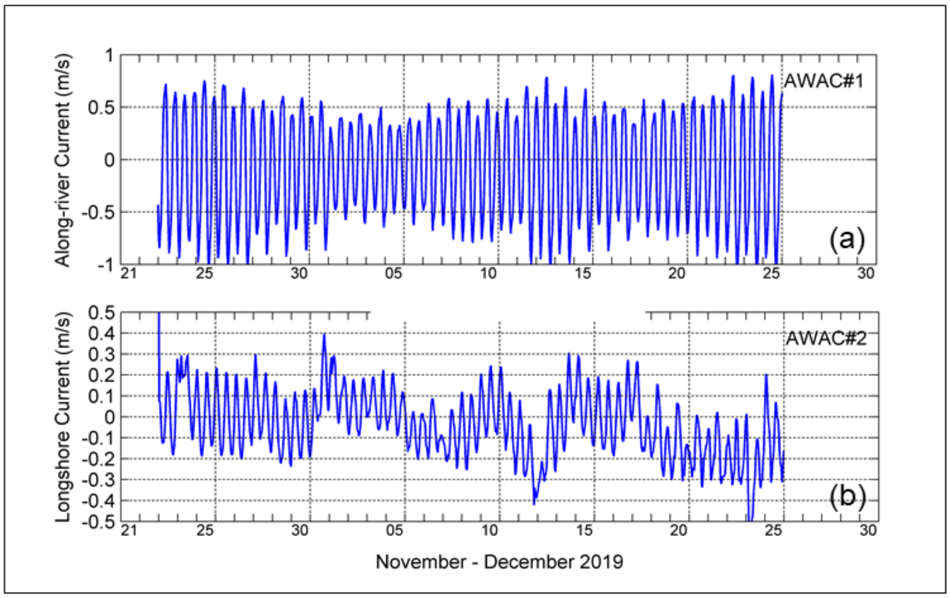

Figure 2-7. (a) Along-channel current at AWAC#1 and (b) longshore current at AWAC#2 from 22 November to 25 December, 2019. Positive values at AWAC#1 are designated for the flood current and negative for the ebb current. Positive values at AWAC#2 are designated for the ebb current (northeastward) and negative for the flood current (southwestward).

Figure 2-8 shows water surface elevations at AWAC#1 and AWAC#2 from 21 November to 25 December 2019. Both survey sites display similar tidal fluctuations, varying from -1.2 to 1.4 m. Examining the values at AWAC sites and NOAA Charleston Harbor site (Figure 2-3), the water surface elevation observed around Stono Inlet well corresponds to the measurements at the NOAA Charleston gage. The primary difference is that the tidal phase at Stono Inlet is leading that at Charleston Harbor by about 45-50 minutes.

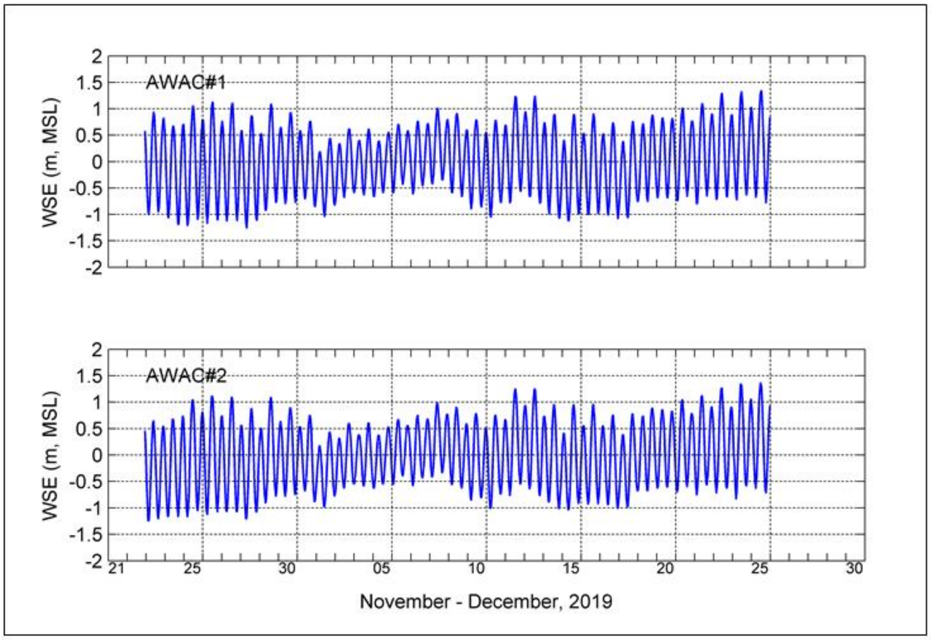

Figure 2-8. Water surface elevation at (a) AWAC#1 and (b) AWAC#2 from 22 November to 25 December, 2019.

The measured wave parameters (significant wave height, peak wave period, and mean wave direction) at AWAC#2 are shown in Figure 2-9 for the late fall and early winter period. Corresponding wind speed and direction observed at NDBC Buoy 41029 are shown in Figure 2-10. The mean significant wave height was 0.7 m and the mean wave period was 8.2 sec during the period. A few weather events with a wind speed greater than 10 m/s caused relatively large waves and the maximum wave height greater than 2 m occurred on 23 December, 2019. The weather events were mostly related to winter storms with wind blowing from the north-northeast direction. Nearshore waves measured at AWAC#2 primarily propagated from the south-southeast direction approximately normal to the shoreline, which was due to wave refraction when approaching shallow coastal area.

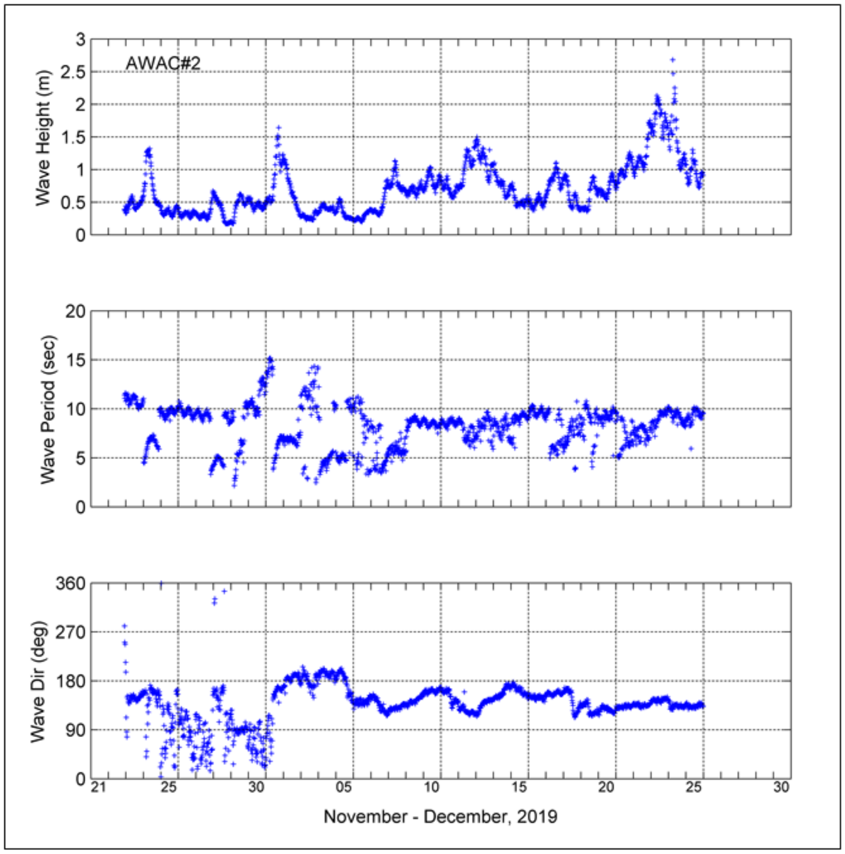

Figure 2-9. Significant wave height, peak wave period, and mean wave direction at AWAC#2 from 22 November to 25 December, 2019.

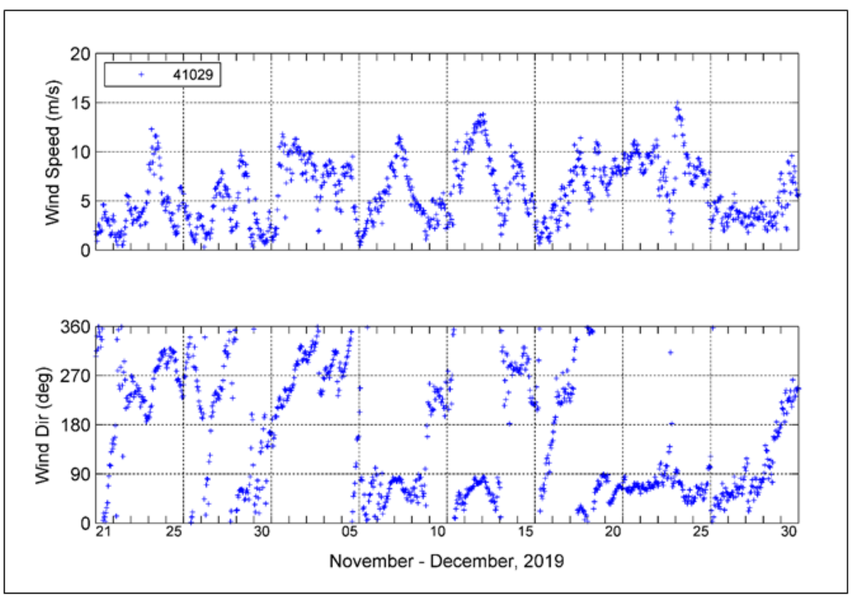

Figure 2-10. Wind speed and direction at NDBC Buoy 41029 from 21 November to 30 December, 2019.

3 CMS Modeling

The Coastal Modeling System (CMS) is an integrated suite of numerical models for waves, flows, sediment transport, and morphology change in coastal and inlet applications. This modeling system includes representation of relevant nearshore processes for practical applications of navigation channel performance and sediment management at coastal inlets and adjacent beaches. The CMS consists of a hydrodynamic and sediment transport model (CMS-Flow) and a spectral wave transformation model (CMS-Wave) (Sanchez et al. 2011a, 2011b, Lin et al. 2008). All pre- and post-processing for these models is performed within the ERDC Surface-water Modeling System (SMS) interface (Aquaveo 2020). The framework of CMS is shown in Figure 3-1. .

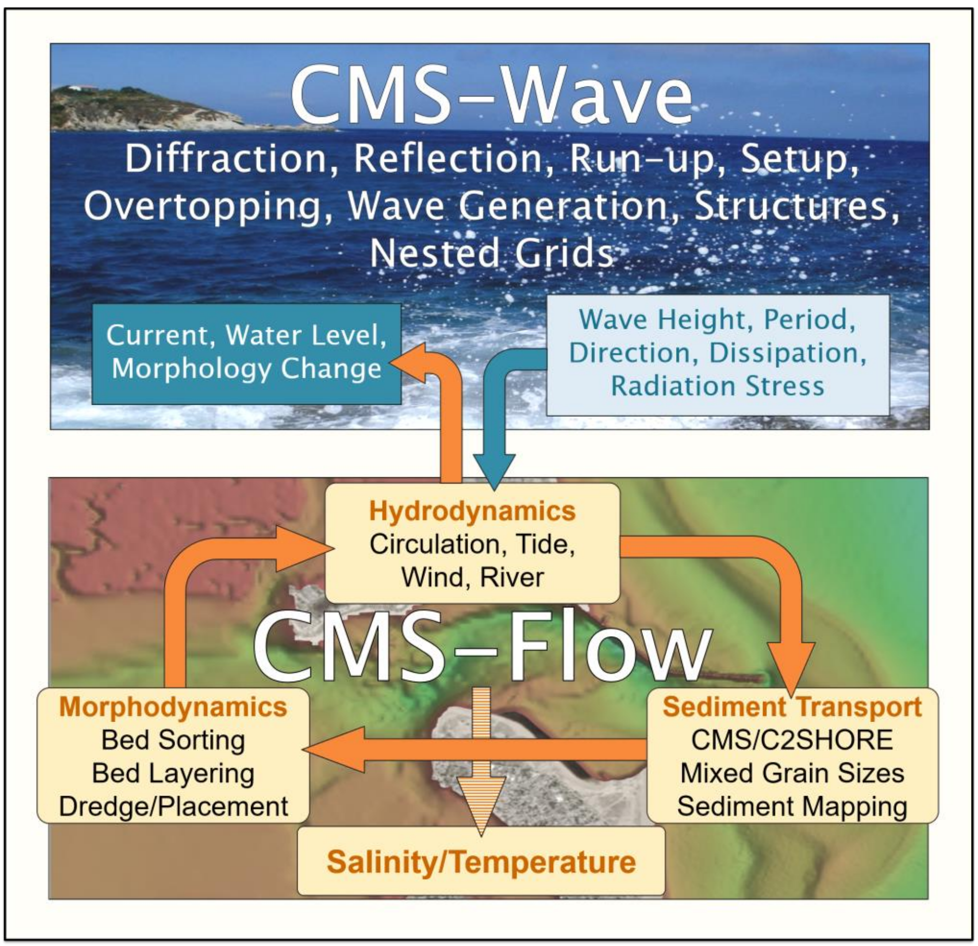

Figure 3-1. The CMS framework.

CMS-Flow is a two-dimensional depth-integrated (2-D) finite-volume model that solves the mass conservation and shallow-water momentum equations of water motion on a non-uniform Cartesian grid. CMS-Flow calculates hydrodynamics, sediment transport, and morphology change due to tide, wind, and waves. Wave radiation stresses and other wave parameters are calculated by CMS-Wave and supplied to CMS-Flow for hydrodynamic and sediment transport calculations.

CMS-Wave is a spectral wave transformation model. It solves the steady-state wave-action balance equation on a non-uniform Cartesian grid and is designed to simulate wave processes with ambient currents at coastal inlets and in navigation channels. The model can be used either in half-plane or full-plane mode and includes coastal wave processes, such as wind wave generation and growth, refraction, diffraction, reflection, dissipation due to bottom friction, white-capping and breaking, wave-current interaction, wave runup, wave setup, and wave transmission through structures.

CMS-Flow and CMS-Wave have a dynamic coupling at a certain time interval specified by users. For the Stono Inlet application, CMS-Wave was run at a one-hour interval in-between CMS-Flow simulations.

3.1 Model domain and model setup

A telescoping variable-resolution CMS-Flow grid was developed for the Stono Inlet and Folly Beach area (Wu et al. 2011). The areal extent for the modeling domain is approximately 26.4 kilometers alongshore and 24.8 kilometers across shore. The CMS domain consists of 126,000 ocean cells, which covers the Stono Inlet, Folly Beach, Folly and Kiawah Islands, Stono and Folly Rivers, and the open ocean region (Figure 3-2). The water depth ranges from 1-2 m above mean sea level at tidal marsh areas to 11 m at the Folly River Navigation Channel, and further increases to 16 m at the seaward boundary of the CMS domain. The telescoping grid system permits much finer local grid resolution to resolve hydrodynamic and sediment features in areas of high interest. For this study the cell sizes vary from 10 m in front of Folly Beach and the Stono Inlet navigation channel to 320 m in the open ocean. The CMS-Wave grid with varying cell sizes was generated for wave modeling, covering a smaller domain and with similar spatial resolution as the CMS-Flow grid (Figure 2-1 and Figure 3-2).

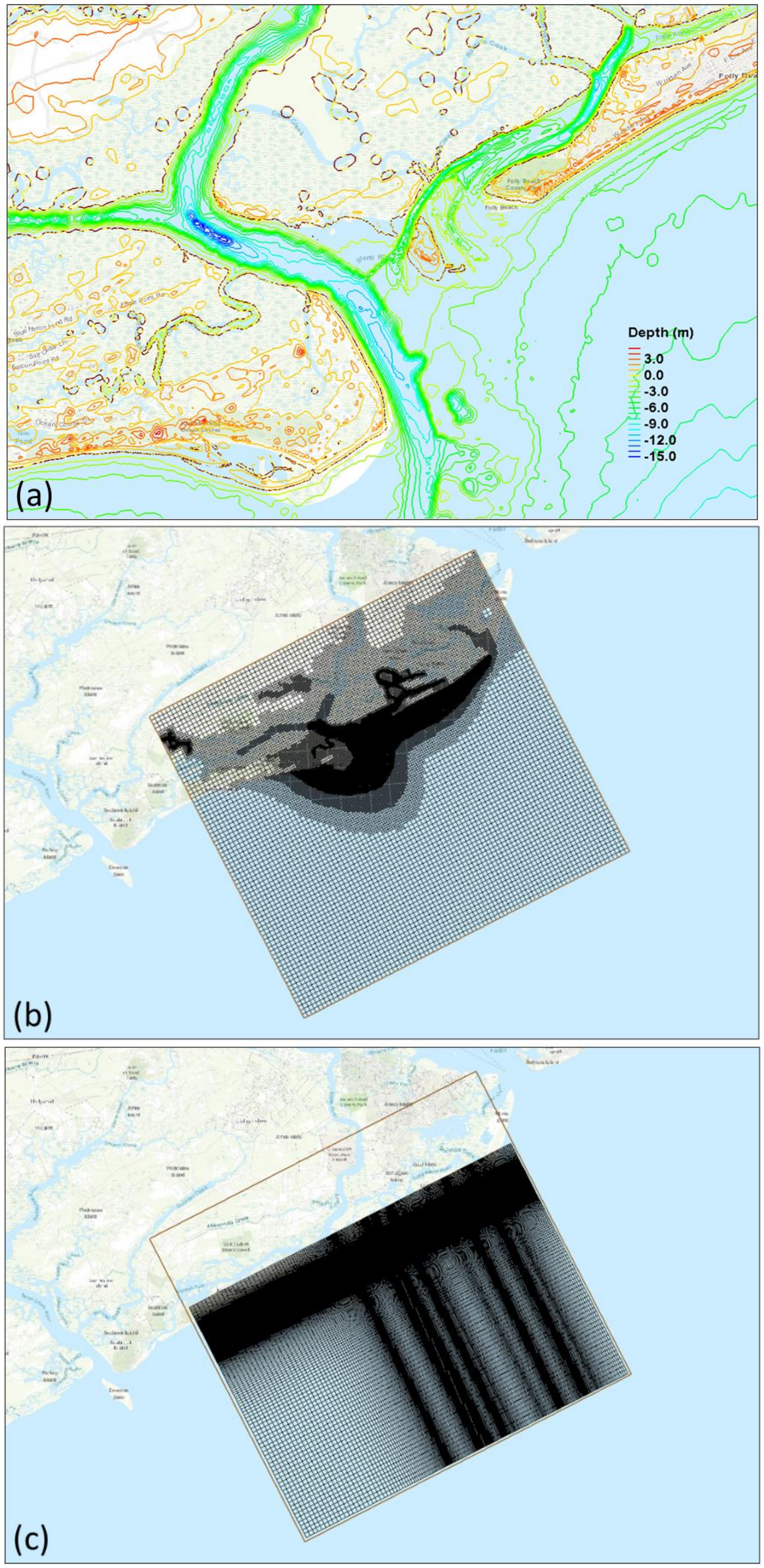

Figure 3-2. The CMS domain. (a) Bathymetry at the inlet entrance channel and the bay, (b) CMS-Flow telescoping grid, and (c) CMS-Wave variable rectangular grid.

3.2 Simulation periods

The field survey was conducted from 22 November 2019 to 11 April 2020. Corresponding to the survey period, simulations for the CMS calibration and validation were set up from 21 November to 30 December 2019. For production simulations, a storm simulation (Hurricane Hugo, 1989) and a medium-to-long term simulation (one year) were set up.

Hurricane Hugo

Hurricane Hugo is one of the most damaging hurricanes on the coast of South Carolina in September 1989. When making landfall on Sullivan's Island, 20 km northeast of Stono Inlet, on 22 September 1989, it was a Category 4 hurricane. After moving inland, the wind speed quickly decreased and Hurricane Hugo dissipated on 25 September 1989 (https://en.wikipedia.org/wiki/Hurricane_Hugo, accessed 19 October, 2020). Figure 3-3 shows the water-surface elevation and wind data, with the maximum surge and tidal level at 2.5 m above the MSL and the maximum wind speed of approximately 30.0 m/s at the NOAA coastal stations. Figure 3-4 shows the wave conditions with a maximum wave height close to 6.0 m and predominant wave direction from the east-southeast. As shown in Figure 3-3 and Figure 3-4, the CMS simulation period for Hurricane Hugo is from 18 to 25 September, 1989.

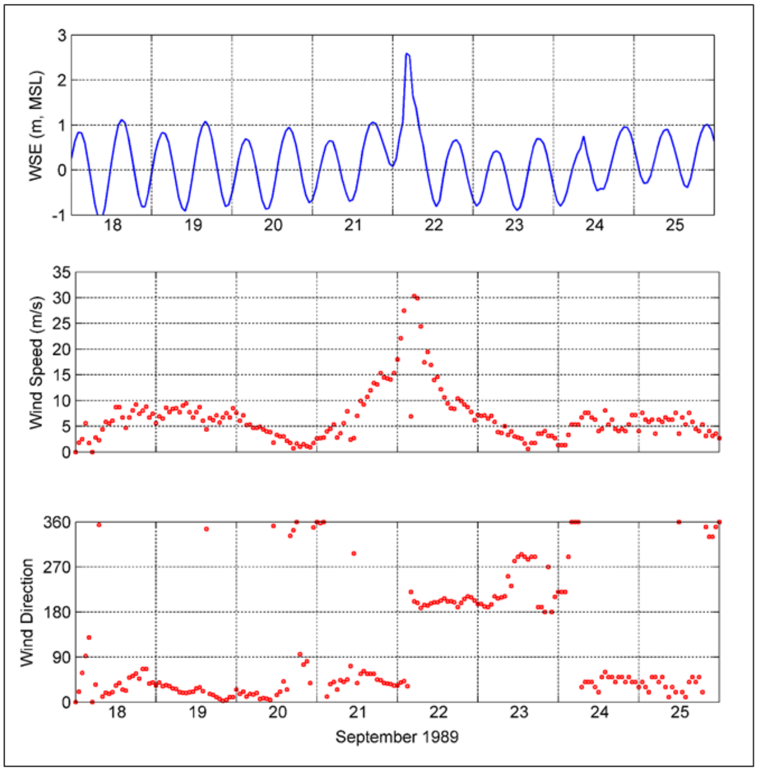

Figure 3-3. Water surface elevation and wind of Hurricane Hugo at NOAA gauge 8638610 (Charleston Harbor, South Carolina) and NDBC land station, FBIS1, respectively.

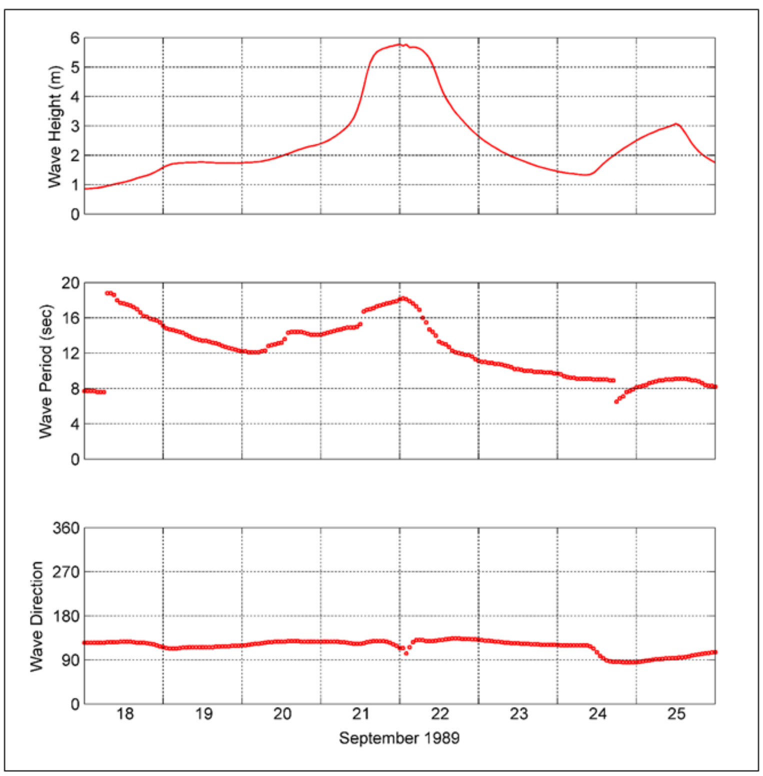

Figure 3-4. Wave parameters of Hurricane Hugo at WIS Station 63350.

2018

As shown in Figure 2-4 and Figure 2-5, the complete wind and wave records were obtained at NDBC buoys 41029 and 41004, respectively, from 2015 to 2019. In order to select a typical year for the medium-to-long term CMS simulation, wind and wave roses for each of the annual datasets were compared with 5-year average wind and wave conditions.

Like the 2015-2019 wind rose, the 2018 wind rose shown in Figure 3-5 also displays two dominant shore parallel wind directions, southwesterly and northeasterly, in the region. Southwesterly wind occurred close to 36% of the time while northeasterly wind approximately 29% of the time. Similarly, the northeasterly wind occurring in 2018 had a stronger speed. For the wind blowing from the northeast direction, a little more than 10% of time the wind speed reached 10 m/sec and above. The 5-year mean wind speed is around 5.7 m/sec.

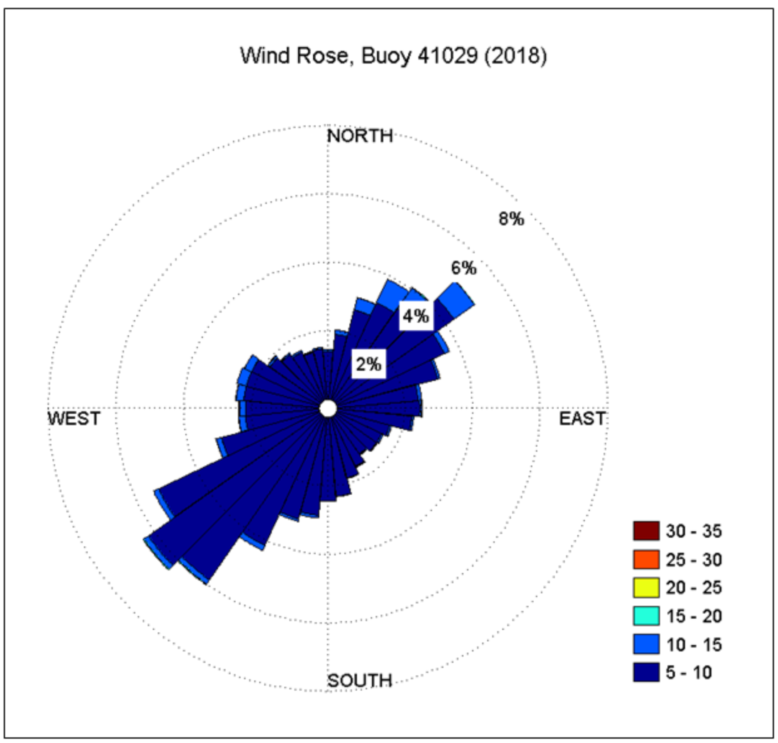

Figure 3-5. 2018 wind rose at the NDBC buoy 41029.

As shown in Figure 3-6 the 2018 wave rose at Buoy 41004 indicates that 59% of the time waves propagate from the southeast sector, close to the shore normal direction. The secondary dominant wave direction is approximately parallel to shoreline, propagating from the southwest direction and occurring close to 19% of the time. Benign wave conditions were also evident in the 2018 wave data. About 11% of the time significant wave heights are above 2 m for waves propagating from the southeast and southwest directions. The 2018 mean significant wave height is the same as the 5-year mean of 1.3 m and the peak wave height is also between 5 and 6 m, occurring in April 2018.

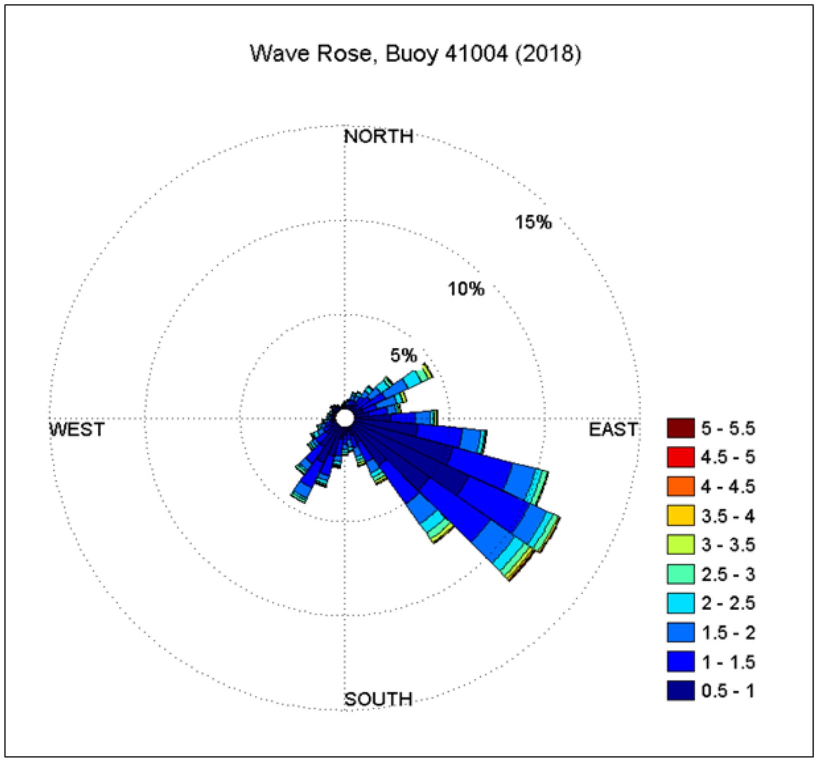

Figure 3-6. 2018 wave rose at the NDBC buoy 41004.

Figure 3-5 and Figure 3-6 illustrate that the 2018 wind and wave data present similar rose patterns and statistical properties as the 2015-2019 data. Therefore, the year of 2018 is selected to represent a typical year and the 2018 data are used to configure the medium-to-long term simulation.

3.3 Model forcing

The forcing to drive the CMS includes water surface elevation (WSE) and wind stress along the open boundaries and at the surface boundary of CMS-Flow, and wave spectrum at the seaward boundary of CMS-Wave. Normally, measured data at an adjacent site to model boundaries or model domain are the ideal choice for driving forces. But in situ measurements can be interrupted by unforeseen factors, such as weather conditions or instrument failure. Very often the driving forces of a numerical model have to rely on hindcast products like the WIS dataset or use multiple survey sites to obtain one complete time series. Three simulation periods were selected in the study; (1) the calibration and validation, (2) Hurricane Hugo, and (3) the year of 2018, for which expected forcing data may or may not be available.

NOAA tide gauge (#8665530) at Charleston Harbor, South Carolina provides the complete water level records for all three simulation periods. Initial data analysis shows that tidal phase at the harbor gage site leads that at Stono Inlet by about 0.8 hours. The phase difference was adjusted when water levels were specified along the CMS ocean boundary.

For the model calibration and validation period, wind data were downloaded from the Charleston Harbor gauge #8665530, the NOAA coastal station FBIS1, and the NOAA offshore buoy #41029. Wind speeds and directions were compared among the three sites and were used to test the CMS (Figure 3-7). Overall the stronger wind speeds occurred at the offshore buoy and the wind directions among three gauges had an angle difference of about 0-90° through the period. Because of the sheltering effect the harbor gauge shows the weakest wind speeds and the largest difference in wind directions. The CMS simulation driven with the offshore buoy wind yielded the best results.

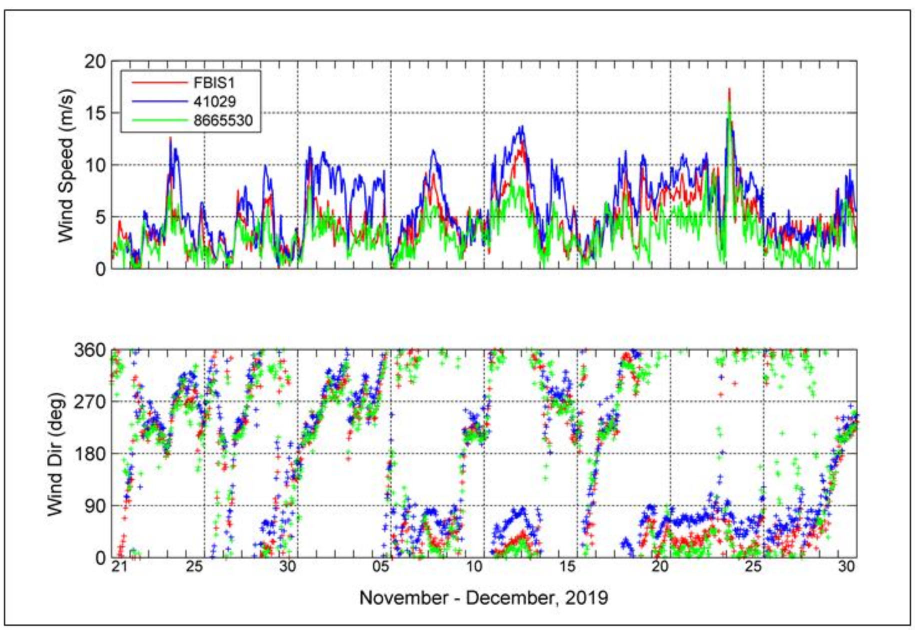

Figure 3-7. Wind speeds and directions at NOAA Charleston Harbor tide gauge #8665530, the NOAA coastal station FBIS1, and the NOAA offshore buoy #41029 from 21 November to 30 December 2019.

Besides selecting the buoy wind for the model calibration and validation period, the wind data from Buoy #41029 were also used for the 2018 simulation. Data gaps existing in the original measurements were filled with the wind data from the NOAA coastal station FBIS1.

NOAA buoy #41029 had not been operational until 2005. Therefore, during the Hurricane Hugo period, the wind data at FBIS1 were employed as the forcing term to drive the model and to reproduce wind-driven current in the model domain. After the hurricane passage, there is a 20-day data gap starting from 23 September 1989 at the station, which corresponds to the last two days of the CMS simulation with a wind speed less than 10 m/s. The gap was filled using the measured wind data at the Charleston Harbor (Figure 3-3).

Directional wave spectra are often used to drive a wave model because of the inclusion of the total wave energy. NOAA Buoy #41004 is the closest offshore site to Stono Inlet, where directional wave spectra are measured. For the calibration/validation and the 2018 simulations the hourly spectra at this location were transformed to the seaward open boundary of CMS-Wave. During the Hurricane Hugo period, the spectral wave data were not available at this buoy site. Hindcast wave parameters at WIS station #63350 were used to generate wave spectra by a TMA spectral shape for the CMS-Wave input (Lin et al 2008).

3.4 Model alternatives

Sediment management alternatives were developed based on sand dredge in designated areas around Stono Inlet and placement on Folly Beach. 2.5 million cubic yards (MCY) (1.9 million cubic meters) of sediment are removed from a projected borrow site and the materials are placed in 26 reaches along the beach for building berms and dune (Figure 3-8). Figure 3-9 shows the sketches of designed berms and dunes for reaches 2-21 and 22-26. Along the stretch of the shoreline, the dune has a height of 4.64 m (15.2 ft) with a crest width of 1.5 m (5 ft) relative to MSL. The berm has a height of 2.51 m (8.2 ft) with a top width of 10.7 m (35 ft) for reaches 2-21 and 15.2 m (50 ft) for reaches 22-26. This initial design was used for the current modeling study. The final design will include the same general berm and dune but might differ slightly in overall project length and reach scales.

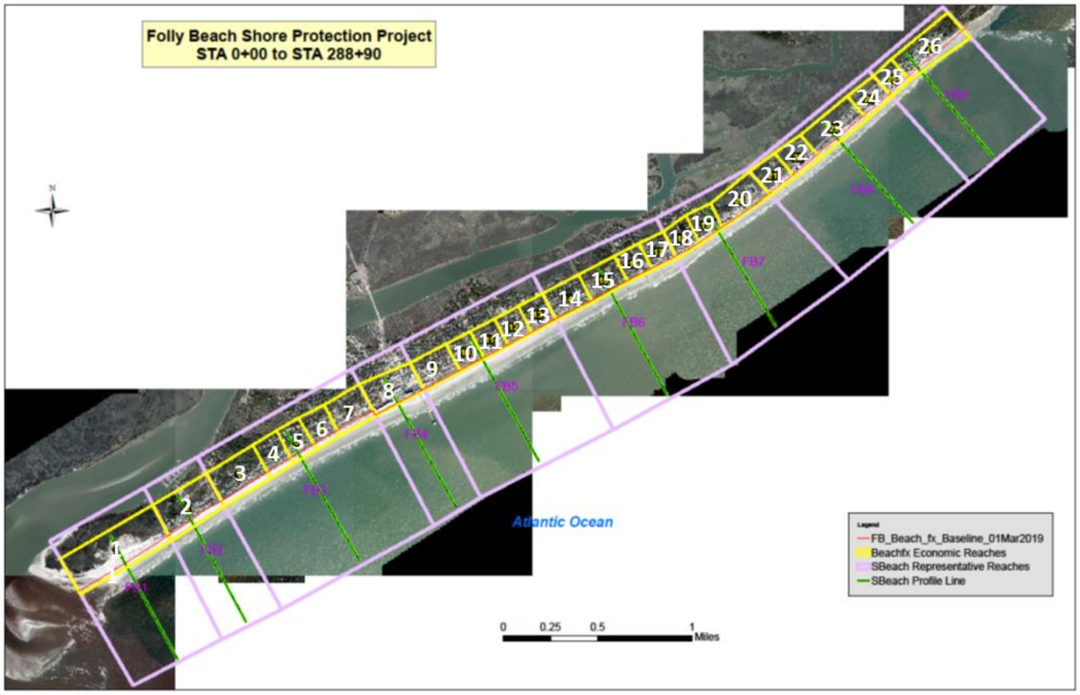

Figure 3-8. The reach map along Folly Beach.

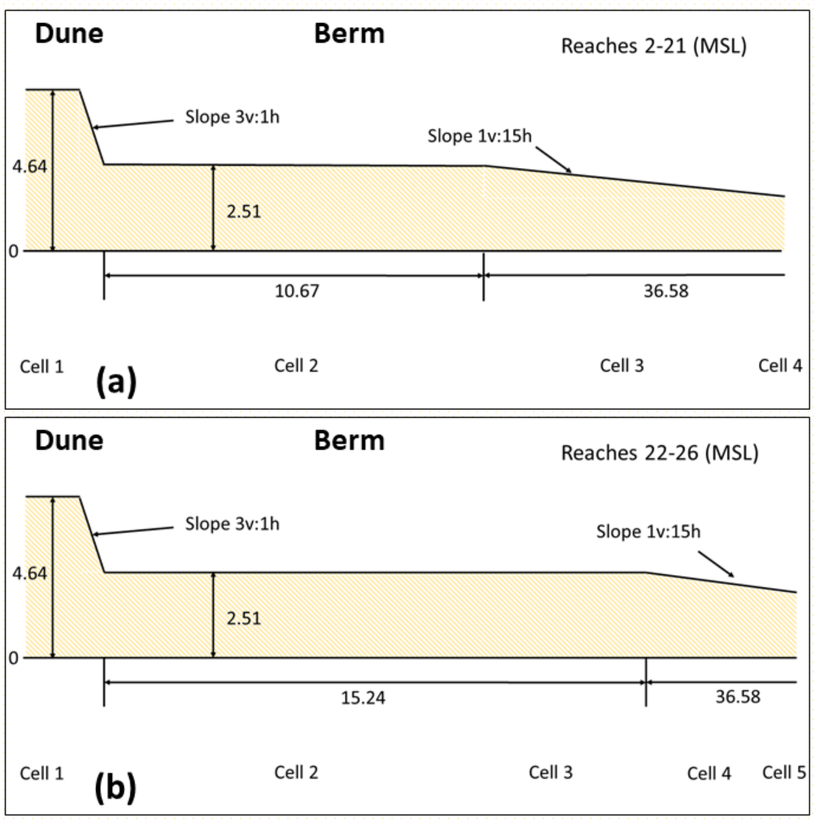

Figure 3-9. Sketches of designed berms and dunes for (a) reaches 2-21, (b) reaches 22-26.

Figure 3-10 shows the distribution of bathymetry and topography along Folly Beach before and after berm/dune placement. With this setup along the shoreline, five alternatives were specified corresponding to five designated borrow areas (Figure 1-1). From each area approximately 2.5 MCY of beach quality materials were dredged for the beach fill.

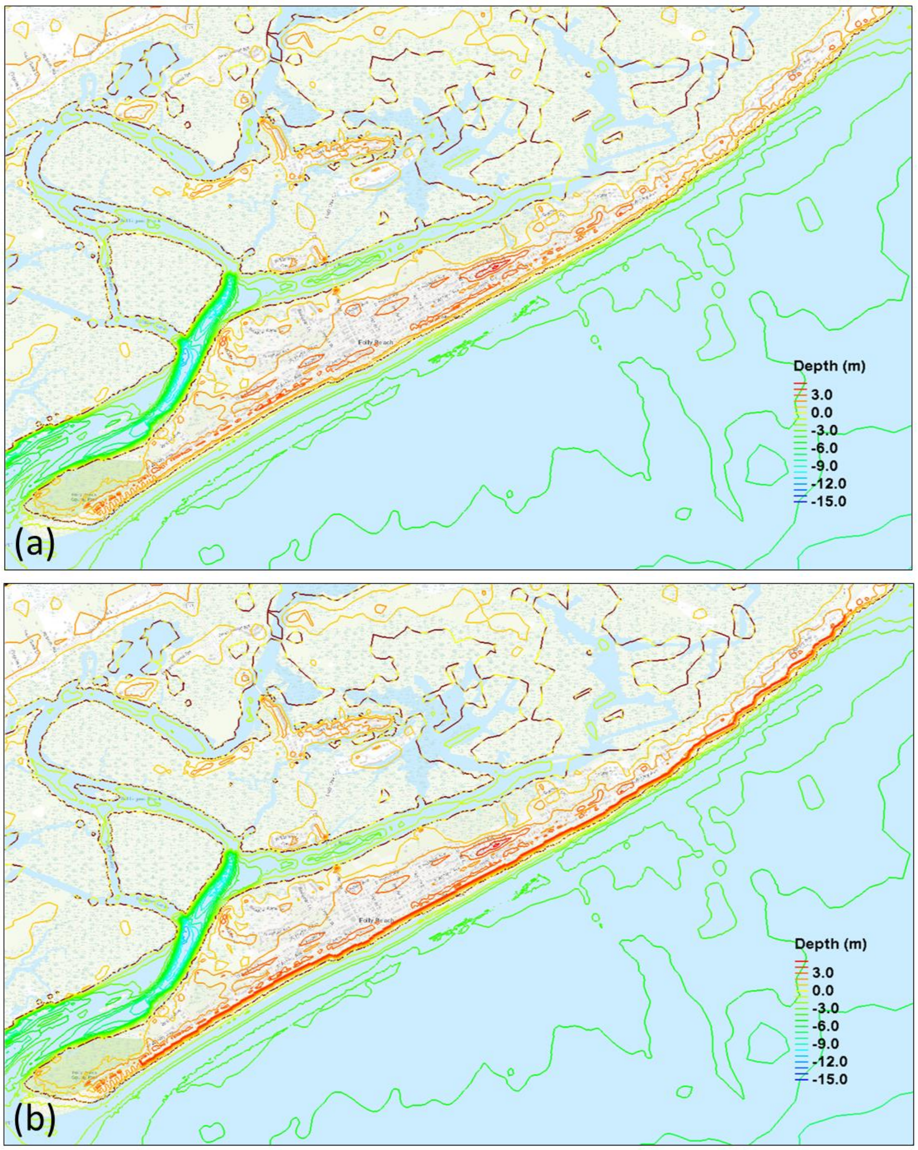

Figure 3-10. CMS bathymetry/topography along Folly Beach for (a) base case and (b) with built berm and dune.

Borrow area: Folly River (alternative 1)

The Folly River borrow area is located downstream of the Folly River and interrupts the Folly River navigation channel with shallow depths (Figure 3-11). The base case shows that the average water depth is 2.56 m within the area and the deepest portion is around 7.0 m relative to MSL (Table 3-1 and Figure 3-11a). For the alternative setup, most of the area was dreadged to 7.85 m and only the small portion of the area at the southwest corner was not dredged. The total sediment volume obtained from the area is about 2.50 MCY (1.91 million cubic meters). After dredging, the average water depth within the area is 5.3 m (Figure 3-11b).

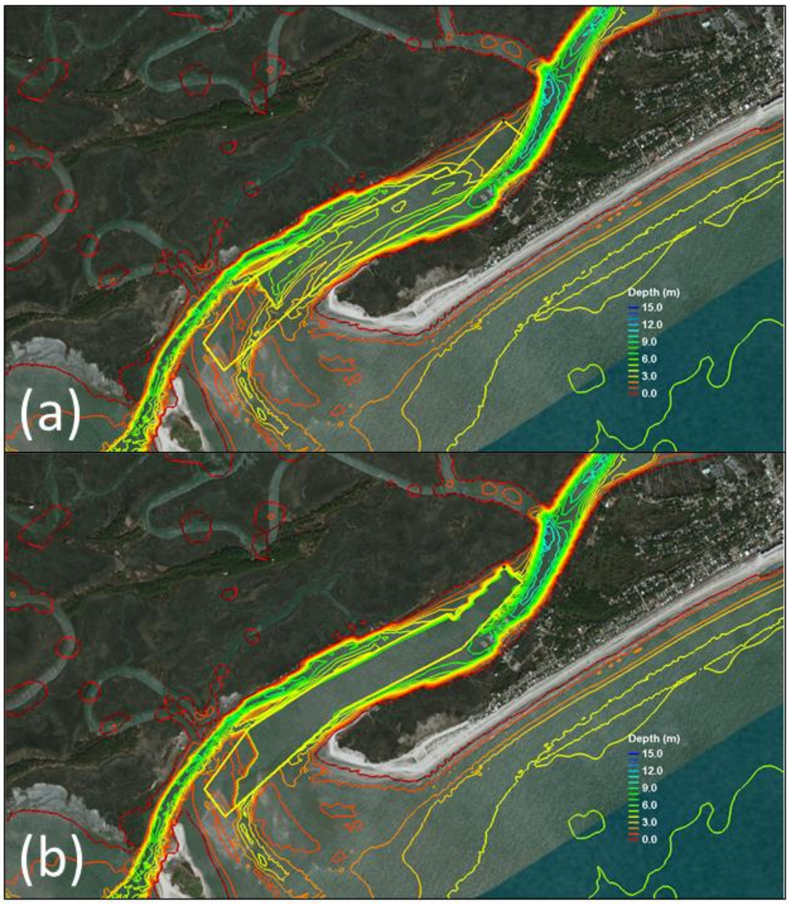

Figure 3-11. Bathymetry of Folly River borrow area. (a) base case (before dredge). (b) Alternative 1 (after dredge).

Table 3-1. Physical scales of the five borrow areas.

Alternative	Borrow Area	Size (m^2)	Average Depth before Dredge (m)	Average Depth after Dredge (m)
1	Folly River	613700	2.6	5.3
2	Stono Inlet Throat	3148200	4.9	5.3

3	Stono Ebb Shoal 1	2024930	5.9	6.9
4	Stono Ebb Shoal 2	874680	10.2	12.1
5	Stono Inlet	10543800	11.2	11.4

Borrow area: Stono Inlet Throat (area I, alternative 2)

Part of the Stono Inlet Throat borrow area covers the Stono River navigation channel and about 70% of the borrow area is located in the shallow part of the area (Figure 3-12). The base case shows that the average water depth is 4.9 m within the area and the deepest portion is around 10.0-11.0 m relative to MSL (Table 3-1 and Figure 3-12a). Because the channel area already reached an average depth of 8.6 m, sediment dredging was focusing on the shallow area for the development of the alternative, which was dredged to 7.55 m. The total sediment volume obtained from the area is also around 2.50 MCY (1.91 million cubic meters). After dredging the average water depth within the area is 5.3 m (Figure 3-12b), a 0.4 m increase comparing with the average before dredging.

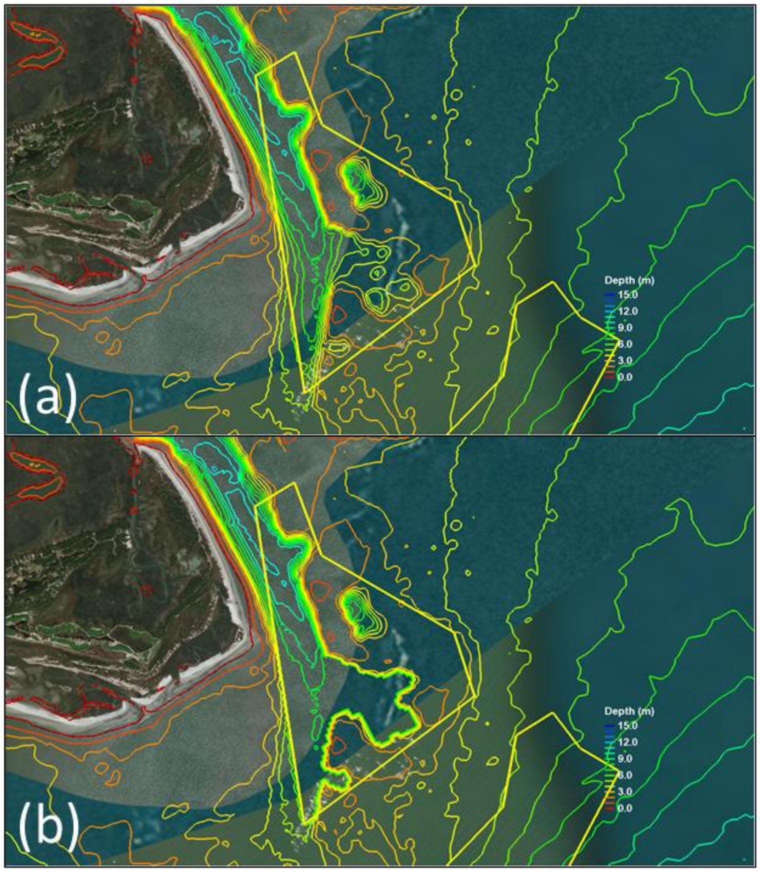

Figure 3-12. Bathymetry of Stono Inlet Throat borrow area (area I). (a) base case (before dredge). (b) Alternative 2 (after dredge).

Borrow area: Stono Ebb Shoal 1 (Area J, alternative 3)

The Stono Ebb Shoal 1 borrow area is just off the shallowest ebb shoal on the slope facing the open ocean (Figure 3-13). The base case shows that the average water depth is 5.9 m within the area and the greatest water depth is around 8.0 m relative to MSL (Table 3-1 and Figure 3-13a). Within the area, sediment dredging also went to 7.55 m if the water depth was less than that value. The total sediment volume obtained from the area is around 2.52 MCY (1.92 million cubic meters). After dredging the average water depth within the area is 6.9 m (Figure 3-13b), a 1.0 m increase comparing with the average before dredging.

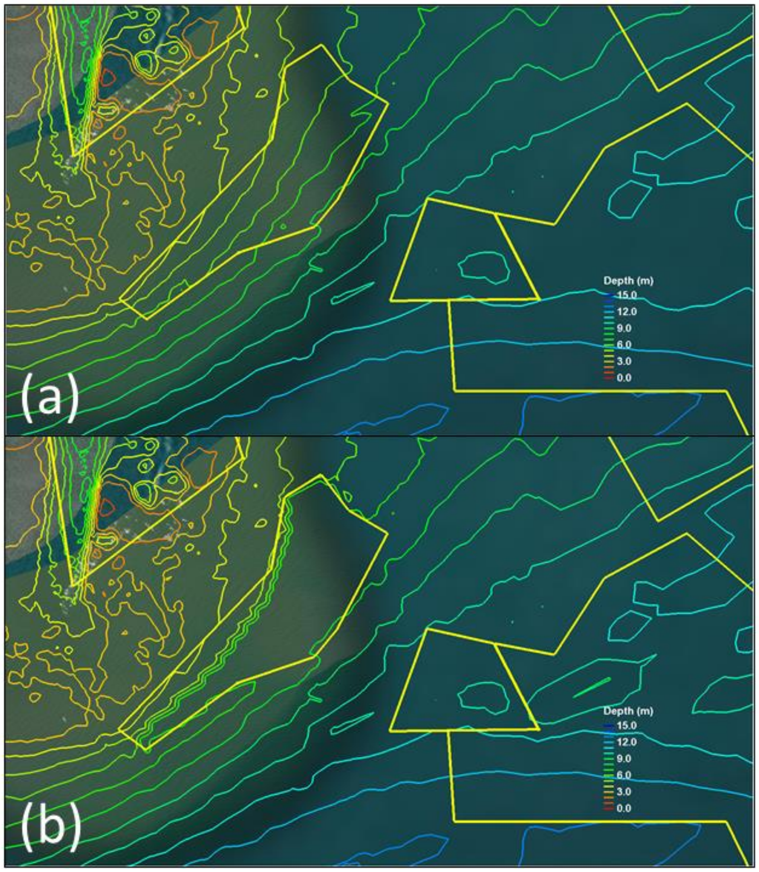

Figure 3-13. Bathymetry of Stono Ebb Shoal 1 borrow area (area J). (a) base case (before dredge). (b) Alternative 3 (after dredge).

Borrow area: Stono Ebb Shoal 2 (area K, alternative 4)

The Stono Ebb Shoal 2 borrow area is the second smallest area, and only a little larger than the Folly River area, but the water is much deeper (Table 3-1). The base case shows that the average water depth is 10.2 m within the area and the greatest water depth is close to 11.0 m relative to MSL (Figure 3-14a). For this alternative the entire area had to be dredged to 12.1 m to obatain sufficient amount of sediment materials (Figure 3-14b). The total sediment volume obtained from the area is around 2.54 MCY (1.94 million cubic meters). After dredging the average water depth within the area is 12.1 m (Figure 3-14b), approximately a 2.0 m increase comparing with the average before dredging.

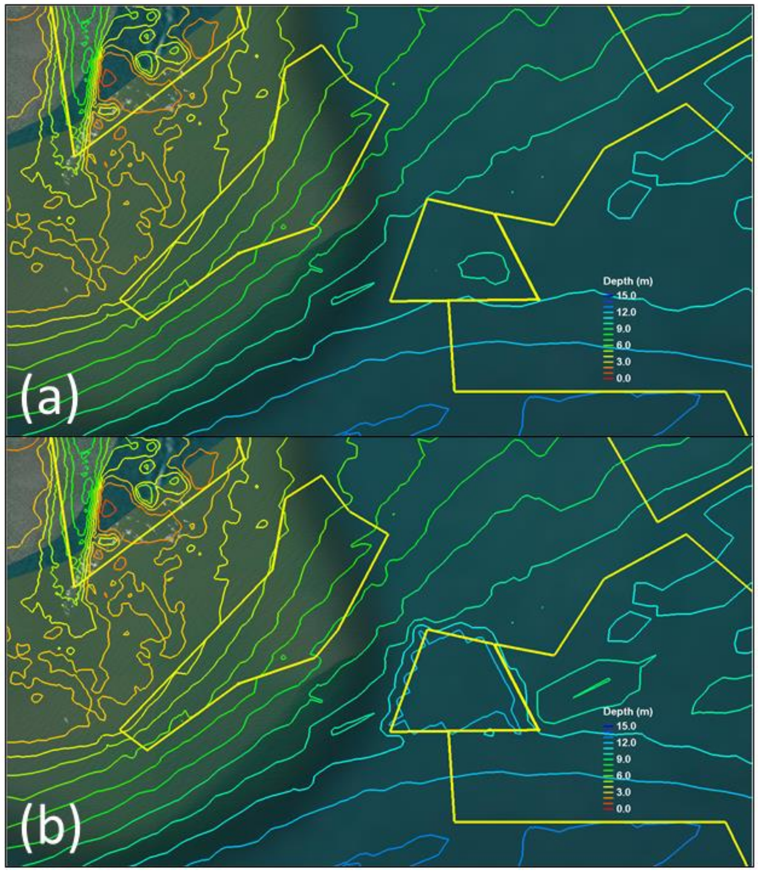

Figure 3-14. Bathymetry of Stono Ebb Shoal 2 borrow area (area K). (a) base case (before dredge). (b) Alternative 4 (after dredge).

Borrow area: Stono Inlet (area E, alternative 5)

The Stono Inlet borrow area is the largest among the five areas and located in deeper offshore zone (Table 3-1 and Figure 3-15). The base case shows that the average water depth is 11.2 m within the area and the greatest water depth is close to 13.0 m relative to MSL (Figure 3-15a). For this alternative the central portion of the area with the depth less than 11.1 m was dredged to 10.6 m or 11.1 m to obatain proper amount of sediment materials. The total sediment volume obtained from the area is around 2.50 MCY (1.91 million cubic meters). After dredging, the average water depth within the area is 11.4 m (Figure 3-15b), only an increase of 0.2 m comparing with the average before dredging because of the large coverage of this borrow area.

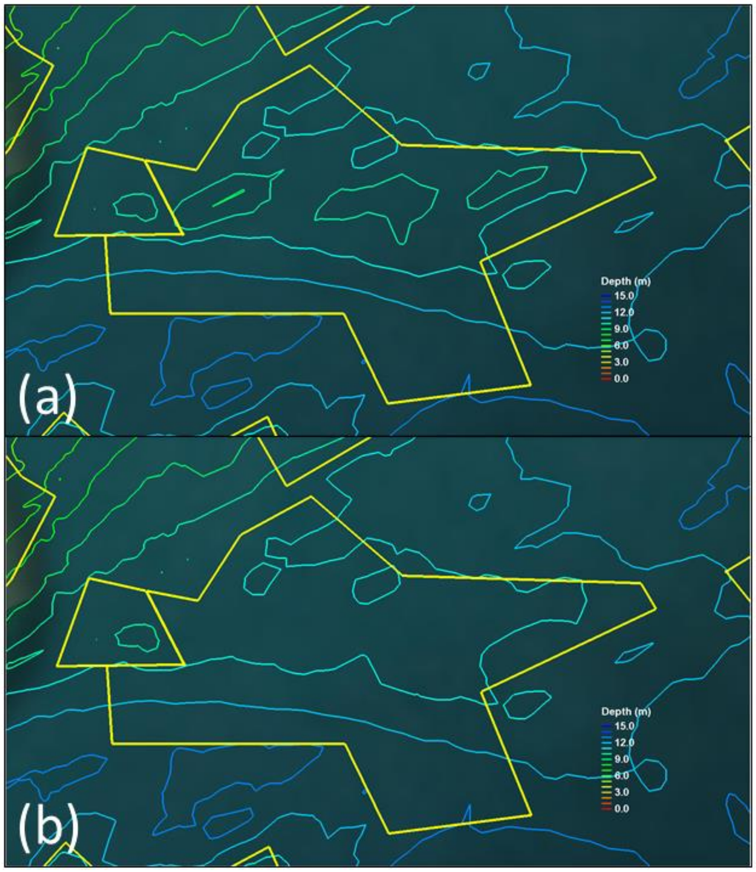

Figure 3-15. Bathymetry of Stono Inlet borrow area (area E). (a) base case (before dredge). (b) Alternative 5 (after dredge).

Five alternatives were described above. For each alternative, 2.5 MCY of sediment are required to be dredged from the respective borrow area. Accounting for the different size of the designated areas, the actual borrow site can be smaller to obtain required materials. Figure 1-1 shows both designated borrow areas and actual borrow areas for each alternative.

4 Simulation Results and Analysis

4.1 Model calibration and validation

Water surface elevation (WSE), current, and wave measurements in the Stono River channel and in the nearshore open area (AWAC#1 and AWAC#2) were used as the comparative data for the CMS calibration/validation (Figure 2-2). The calibration/validation period is from 21 November to 25 December 2019. Water surface elevation, wind, and wave data were used as input and model results were compared with the AWAC data. The input data were obtained from NOAA ocean buoys, NOAA coastal gages, and a WIS station (see Chapter 2). Calibration procedures included the examination of boundary conditions and tuning of adjustable model parameters, such as bottom friction, wall friction, tidal prism and so on.

For this coastal and estuarine application, spatially varying Manning's *n* values were specified in the model domain. In the main river channels and in shallow coastal area Manning's *n* was 0.017. The values were increased to 0.025 moving from nearshore inlet to offshore areas. Considering the extent of tidal marsh vegetation, Manning's *n* values of 0.025 and 0.03 were used for wetlands in the estuary. Along river banks and coastal line, the default wall friction in the CMS was turned off. In addition, single-grain size sediment transport modeling was conducted in the study. Referring to the sediment grab sample analysis (Figure 2-6), the sediment transport grain size, D50, was set to 0.18 mm.

Four radiation open boundaries were specified in the model (Figure 3-2). The offshore open boundary was driven by water surface elevation. The Stono River, Kiawah River, and Schooner Creek open boundaries had been assigned a zero value. With spatially varying Manning coefficients, the settings of the other parameters, and specifications of boundary forcing, final hydrodynamic and wave calibration and validation results were obtained.

In order to quantitatively demonstrate model skill in the calibration and validation process, goodness-of-fit statistics were calculated for water levels, current velocities, and wave parameters, which included the calculation of the correlation coefficient (R), Root-Mean-Square Error (RMSE), and Normalized Root-Mean-Square Error (NRMSE). The correlation coefficient R measures the linear co-variation between two datasets and can range from -1 to 1, with negative R values indicating inverse correlation and a value of 1 indicating perfect agreement. The RMSE measures the actual differences between the measured and calculated datasets, and the NRMSE is defined as RMSE/(data range) and measures the relative differences between the measured and calculated datasets.

Figure 4-1 shows the scatter plots of calculated and measured currents at AWAC#1 and AWAC#2. U is the east-west velocity component and V the north-south velocity component. The principle current direction represents the along-channel flow at AWAC#1 and the longshore current at AWAC#2. The positive V values at AWAC#1 indicate the flood tidal current and the negative indicate the ebb tidal current. Both the calculated and measured principle current axes have an angle of about -60° relative to north (Figure 4-1a).

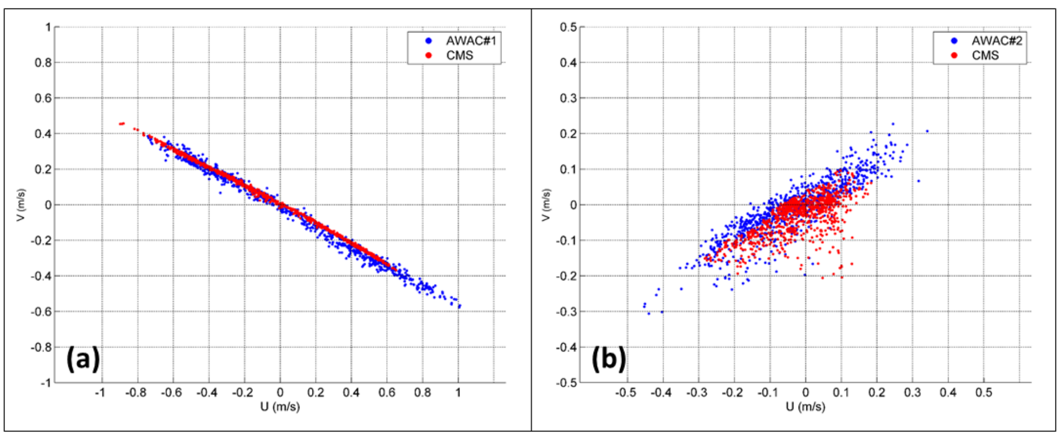

Figure 4-1. Scatter plots of the calculated and measured currents at (a) AWAC#1 and (b) AWAC#2 from 21 November to 25 December 2019.

The positive U and V values at AWAC#2 indicate water flowing along the shoreline in the northeast direction. Relative to north, both the calculated and measured principle current axes have an angle of about 60°. At this open ocean site, although still small, the cross-shore current is relatively strong comparing to longshore current component. The scatter plot shows similar distribution pattern of currents (Figure 4-1b).

The velocity scatter figure clearly displays the principle current directions, along the river channel at AWAC#1 and parallel to the coastline at AWAC#2. Both the calculated and the measured currents are rotated to this direction for model current calibration/validation at these two locations.

The current comparisons in the principle directions between the CMS results and the measurements are shown in Figure 4-2. At AWAC#1 the tidal signal is predominant with the measured currents ranging from -1.0 (ebb current) to 0.8 m/sec (flood current) in the along-channel direction. The asymmetry in the tidal current indicates that the estuarine system is ebb-dominated. The CMS calculated a consistent flood current (0.8-1.0 m/s) but under-predicted the ebb current (-0.7 m/s) (Figure 4-2a).

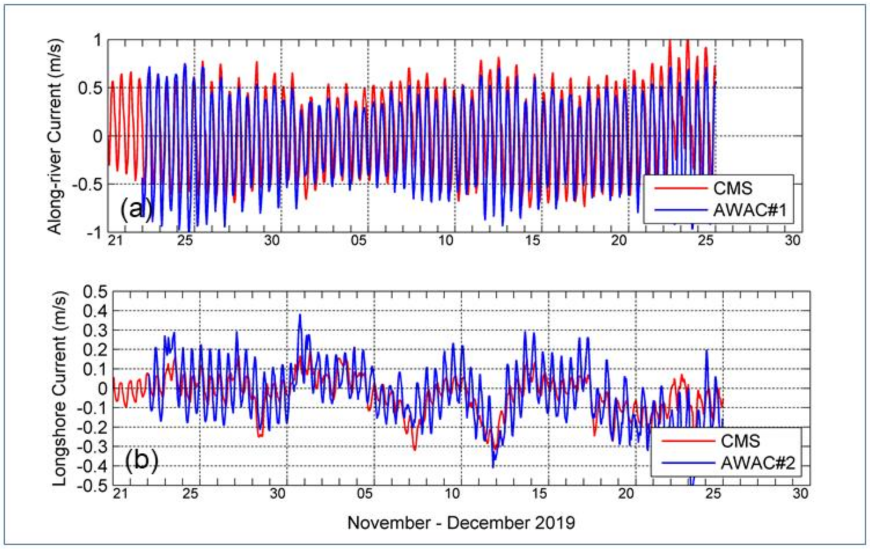

Figure 4-2. Current comparisons between the measurements and the CMS calculations at (a) AWAC#1 and (b) AWAC#2 from 21 November to 25 December 2019).

At AWAC#2, tidal currents were clearly overlapping with low-frequency currents during the simulation period, which was mostly due to wind effect. Using the wind at the nearby NOAA buoy to drive the model, the low-frequency longshore current variations were reproduced very well. The calculated tidal current phases matched well with the measurements, but the tidal current speeds were underestimated (Figure 4-2b).

Table 4-1 and Table 4-2 list the Goodness of fit statistics between the measurements and the model calculations at AWAC#1 and AWAC#2, respectively. Overall the calculated principle currents are in good agreement with the measured currents. At AWAC#1, the RMSE is 0.29 m/sec, the NRMSE is 16.3%, and the correlation coefficient R between the model and data is 0.8. At AWAC#2, the RMSE is 0.11 m/sec, the NRMSE is 14.3%, and the correlation coefficient R between the model and data is 0.66. Major discrepancies between the calculated results and measured data could be due to the limited spatial coverage of wind and wave data, and accuracy and resolution of wetland topography and bathymetry.

Table 4-1. Goodness of fit statistics between the AWAC#1 measurements and the CMS calculations from 21 November to 25 December 2019.

Variable	Goodness of Fit Statistics		
	R	RMSE	NRMSE (%)
Water Surface Elevation (m)	0.963	0.169	6.8
Along-channel Current (m/s)	0.804	0.294	16.3

Table 4-2. Goodness of fit statistics between the AWAC#2 measurements and the CMS calculations from 21 November to 25 December 2019.

Variable	Goodness of Fit Statistics		
	R	RMSE	NRMSE (%)
Water Surface Elevation (m)	0.969	0.151	6.0
Longshore Current (m/s)	0.663	0.114	14.3
Significant Wave Height (m)	0.858	0.329	12.2
Peak Wave Period (s)	0.506	2.052	13.3

Figure 4-3 shows the calculated and measured WSEs at the AWAC gauges. Both the measurements and calculations show that the spring tidal range is close to 2.5 m and the neap tidal range is about 1.5 m. Visual inspection indicates that the CMS results reproduce the tidal signals displayed in the river channel and the open coastal area very well. The RMSEs at the two gauge locations are around 0.15 m and the NRMSE 6.0%. The correlation coefficient R between the model and data is greater than 0.96 (Table 4-1 and Table 4-2).

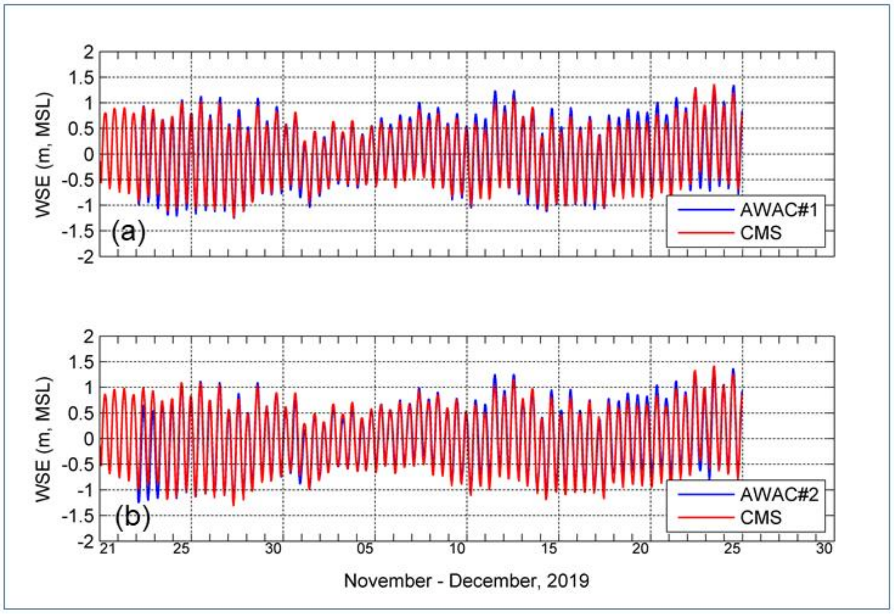

Figure 4-3. Comparisons of water surface elevation between the measurements and the CMS calculations at (a) AWAC#1 and (b) AWAC#2 from 21 November to 25 December 2019.

AWAC#1 was located in the river channel and wave impact was insignificant. Average significant wave height over the simulation period is less than 0. 1 m. Therefore, only the wave parameters at AWAC#2 were used to calibrate/validate the CMS. The comparisons of the measured and calculated wave parameters are shown in Figure 4-4. The measured and calculated mean significant wave heights at this location are 0.70 and 0.85 m, respectively. There were a few occasions when the measured wave heights were close or greater than 1.5 m during the 35-day period. While the calculated wave heights show corresponding peaks, the values were generally overestimated. Examining wind conditions in Figure 4-4, it can be seen that those high wave conditions are well correlated with weather (storm) events.

Both the measured and calculated mean wave period is 8.2 sec, and the predominant wave direction is southeast. The correlation coefficients are 0.86 and 0.51 for wave height and wave period, respectively. The RMSE and the NRMSE are 0.33 m and 12.2% for wave height, 2.05 sec and 13.3% for wave period, respectively (Table 4-2). The sensitivity tests on wave transformation show that the calculated wave parameters are closely associated with the specifications of boundary conditions. Close to this study area, the only offshore buoy that provides directional wave spectra is NOAA Buoy #41004. Other options are to utilize wave parameter generated spectra from nearby sites of hindcast products, such USACE WIS and NOAA

Wave Watch III (WWIII). Data limitation might hinder the model performance in reproducing better wave simulation results. It is also noted in Figure 4-4 that relatively large discrepancies between measured and calculated wave periods occur when wave heights are usually smaller than 0.5 m and the noise exists in the measuremens of wave direction. Therefore the model computational error is also due to instrument accuracy and stability.

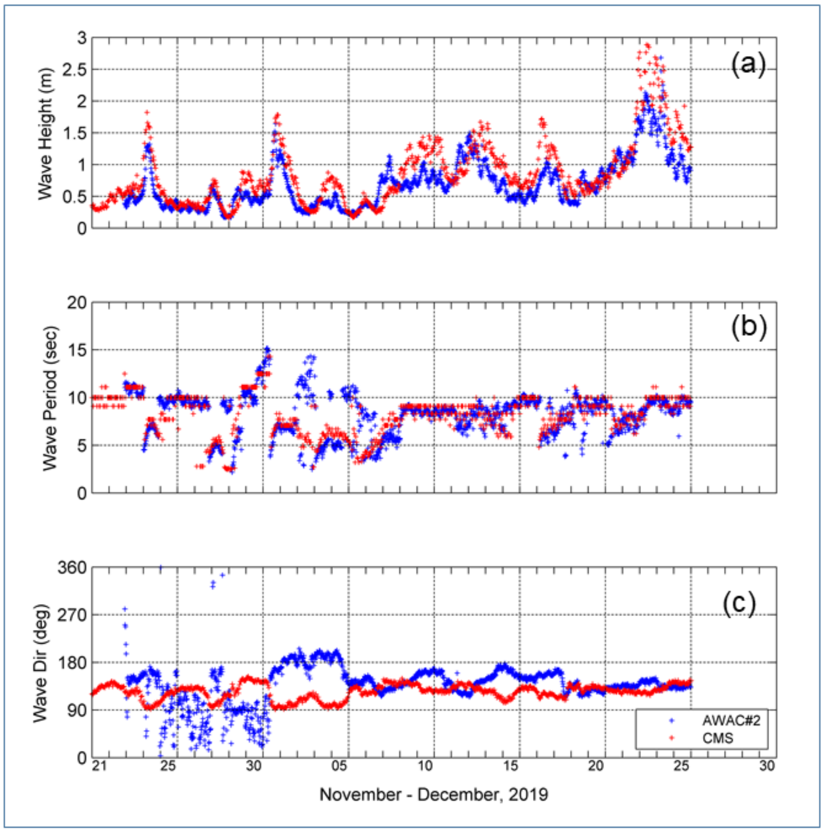

Figure 4-4. Comparisons of wave parameters between the measurements and the CMS calculations at AWAC#2 from 21 November to 25 December 2019. (a) Significant wave height, (b) Peak wave period, and (c) Mean wave direction.

4.2 Hurricane Hugo

Waves

As shown in Figure 3-7, Hurricane Hugo brought high waves up to 6.0 m to the coast. The maximum wave heights in the study area occurred on 22 September, 1989 at 05:00 (GMT). Spatial distribution of significant wave heights and wave directions is shown in Figure 4-5 and the calculated results correspond to the peak wave period of the storm. The figure clearly displays that hurricane waves propagated from south-southeast and refracted approaching to shoreline. Significant wave refraction can be seen around Kiawah Island and the Stono Inlet Throat borrow area because of the protruding shoreline of the island and sudden water depth decrease within the borrow area. Significant wave heights have a value between 5.0 and 5.6 m in the offshore area. Close to shoreline wave heights are reduced but still have a value between 2.2 and 3.0 m due to water depth increase related to hurricane-induced storm surge. Wave heights are further reduced to 0.5 - 1.0 m at the confluent region of Stono, Folly Rivers, and ocean, and are smaller than 0.3 m into the rivers.

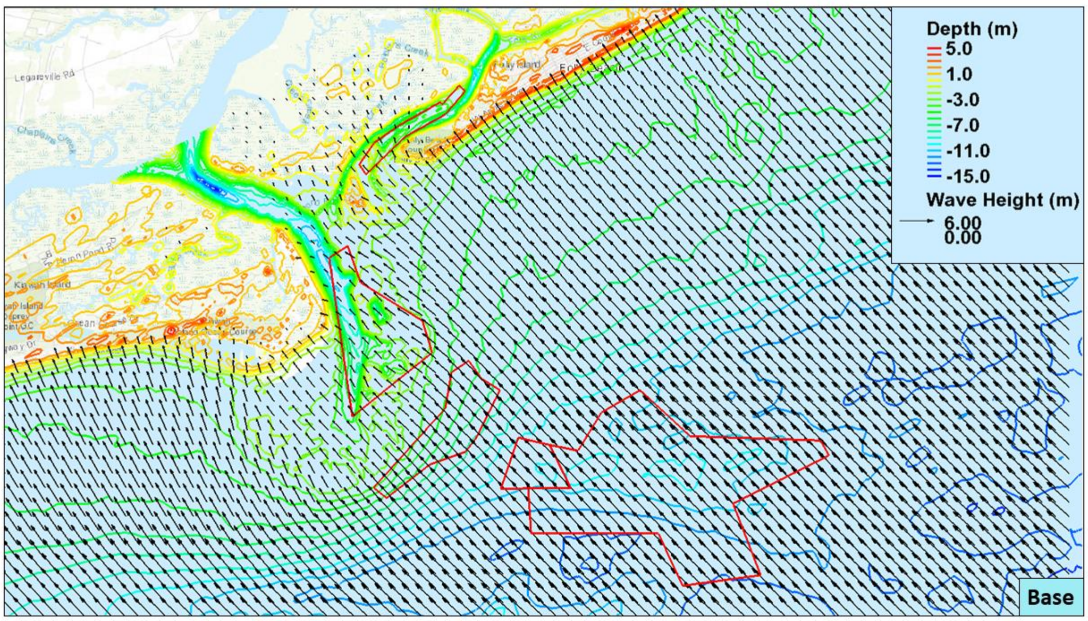

Figure 4-5. Calculated maximum significant wave heights during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

Changes in significant wave heights before (base) and after (alternatives) dredge are examined within the five sand borrow areas in Figure 4-6 to Figure 4-10, respectively. Detail value comparisons are also listed in Table 4-3.

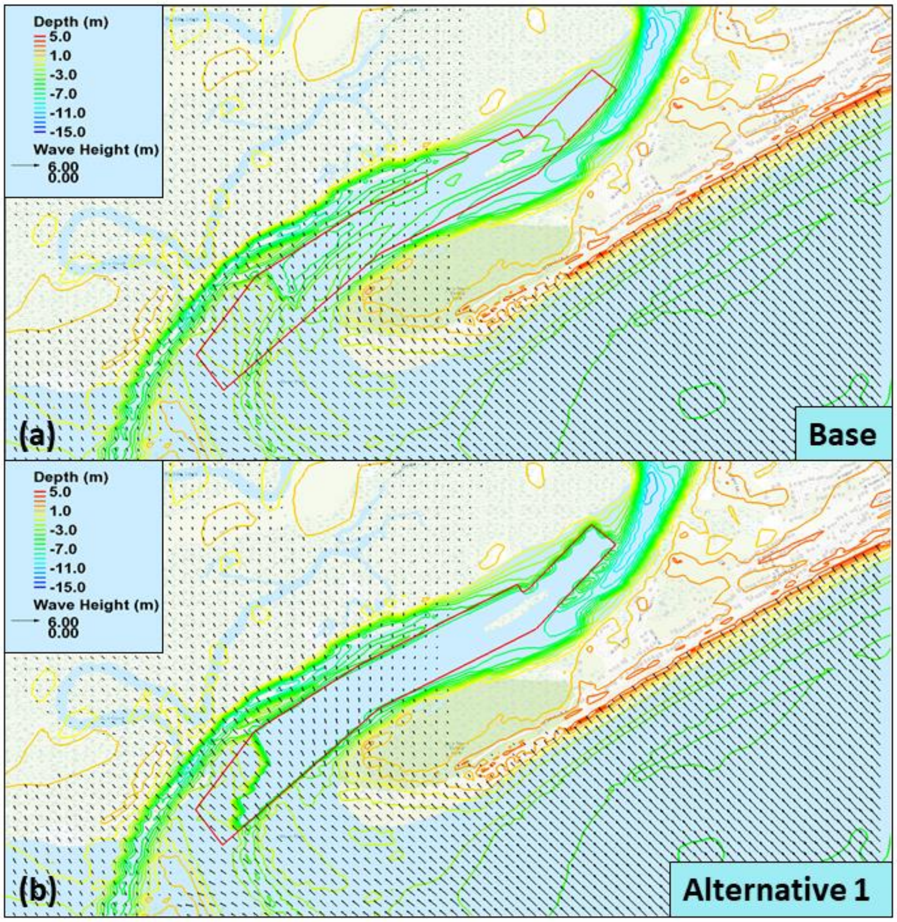

Figure 4-6. Comparison of significant wave heights between (a) base case and (b) Alternative 1 within the Folly River borrow area during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

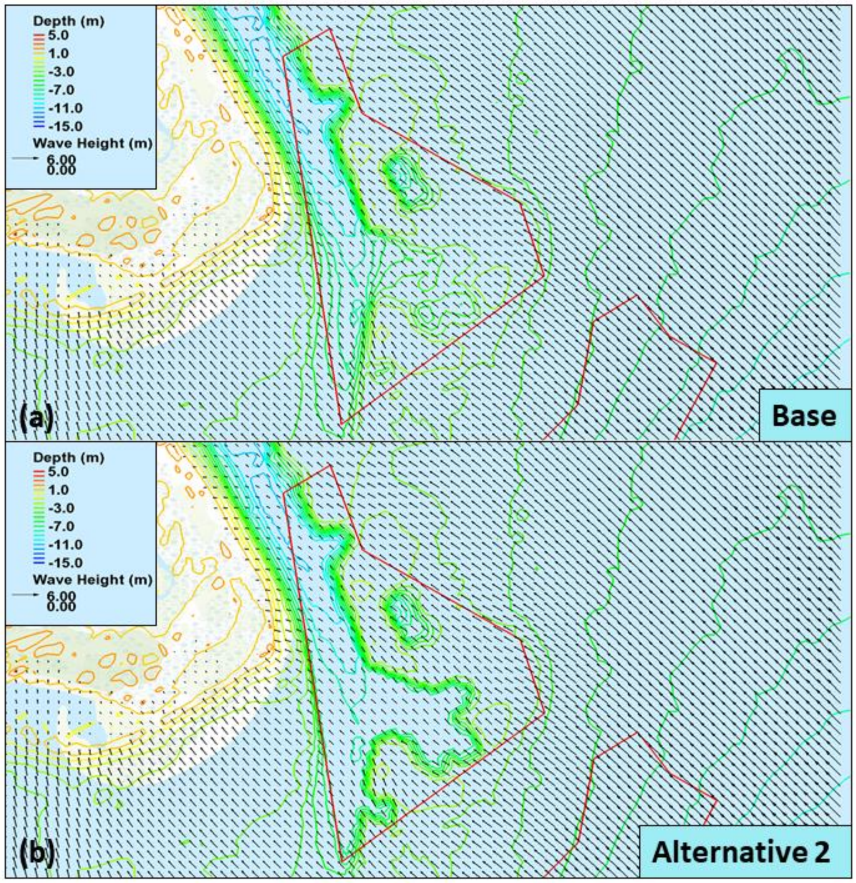

Figure 4-7. Comparison of significant wave heights between (a) base case and (b) Alternative 2 within the Stono Inlet Throat borrow area during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

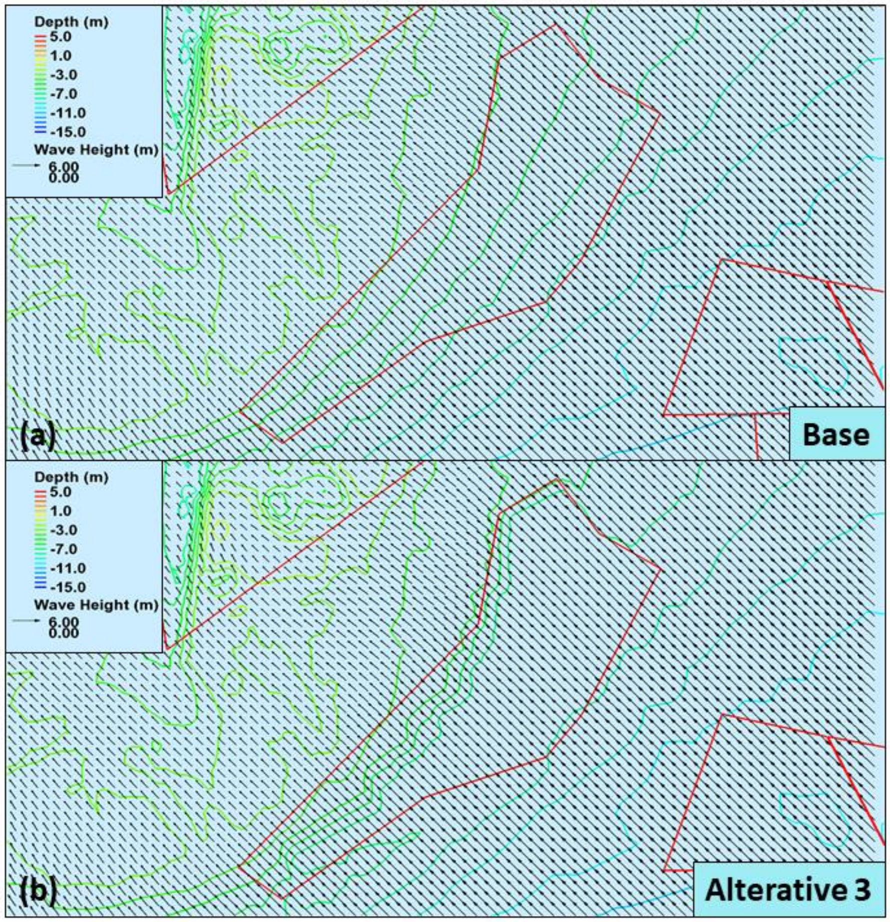

Figure 4-8. Comparison of significant wave heights between (a) base case and (b) Alternative 3 within the Stono Ebb Shoal 1 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

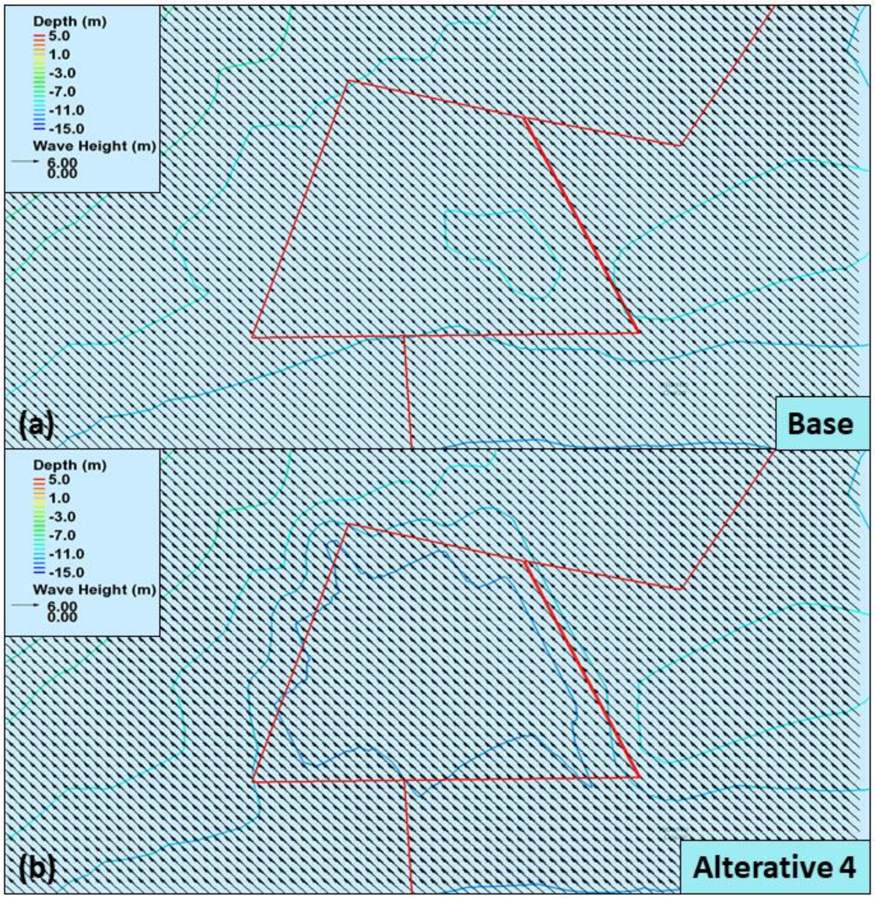

Figure 4-9. Comparison of significant wave heights between (a) base case and (b) Alternative 4 within the Stono Ebb Shoal 2 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

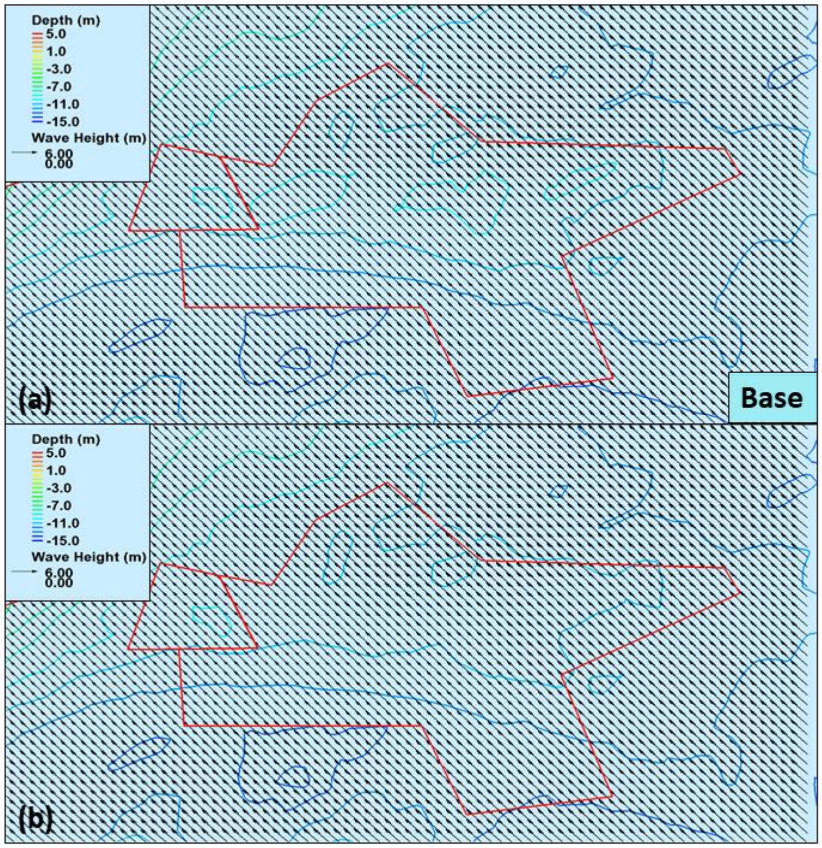

Figure 4-10. Comparison of significant wave heights between (a) base case and (b) Alternative 5 within the Stono Inlet borrow area during the Hurricane Hugo Passage on 22 September 1989 at 05:00 (GMT).

Table 4-3. Comparisons of significant wave heights between the base case (before sand dredge) and alternatives (after sand dredge) in borrow areas during Hurricane Hugo.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Significant Wave Height (m)	1.0-1.6	1.3-1.7	1.5-2.8	1.4-2.6	3.8	4.4	5.1-5.2	5.2-5.3	5.1-5.5	5.1-5.5

The Folly River borrow area has an original average depth of 2.56 m (8.53 ft) relative to MSL. In order to obtain 2.5 MCY of sand materials with a target depth of 7.85 m (25.76 ft), it does not need to dredge the entire area and therefore no sand materials are dredged at the southwest corner of the area for the Alternative 1 simulation (Figure 1-1 and Figure 3-11). Corresponding to this setup, Figure 4-6 does not show much change in significant wave heights before (base) and after (Alternative 1) the dredge over the southwest portion but does show slightly higher waves propagating over the dredged portion of borrow area.

In order to dredge to a target depth of 7.55 m (24.77 ft) in the Stono Inlet Throat borrow area (Alternative 2), only a small portion of the area off the Folly Rive Navigation Channel needs to be dredged (Figure 1-1 and Figure 3-12). Generally, significant wave heights were larger in the base simulation than those after dredging. But the east part of the borrow area was not dredged and relatively shallow, over which significant wave heights were larger after dredging than those in the base simulation due to stronger wave refraction related to deepening on the west side of the area (Figure 4-7).

In the Stono Ebb Shoal 1 borrow area, the average water depth is 5.94 m (19.49 ft) and the targeted dredge depth is 7.55 m (24.77 ft). After sand removal higher offshore waves propagated in the area with less dissipation (Figure 4-8). Significant wave heights were increased by more than 0.5 m in the actual borrow area (Figure 1-1 and Table 4-3).

The Stono Ebb Shoal 2 borrow area has an original water depth of 10.28 m (33.73 ft). To obtain 2.5 MCY of sand materials, the target dredge depth was set to 12.0 m (39.37 ft). The large water depths before and after sand removal did not affect significant wave heights traveling over the area (Figure 4-9 and Table 4-3).

Similar to the Stono Ebb Shoal 2 borrow area, the Stono Inlet borrow area has a large base water depth of 11.19 m (36.71 ft). Due to the size of the area, it does not require much adjustment in water depth. After sand removal the average water depth of the area was only increased to 11.36 m (37.27 ft). Therefore, there was no significant impact on wave propagation due to water depth changes over the area (Figure 4-10 and Table 4-3).

Current

During the passage of Hurricane Hugo, storm surge coninciding with flood tide generated extreme currents in the study area on 22 September, 1989 at 03:00 (GMT). Spatial distribution of the currents is shown in Figure 4-11, which illustrates offshore and nearshore current patterns around the Stono Inlet estuarine system. The strongest currents with a speed close to 3.0 m/s occurred in the Stono River channel. The nearshore zone in front of Folly Beach shows strong longshore current flowing from northeast to southwest. The longshore flow had a current speed between 1.0 and 1.5 m but intensified with a speed of more than 2.0 m/s when making a turn towards northwest into the Folly River channel area around the southwest corner of Folly Island. In the offshore area, the current field shows a general flow direction towards land with a small current speed of 0.3 m/s.

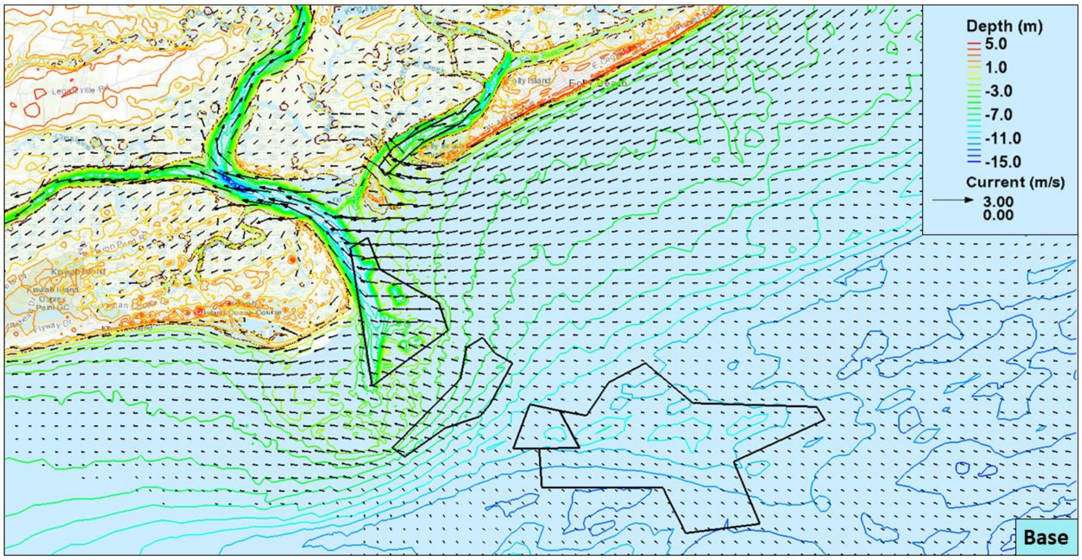

Figure 4-11. Calculated currents during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

Impact on current changes due to sand removal from each of the borrow areas is evaluated in Figure 4-12 to Figure 4-16, respectively. Detail value comparisons are listed in Table 4-4.

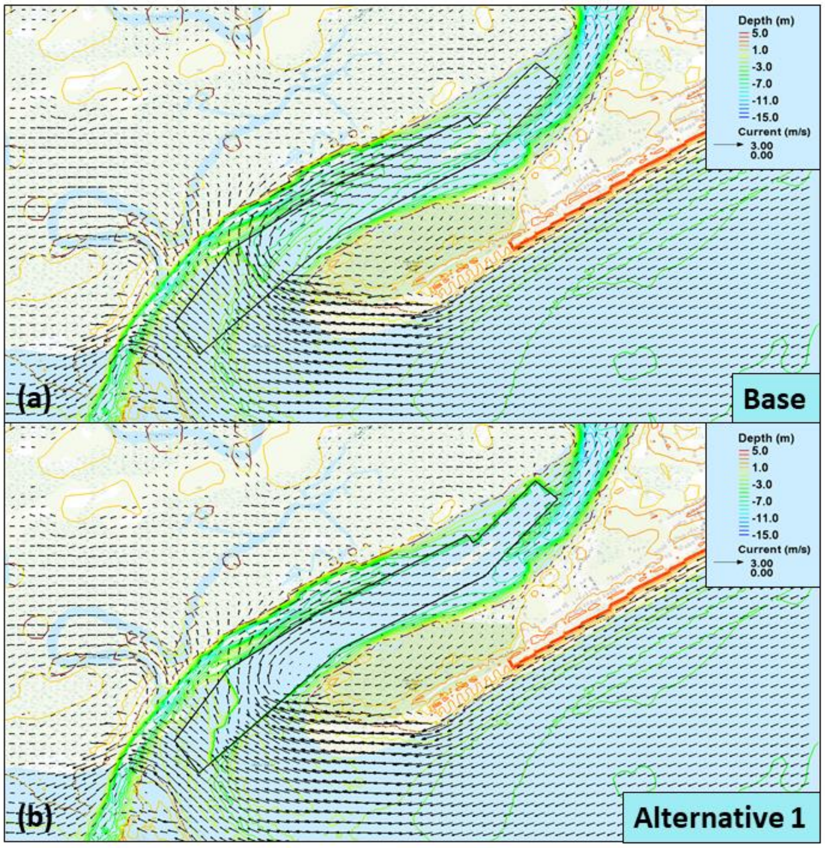

Figure 4-12. Comparison of currents between (a) base case and (b) Alternative 1 within the Folly River borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

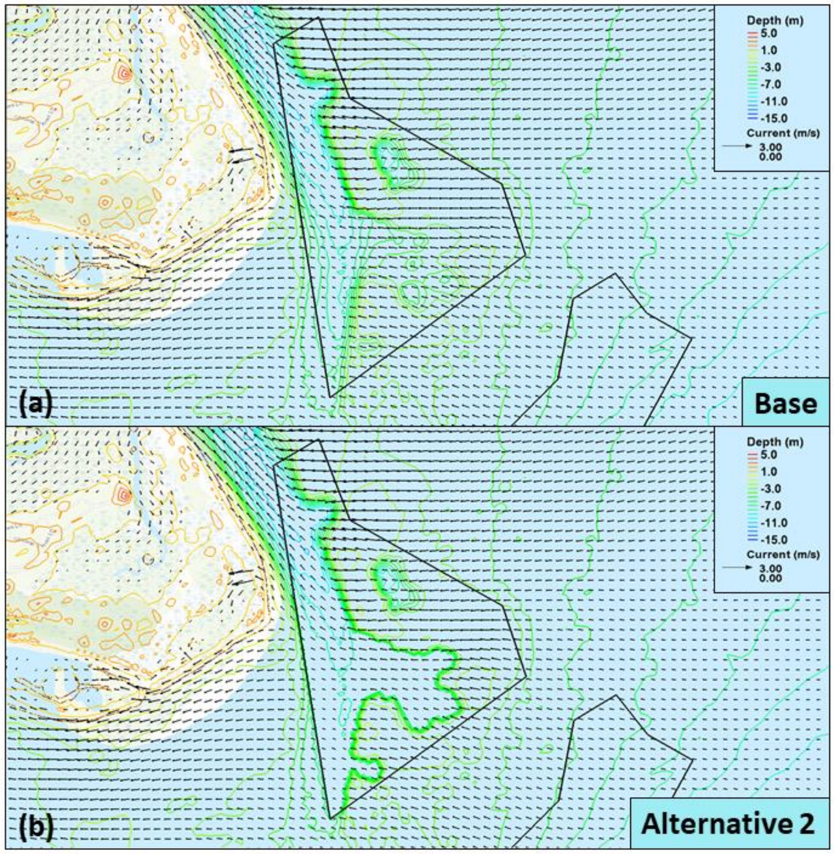

Figure 4-13. Comparison of currents between (a) base case and (b) Alternative 2 within the Stono Inlet Throat borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

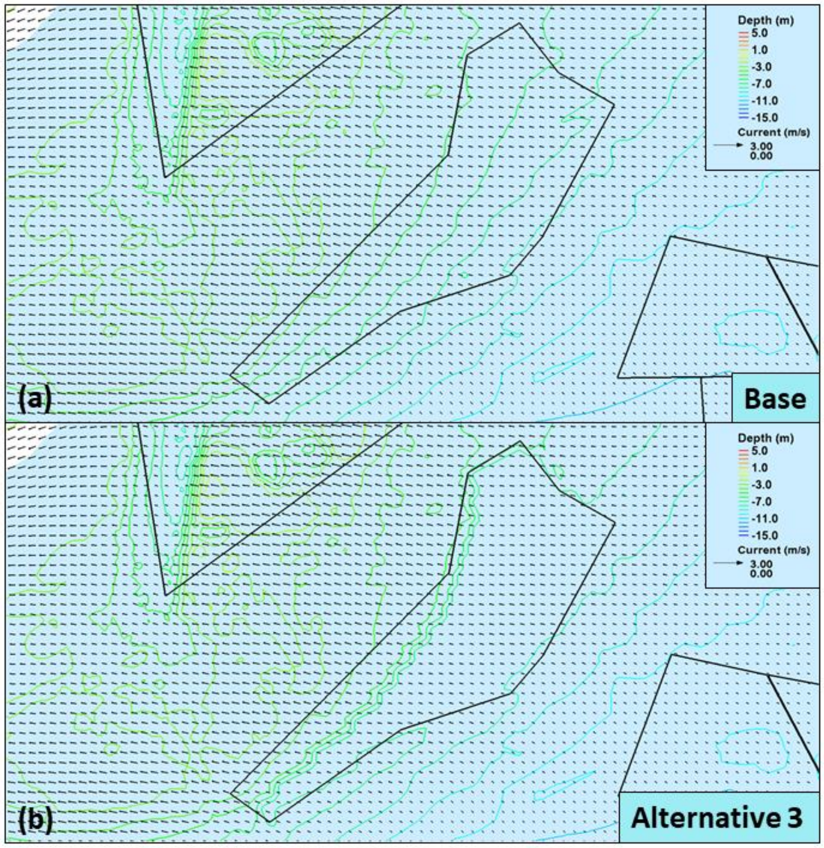

Figure 4-14. Comparison of currents between (a) base case and (b) Alternative 3 within the Stono Ebb Shoal 1 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

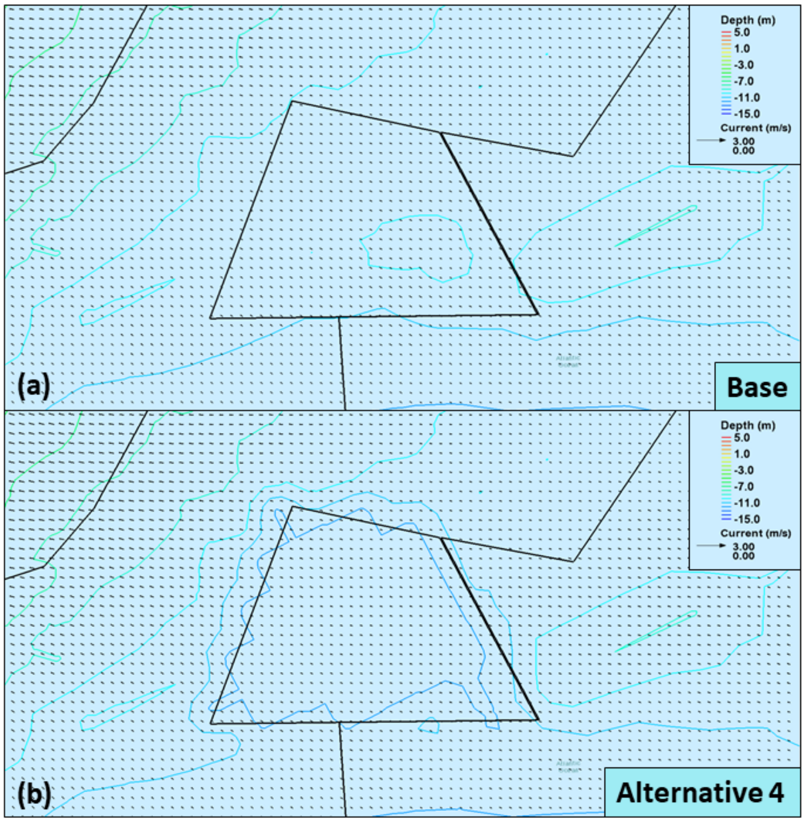

Figure 4-15. Comparison of currents between (a) base case and (b) Alternative 4 within the Stono Ebb Shoal 2 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

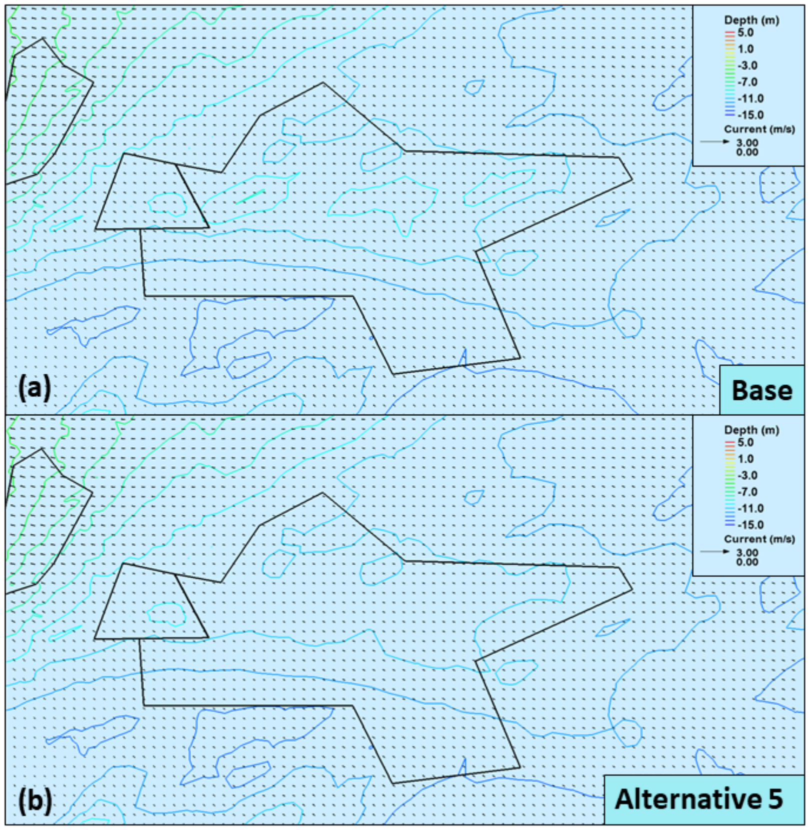

Figure 4-16. Comparison of currents between (a) base case and (b) Alternative 5 within the Stono Inlet borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

Table 4-4. Comparisons of currents between the base case (before sand dredge) and alternatives (after sand dredge) in borrow areas during Hurricane Hugo.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Current (m/s)	0.96-1.66	0.91-1.33	0.85-1.01	0.66-0.98	0.43-0.69	0.4-0.61	0.28	0.28	0.28	0.25

The current differences associated with sand removal can be seen in the central and southwest portions of the Folly River borrow area. For the base case the longshore current turning around the southwest tip of Folly Island entered the borrow area and retained the strength because of shallow water depth there. The current continued to flow north across the borrow area. After entering the deeper river channel, a small branch flew towards the Stono River in the southwest and the major branch went in the northeast direction towards the upstream of the Folly River. For the alternative the longshore current lost the strength after entering the dredged borrow area and turned to the upstream of the Folly River inside the borrow area before reaching the river channel (Figure 4-12 and Table 4-4). The maximum changes in current speed due to sand removal occurred when the longshore current flew into the borrow area, which was decreased from 1.66 m/s for the base case to 1.08 m/s for the alternative.

In the Stono Inlet Throat borrow area (Alternative 2), the main channel area and a large part of the shoaling area were not dredged. The former has an average depth of 8.47 m (27.79 ft) and the latter 2.46 m (8.07 ft). While the sand removal from the small off-channel area did not change the current pattern in the entire borrow area, it did reduce current speed by about 0.20 m/s in the actual dredged site (Figure 1-1 and Figure 4-13), which was also related to the decrease of current speed by about 0.10 m/s in the main channel.

The actual area with sand removal in the Stono Ebb Shoal 1 borrow area has an average change in water depth from 6.35 m (20.83 ft) to 7.55 m (24.77 ft). As shown in Figure 4-14 and Table 4-4 the change was causing a corresponding decrease in current speed from 0.56 m/s to 0.40 m/s over the sand removal area.

Both the Stono Ebb Shoal 2 and the Stono Inlet borrow areas are located in the offshore area and have a base water depth of more than 10.0 m (32.81 ft). The large water depths did not change current speeds in those two areas (Figure 4-15 and Figure 4-16). The speed values in Table 4-4 show a maximum change in current speed by about 0.03 m/s before and after sand removal.

Sediment transport

Corresponding to wave and current analysis in the previous section, sediment transport rates were calculated. Resulting from sand movement, and bed erosion and deposition, morphology (bed volume) changes were examined around each borrow area for the Hurricane Hugo and 2018 periods.

Figure 4-17 shows the spatial distribution of sediment transport rates in the study area on 22 September, 1989 at 03:00 (GMT). Comparing with the current distribution in Figure 4-11, it can be seen that the sediment transport pattern is consistent with the current pattern around the Stono Inlet estuarine system. The strongest transport rates occurred in the Stono River channel with a magnitude between 110 and 120 kg/(m·s). Nearshore in front of Folly Beach sediment moved in the longshore direction from northeast to southwest. The longshore transport rate with a value between 10 and 20 kg/(m·s) was one order of magnitude smaller than that in the Stono River channel but doubled with a transport rate between 25 and 30 kg/(m·s) as turning around the southwest corner of Folly Island. In the offshore area, the sediment transport rate is generally small with a magnitude less than 5 kg/(m·s).

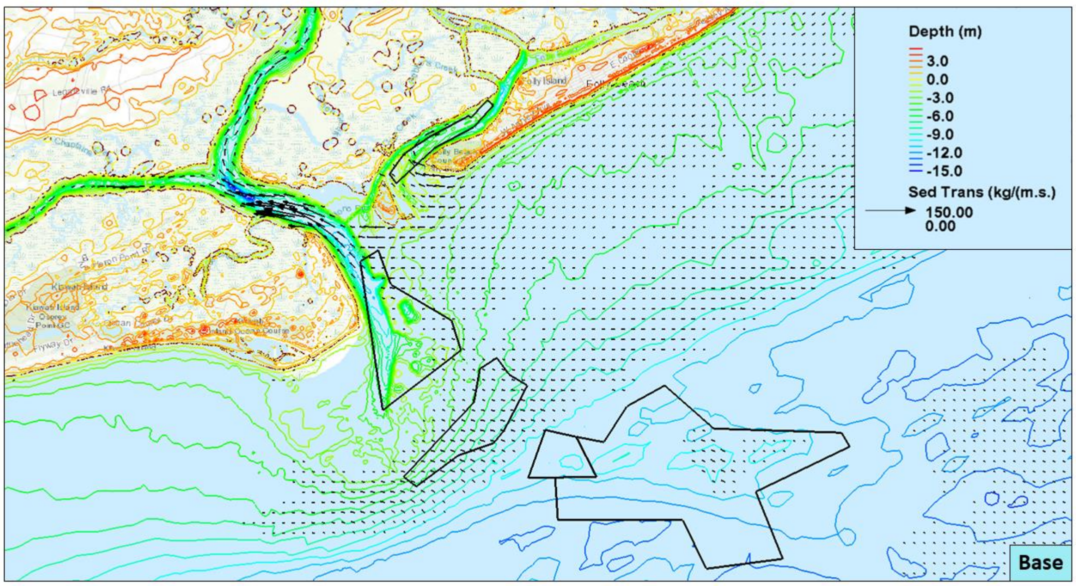

Figure 4-17. Calculated sediment transport rates during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

Comparisons of sediment transport rates before (base) and after (alternatives) sand removal from each of the borrow areas are shown in Figure 4-18 to Figure 4-22, respectively. Detail transport values are listed in Table 4-5.

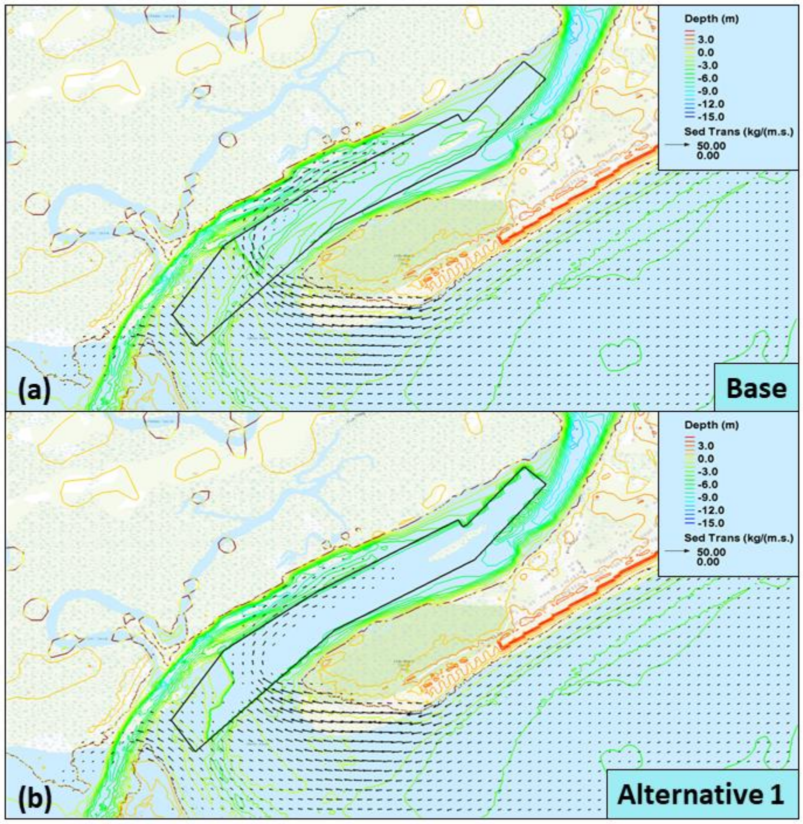

Figure 4-18. Comparison of sediment transport rates between (a) base case and (b) Alternative 1 within the Folly River borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

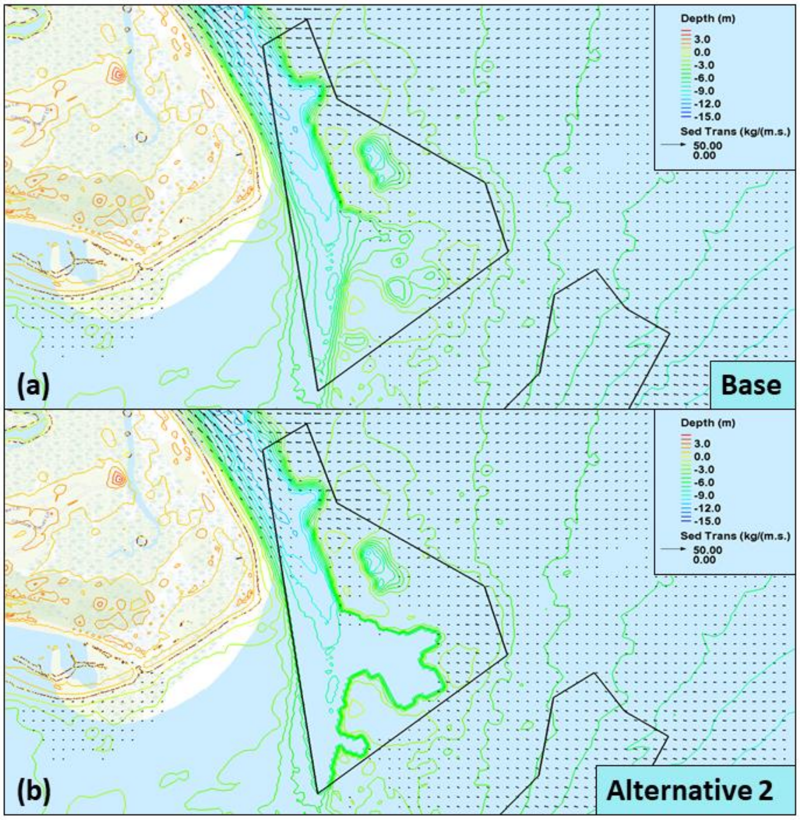

Figure 4-19. Comparison of sediment transport rates between (a) base case and (b) Alternative 2 within the Stono Inlet Throat borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

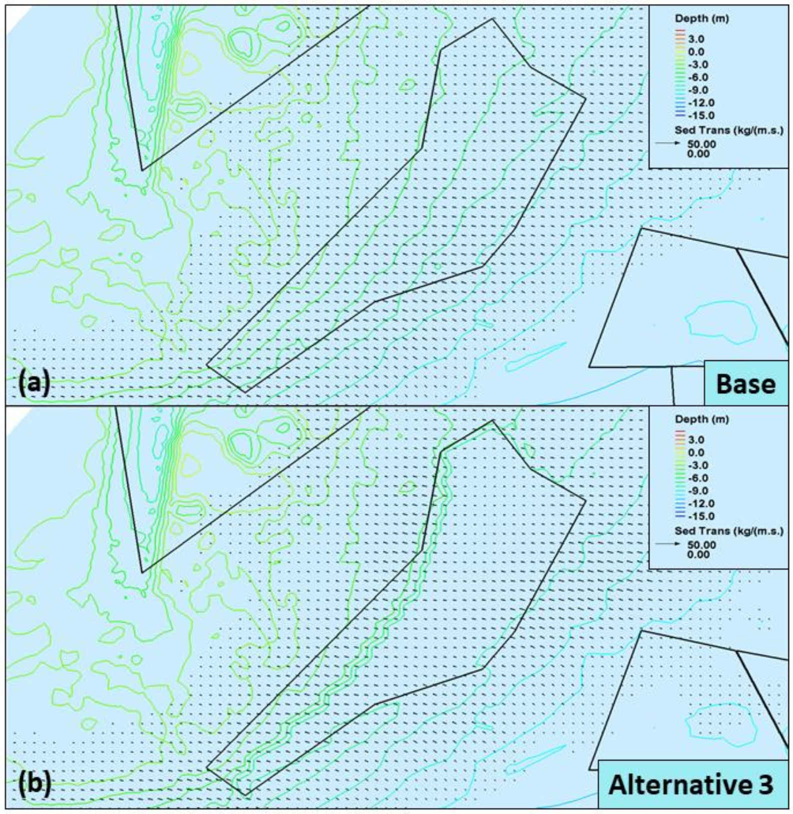

Figure 4-20. Comparison of sediment transport rates between (a) base case and (b) Alternative 3 within the Stono Ebb Shoal 1 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

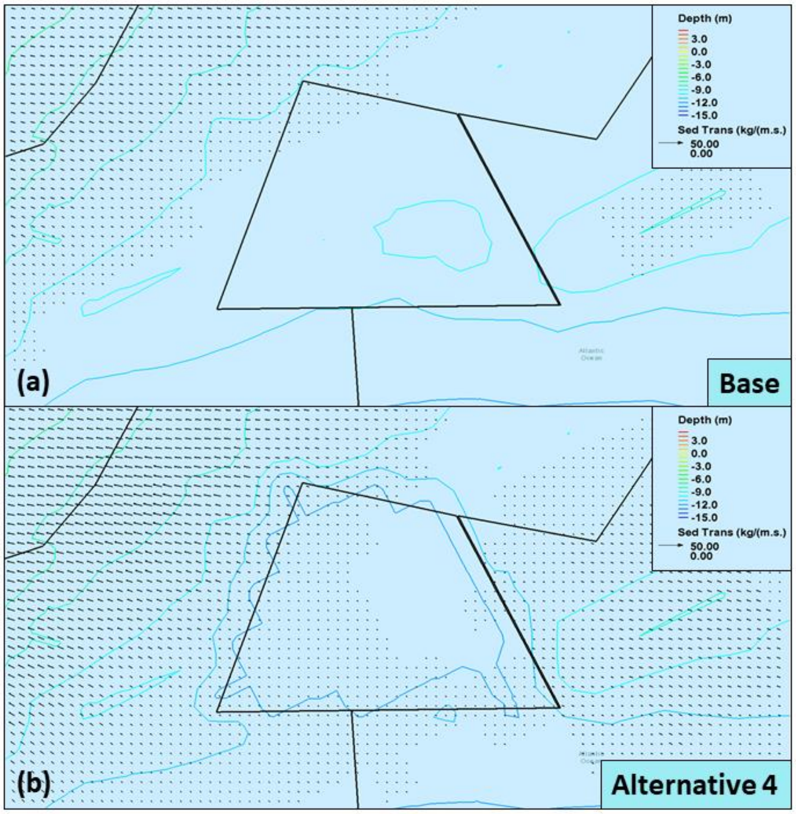

Figure 4-21. Comparison of sediment transport rates between (a) base case and (b) Alternative 4 within the Stono Ebb Shoal 2 borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

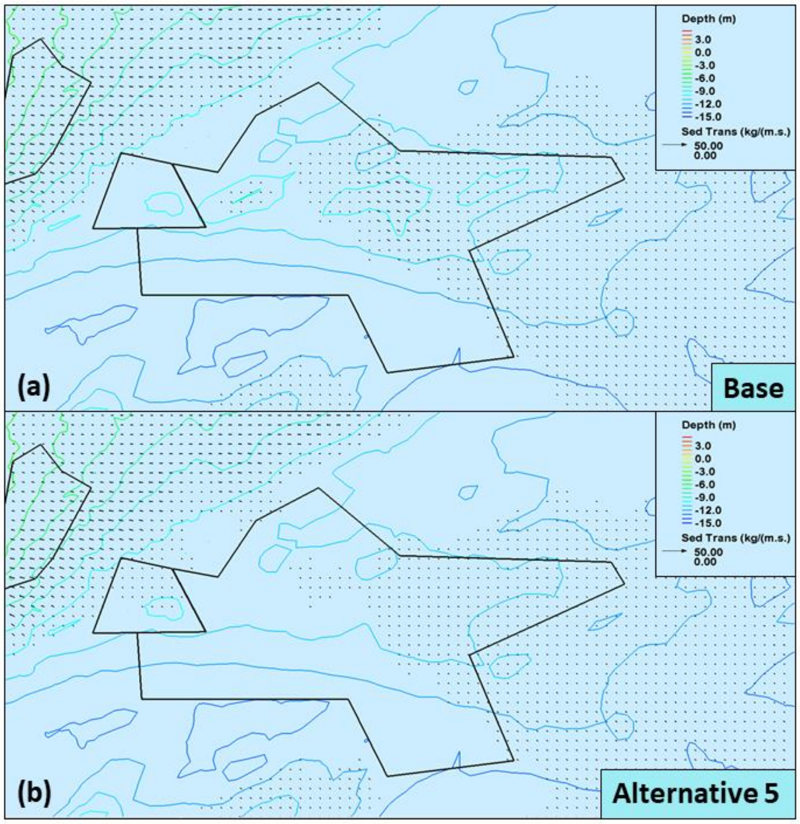

Figure 4-22. Comparison of sediment transport rates between (a) base case and (b) Alternative 5 within the Stono Inlet borrow area during the Hurricane Hugo Passage on 22 September 1989 at 03:00 (GMT).

Table 4-5. Comparisons of sediment transport rates between the base case (before sand dredge) and alternatives (after sand dredge) in borrow areas during Hurricane Hugo.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Sediment Transport Rate (kg/(m.s))	19.7-23.3	11.0-21.4	3.4-7.3	1.6-7.5	7.7-10.3	4.6-10.1	2.6	2.8-3.3	2.7-9.1	2.8-3.4

For the Folly River borrow area, the vector distribution of sediment transport rates is shown in Figure 4-18. The differences of sediment transport associated with sand removal can be compared with the current field in Figure 4-12. Similarly, the central and southwest portions of the area show major changes in sediment transport rates. For the base case the sediment transport turning around the southwest tip of Folly Island almost unchanged entering the borrow area (slightly increased from 19.7 to 20.0 kg/(m·s)). For the alternative sediment transport rate was reduced significantly (21.4 to 11.0 kg/(m·s)) due to water deepening and relatively large transport occurred in the middle section of the borrow area (Table 4-5).

In the Stono Inlet Throat borrow area, the undredged channel and the east part of the shoaling area show relatively large sediment transport rates around 9.0 and 7.5 kg/(m·s), respectively, which did not change both for the base and the alternative cases (Figure 4-19). The dredged small off-channel area shows small sediment transport rates, which were reduced from 3.4 to 1.5 kg/(m·s) from the base to the alternative cases (Table 4-5). The decrease of transport rates in the dredged area well corresponds to the decrease in currents as shown in Figure 4-13.

Corresponding to decreases in current speed over the sand removal area, sediment transport rates decreased from 7.7-10.3 kg/(m·s) to 4.6-10.1 kg/(m·s) for the base and the alternative case in the Stono Ebb Shoal 1, respectively (Table 4-5 and Figure 4-20). Like in the Folly River borrow area, sediment transport rates do not change much from the offshore to the dredged area for the base case, but decrease for the alternative, which results from depth gradient created by sand removal from the borrow area.

Because of the large water depths in the Stono Ebb Shoal 2 and the Stono Inlet borrow areas, changes in current speeds and sediment transport are insignificant comparing the base case with the alternative case (Figure 4-21 and Figure 4-22). Sediment transport rates listed in Table 4-5 are generally smaller than 5.0 kg/(m·s) in these two areas before and after sand removal.

Morphology Change

Sediment transport and morphology changes were calculated for the base case during the period when the Hurricane Hugo passed the study area. The calculated morphology changes were obtained by subtracting depth values at the end of simulation on 26 September at 00:00 from those at the beginning on 18 September 1989 at 00:00. The areas with positive values represent sediment accretion and negative values represent sediment erosion in Figure 4-23.

Major morphologic changes presented by the model results in Figure 4-23 occurred in the Stono River channel, nearby the Folly River borrow area between Folly Island and Bird Key, and around the Stono Inlet Throat borrow area off the Stono Inlet navigation channel. The maximum erosion and deposition in the Stono River had a magnitude between 2.0 to 3.0 m after the hurricane passage. The magnitude of erosion and deposition near Folly Island and at the Stono Inlet Throat area was between 1.0 to 1.5 m.

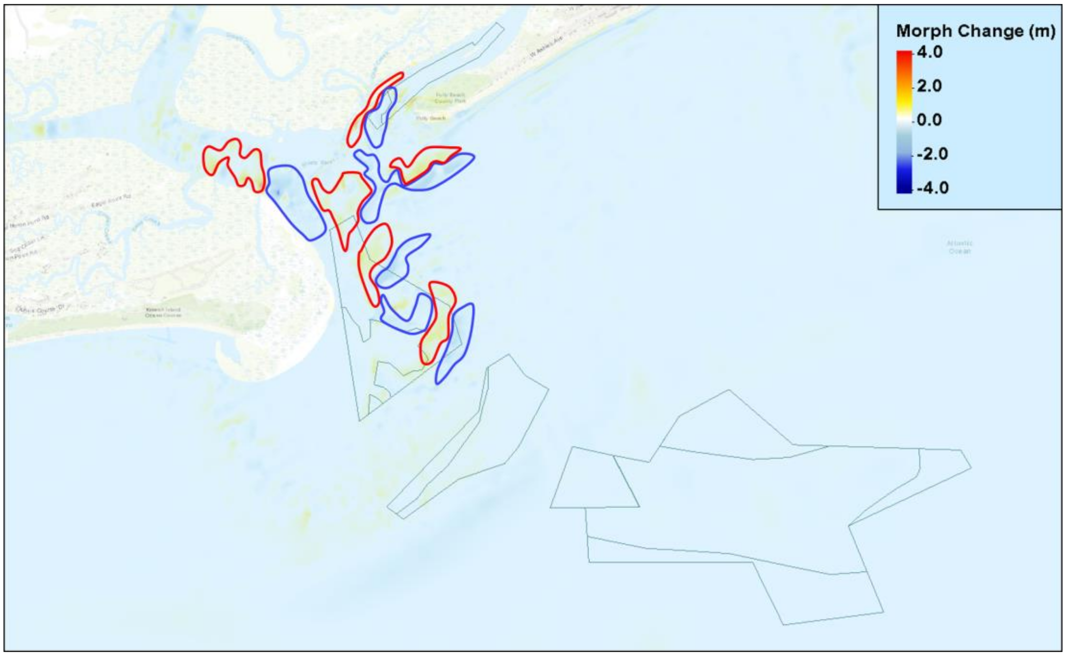

Figure 4-23. Morphology changes for the base case in the study area from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

Figure 4-24 shows four longshore erosion and deposition zones along Folly Beach, within which average depths, depth changes, and volume changes due to the hurricane passage are listed in Table 4-6. Erosion Zone 1 goes along the shoreline all the way from the southwest to the northeast and is located in the nearshore area of Folly Island with an average water depth of 0.89 m. This erosion zone has an average water depth change of 0.32 m and sand loss of 141000 cu yd due to the impact by Hurricane Hugo. Closer to the beach with an average water depth of 0.01 m above MSL is Erosion Zone 2 at the southwest end of Folly Beach, which experiences much less erosion than the deeper erosion zone. In between these two zones is Deposition Zone 2, which shows an average depth change of 0.17 m, and the total sand accumulation there is 17,262 cu yd. Deposition Zone 1 extends from the north end of Deposition Zone 2 to the northeast of Folly Beach. Because the average depth within this zone is 2.4 m above MSL, the calculated average deposition of 0.02 m indicates that extra sand materials are accumulated on this portion of the beach after the passage of Hurricane Hugo.

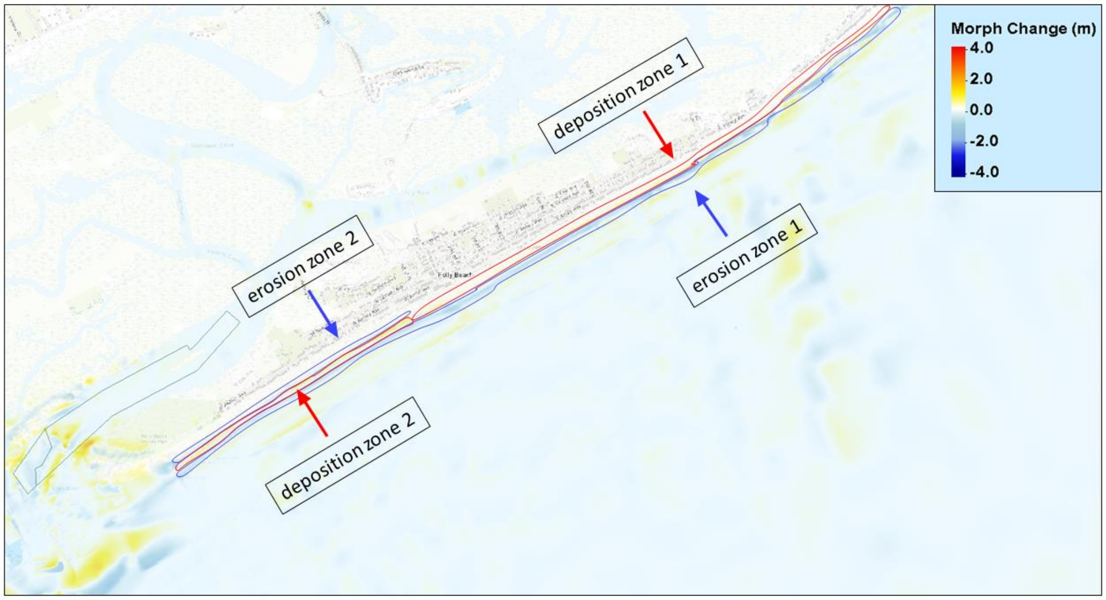

Figure 4-24. Morphology changes for the base case nearshore in front of Folly Beach from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

Table 4-6. Longshore erosion and deposition zones in front of Folly Beach during the passage of Hurricane Hugo in September 1989. Positive depths are the land elevation above MSL.

Variables	Erosion Zone 1	Erosion Zone 2	Deposition Zone 1	Deposition Zone 2
Average Depth Change (m)	0.32	0.07	0.02	0.17
Average Depth (m)	-0.89	0.01	2.4	-1.43
Volume Change (cu yd)	141000	6836	4536	17262

Comparisons of morphology changes before (base) and after (alternatives) sand removal from each of the borrow areas are shown in Figure 4-25 to Figure 4-29, respectively. The maximum erosion and deposition values are listed in Table 4-7.

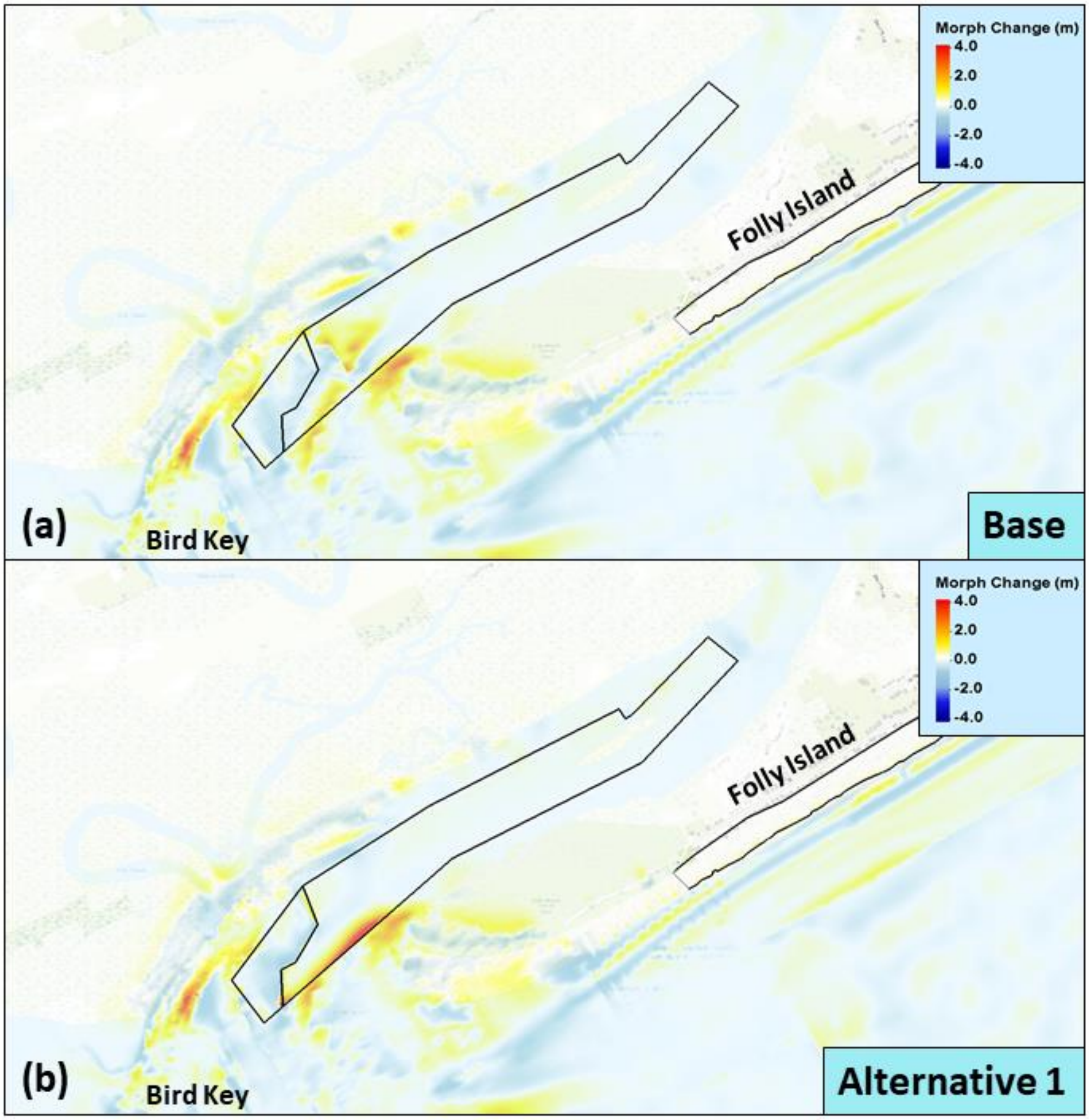

Figure 4-25. Comparison of morphology changes between (a) base case and (b) Alternative 1 within the Folly River borrow area during the Hurricane Hugo Passage from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

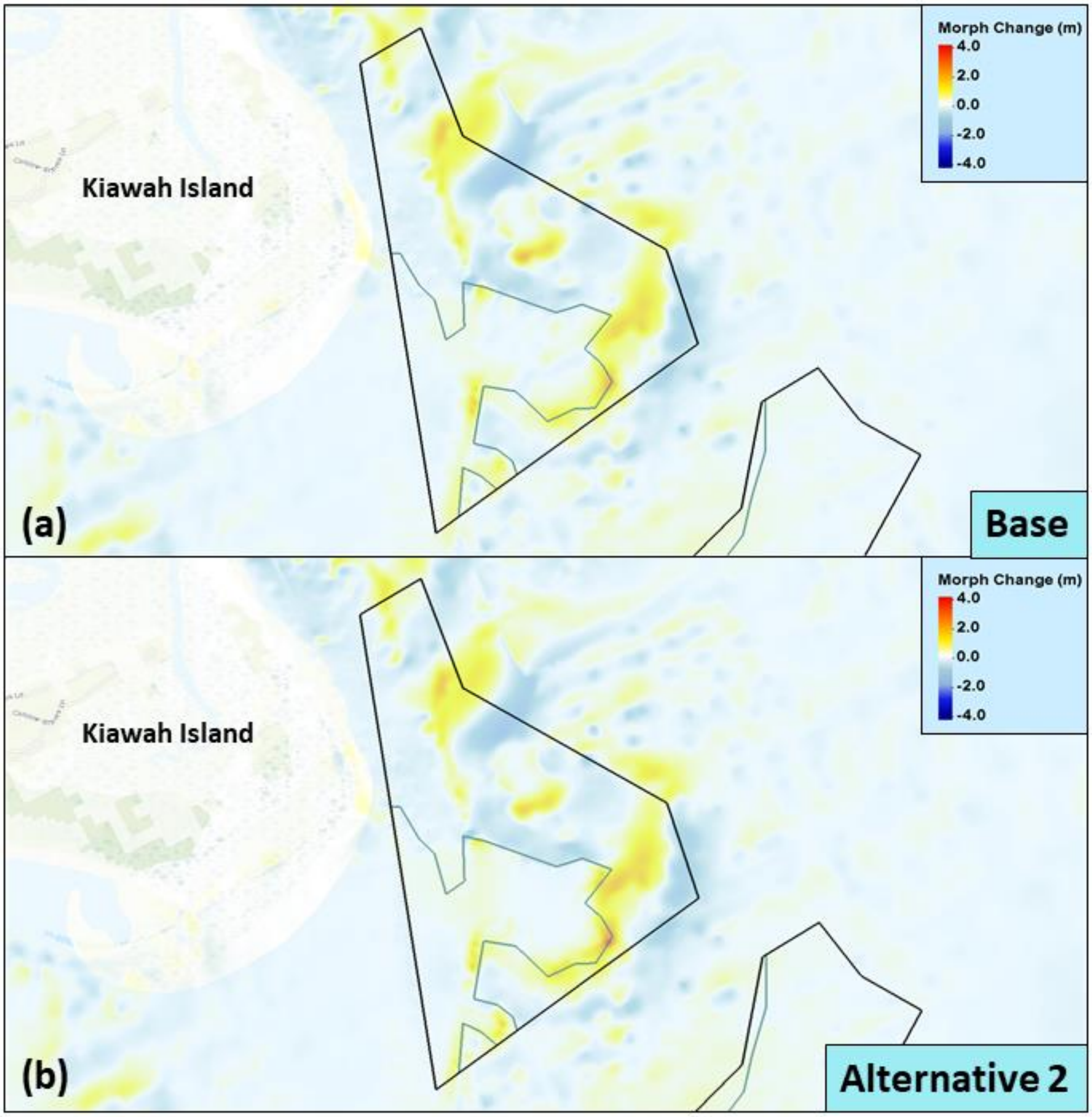

Figure 4-26. Comparison of morphology changes between (a) base case and (b) Alternative 2 within the Stono Inlet Throat borrow area during the Hurricane Hugo Passage from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

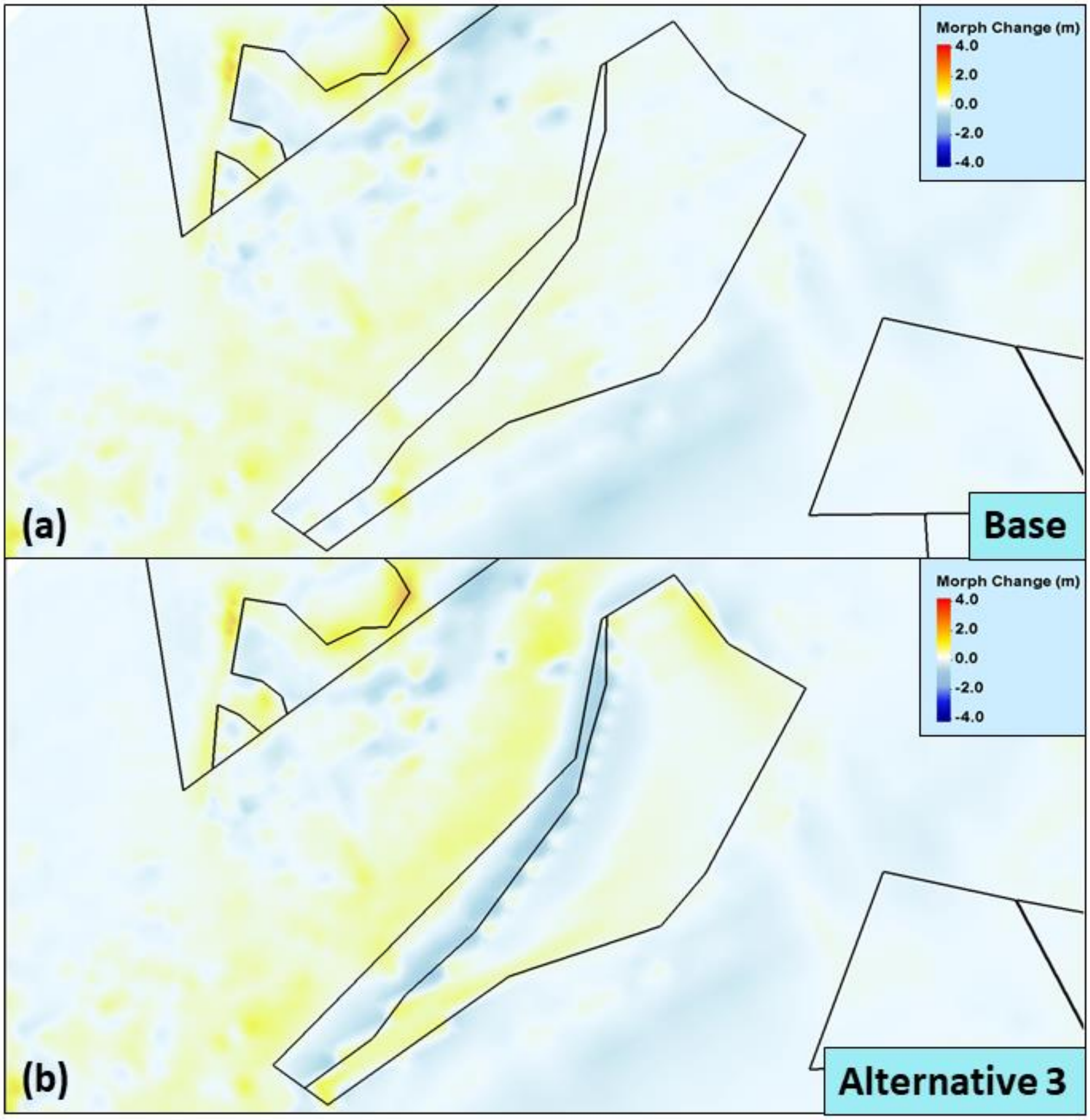

Figure 4-27. Comparison of morphology changes between (a) base case and (b) Alternative 3 within the Stono Ebb Shoal 1 borrow area during the Hurricane Hugo Passage from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

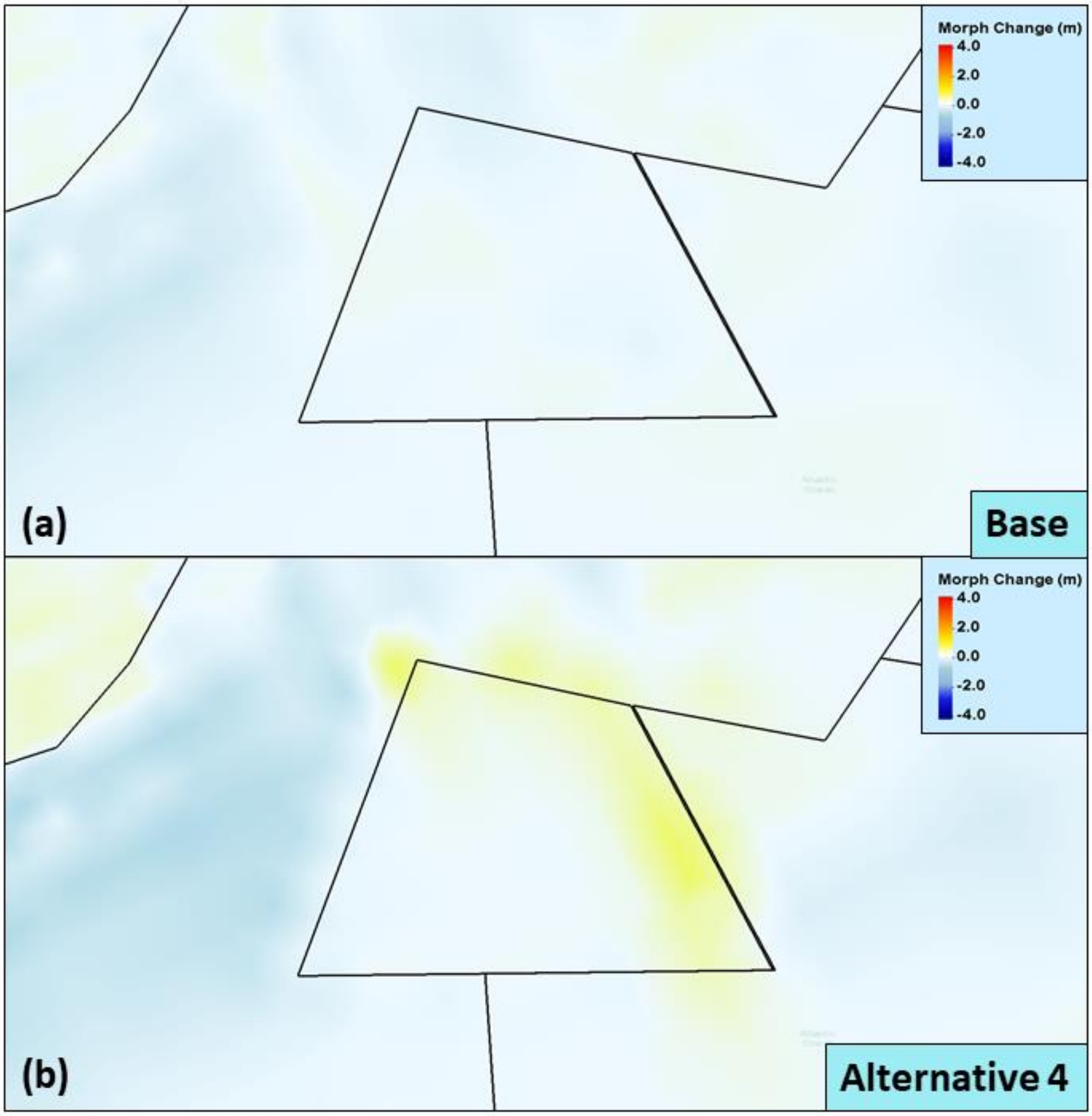

Figure 4-28. Comparison of morphology changes between (a) base case and (b) Alternative 4 within the Stono Ebb Shoal 2 borrow area during the Hurricane Hugo Passage from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

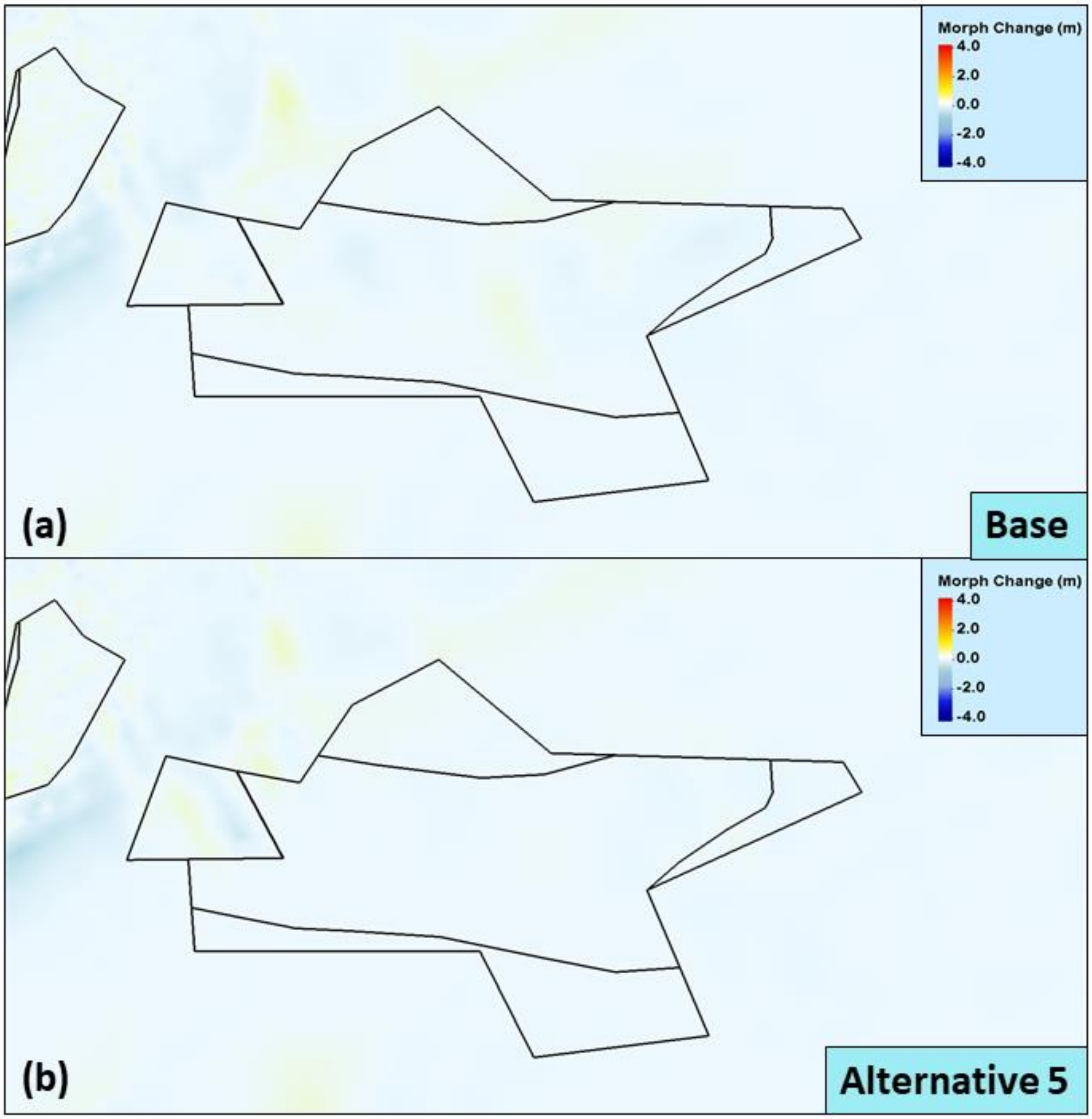

Figure 4-29. Comparison of morphology changes between (a) base case and (b) Alternative 5 within the Stono Inlet borrow area during the Hurricane Hugo Passage from 18 to 25 September 1989. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

Table 4-7. Comparisons of depth changes between the base case (before sand dredge) and alternatives (after sand dredge) in borrow areas during Hurricane Hugo.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Erosion (m)	-0.7	-1.2	-1.3	-1.4	-0.2	-1.2	-0.05	-0.05	-0.05	-0.04

Deposition (m)	1.5	3.5	1.4	2.5	0.7	0.7	0.06	0.6	0.12	0.14

The base case in the Folly River borrow area shows that large depth changes mostly occurred in the area impacted by flows turning around Folly Island. The alternative case in the actual dredged area only shows large deposition at the edge of the borrow area and no significant erosion inside the borrow area because of the decrease of currents due to sand removal (Figure 4-25). For the base case, the maximum erosion and deposition are 0.7 and 1.5 m, and for the alternative case, 1.2 and 3.5 m, respectively (Table 4-7). In the central and northern portion of the borrow area, sand movement is insignificant and depth changes for both cases are between 0.02 to 0.1 m.

In the Stono Inlet Throat borrow area, the large erosion and deposition in the range of 1.0 to 1.5 m occurred in the undredged shoaling and the east part of the borrow area. Sediment deposition can be seen but not much erosion occurred in the actual dredged area (Figure 4-26 and Table 4-7), where the maximum deposition is larger than 1.0 m and the maximum erosion is approximately 0.1 m both for the base and the alternative cases.

Figure 4-27 shows noticeable difference in erosion and deposition patterns between the base and the alternative cases in the Stono Ebb Shoal 1 borrow area. For the base case sediment deposition occurred in the area and erosion happened outside the borrow area. For the alternative case sediment materials were eroded between the actual dredged and undredged portions of the borrow area and relatively larger amount of sand was deposited in the dredged portion. Table 4-7 lists that the maximum erosion in the borrow area is only 0.19 m for the base case but has a much larger value of 1.2 m for the alternative case. The deposition values for both cases are similar, but the deposition area for the alternative case is much larger, implying more materials were moving into the dredged portion of the borrow area.

Examining the Stono Ebb Shoal 2 borrow area, it can be seen that depth changes are minimal for the base case (Figure 4-28). The erosion and deposition values shown in Table 4-7 are approximately 0.05 m. However, the deposition is larger by an order of magnitude for the alternative case comparing to the base case. The spatial contours of the morphology change indicate that sand materials were eroded from the outside of the borrow area.

Located in deep offshore area, the currents in the Stono Inlet borrow area are weak and slight morphology changes are observed in Figure 4-29. As shown in Table 4-7, the maximum erosion in the area is around 0.05 m and the maximum deposition is a little above 0.1 m. The erosion and deposition pattern does not change much before and after sand removal.

Associated with sediment transport and morphology change, volume changes and sediment budget are analyzed to further assess the storm impact on the borrow areas and nearshore Folly Beach area. Comparisons of bed volume changes between the base and alternative cases in the designated and actual borrow areas during the passage of Hurricane Hugo are listed in Table 4-8.

Table 4-8. Comparisons of bed volume changes (cu yd) between the base case (before sand dredge) and alternatives (after sand dredge) in designated and actual borrow areas during Hurricane Hugo. The negative sign indicates the volume loss and the positive the volume gain.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Designated Area	32780	62413	190362	218344	144840	-73061	2672	123504	-3755	24484
Actual Area	47338	78545	111602	135880	101683	132058	2672	123504	-32399	-2378

The calculations show that Hurricane Hugo caused net sediment accretion both for the base and the alternative cases in the Folly River borrow area. Comparing with the base case, the amount of accretion for the alternative case increases by 90% and 66% in the designated and the actual dredged areas, respectively. Comparing with the designated area, the amount of accretion in the actual dredged area increases by 44% and 26% for the base and the alternative cases, respectively (Table 4-8). The general trend of sediment accretion for different scenarios in different dredge areas indicates that (1) the Folly River borrow area is a sand trap zone, (2) sand removal in the borrow area results in more sediment infilling, and (3) more erosion occurs in the undredged portion of the borrow area.

The erosion and deposition values in Table 4-8 indicate that the Stono Inlet Throat borrow area is also located in a sand trap zone and has the largest sand accumulation among the other areas. Comparing with the base case, the accretion for the alternative case increases by 15% and 22% in the designated and the actual dredged areas, respectively. Comparing with the designated area, the amount of accretion in the actual dredged area is smaller for the base and the alternative cases. Considering much smaller actual dredged area within the designated area, the sediment accretion in this area is quite significant.

The Stono Ebb Shoal 1 borrow area is also partially dredged. Table 4-8 shows that net accretion occurs in the actual dredged area but net erosion occurs for the alternative case in the designated area. Also, comparing with the base case, the accretion for the alternative case increases by 30% in the actual dredged areas. Those values indicate that, associated with sand removal, large erosion happened in the undredged portion of the borrow area and large amount of eroded materials moved into the actual dredged area. The trend of volume changes in this area is consistent with morphology change as shown in Figure 4-28.

The Stono Ebb Shoal 2 borrow area has the same designated and actual dredged area. Due to sand dredge, materials deposited in the area increase by two orders of magnitude from the value in the base case (2.7k cu yd) to the alternative case (123.5k cu yd) (Table 4-8).

In the designated Stono Inlet borrow area, Table 4-8 shows net erosion for the base case but net accretion for the alternative case. Although both the base and alternative cases show net erosion in the actual dredged area, the amount of erosion for the alternative case is much smaller than that for the base case, meaning that sand removal still causes new deposition in this borrow area. It is also learned from the volume changes in Table 4-8 that net deposition occurs in the undredged portion of the borrow area.

In order to examine sediment transport and volume changes nearshore Folly Beach, eight polygons are drawn to embrace the eighteen transects in the nearshore zone (Figure 4-30). Along the shoreline from the southwest to northeast of Folly Island, the polygons are named from P1 through P8 and each polygon has an approximate longshore length of 1,000 m. Polygons P1 to P6 coincide with Reaches 1 to 21 and polygons P6 to P8 coincide with Reaches 22-26, which correspond to two different berm designs. Nearshore sediment budget and bed volume changes are analyzed around those polygons.

Figure 4-30. Eight polygons in the nearshore Folly Beach.

Total sediment transport across each polygon line is calculated, the transport directions are drawn, and sediment balance for each polygon is indicated by a positive (volume gain) or negative (volume loss) sign in Figure 4-31. Over all the polygon areas, the persistent longshore transport direction is towards southwest and the cross-shore transport direction is varying. Close to the southwest end of Folly Beach, polygon areas P1 and P2 have net sand gain and in the middle portion of the beach, polygon areas P3, P4, and P5 show net sand loss. Polygon areas P6 and P8 in the northeast part of the beach also show net sand gain.

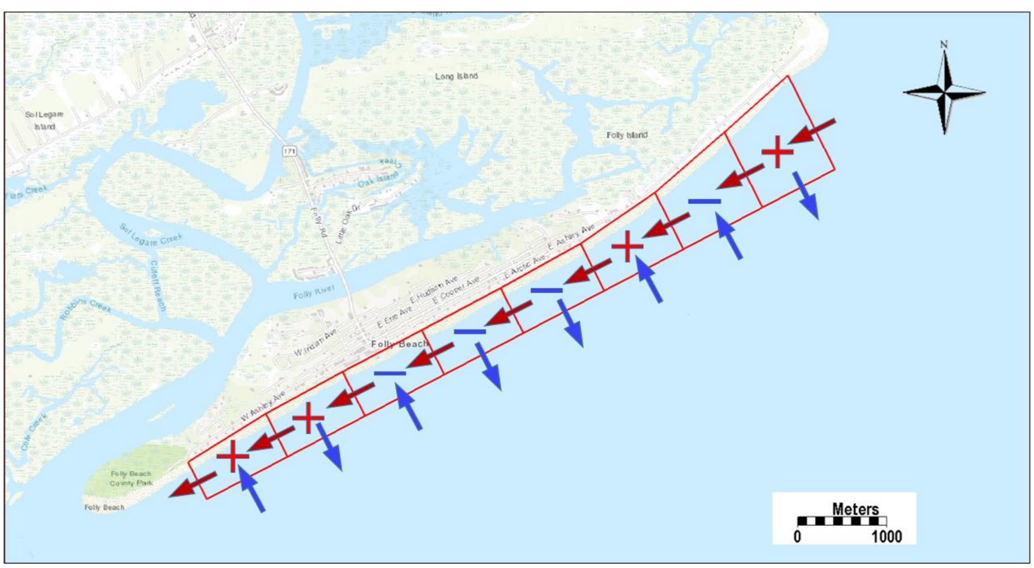

Figure 4-31. The calculated total sediment transport directions across polygon lines. The positive sign indicates bed volume gain and the negative the volume loss within the corresponding polygon area.

Based on the morphology changes, volume changes within each polygon are estimated and values are listed in Table 4-9.

Table 4-9. Bed volume changes (cu yd) for the base case (before sand dredge) and alternatives (after sand dredge) in polygon areas as shown in Figure 4-30 during Hurricane Hugo. The negative sign indicates the volume loss and the positive the volume gain.

Area	Alternative					
	Base	1	2	3	4	5
P1	7617	7655	7781	7942	11923	7690
P2	6119	6205	5894	6054	8412	6194
P3	-9181	-9082	-9094	-8952	-12638	-8969
P4	-2720	-2868	-3227	-2730	-10419	-2916
P5	-18741	-19100	-19058	-18795	-27630	-18938
P6	13575	13681	13763	13564	16066	13968
P7	-2745	-2617	-2443	-2472	-5428	-2514
P8	1002	852	725	928	56	540

For the base case, net sediment accumulation in the southwest and the northeast sections of Folly Beach (P1 and P2) is approximately 13,700 and 11,800 cu yd, respectively, after the hurricane passage. The middle section of the beach (P3 to P5) shows net sand loss of 30,600 cu yd. For the alternatives, the volume gain or loss within each polygon area is not off much from the base case because of the flow pattern in the study area and the distance between those sand borrow areas and the Folly Beach area.

Volume changes around Bird Key Island and Kiawah Island are examined within polygon areas shown in Figure 4-32. For Bird Key Island, the polygon area is surrounding the island. Three polygons are specified in the nearshore area of Kiawah Island, which are on the east, southeast (SE), and south sides of the island. Comparisons among the base and alternative cases are made and results shown in Table 4-10.

Figure 4-32. One polygon area surrounding Bird Key Island and three polygons on the east, southeast (SE), and south sides of Kiawah Island.

Table 4-10. Bed volume changes (cu yd) for the base case (before sand dredge) and alternatives 1, 2, and 3 (after sand dredge) in polygon areas around Bird Key Island and Kiawah Island during the passage of Hurricane Hugo in September 1989. The negative sign indicates the volume loss and the positive the volume gain.

Area		Alternative			
		Base	1	2	3
Bird Key Island		-8516	-7609	-8424	-8537
Kiawah Island	East	-20846	-20662	-21362	-20961
	SE	-24738	-25779	-26119	-24683
	South	-6850	-7088	-6919	-6985

Around Bird Key Island all the cases show material loss after the passage of Hurricane Hugo. Comparing the alternatives with the base case, dredging in the Folly River borrow area (Alternative 1) causes about 10% less material loss. For Alternatives 2 and 3, the volume changes are very close to that for the base case and the differences in volume changes among those cases are less than 1%.

The volume changes in Table 4-10 also show net sediment loss along the shoreline of Kiawah Island. Large material losses occur on the southeast and the east side. Comparing with the loss in those areas, only 25-30% of material loss occur on the south side. Among three alternatives, dredging in the Stono Inlet Throat borrow area has relatively large impact on sediment erosion in the nearshore area of the island. This alternative results in 6% more sediment volume loss on the southeast side of the island.

4.3 2018

Six selected locations, S1 to S6, within the actual dredged areas and near Folly Beach are shown in Figure 4-33. Time series of the calculated waves, current, and sediment transport are examined and compared between the base case and each of the alternative cases at those locations for the 2018 simulation. Morphology and volume changes in the borrow areas and nearshore in front of Folly Beach are obtained at the end of the simulation.

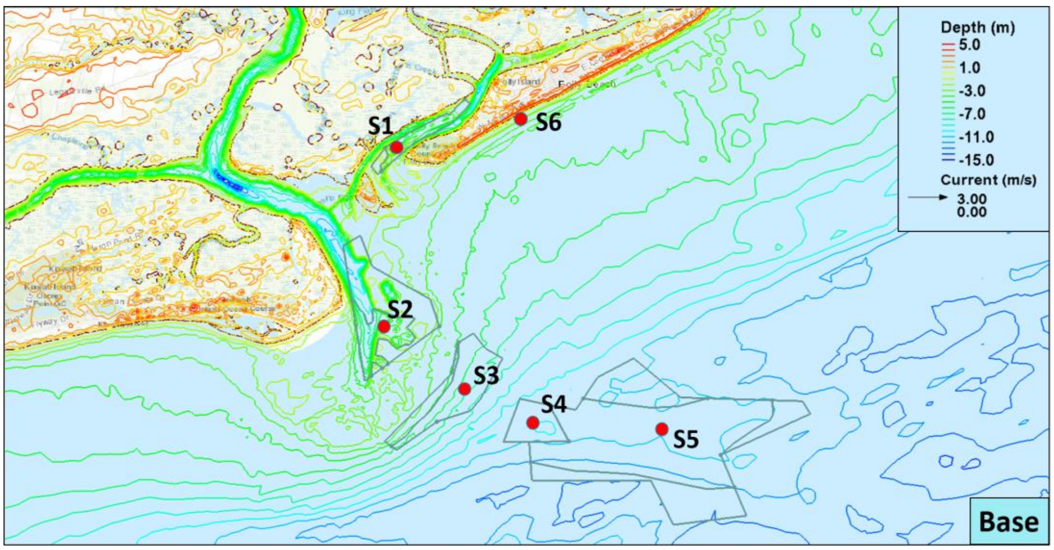

Figure 4-33. Selected locations within the five borrow areas and nearshore Folly Beach.

Waves

Figure 4-34 to Figure 4-39 show that several events occurred in 2018, mostly in January and December. Significant wave heights decreased from above 3 m in the offshore borrow areas (S4 and S5) to less than 1 m in the Folly River borrow areas (S1). In transitional areas significant wave heights decreased from around 3 m in the Stono Ebb Shoal 1 borrow area (S3) to 2 m in the Stono Inlet Throat borrow area (S2). The nearshore Folly Beach area (S6) is located in the break zone. In this area significant wave heights were less than 2 m.

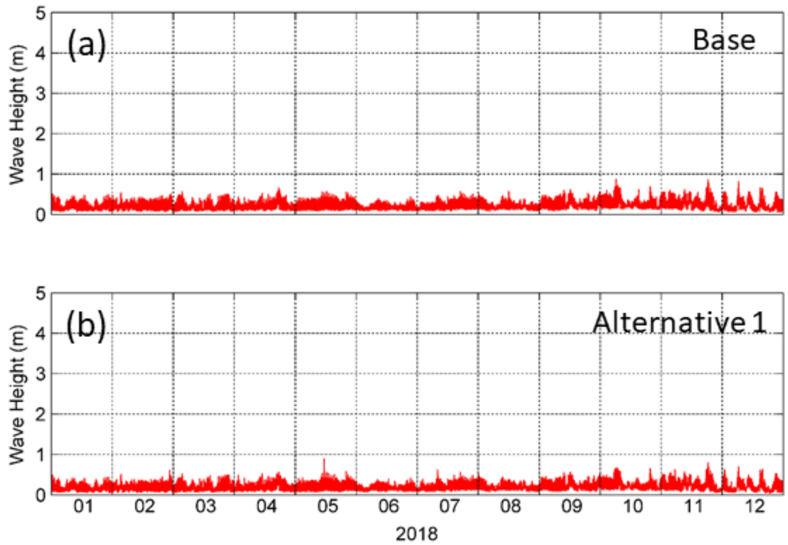

Figure 4-34. Comparison of significant wave heights between (a) base case and (b) Alternative 1 at S1 within the Folly River borrow area in 2018.

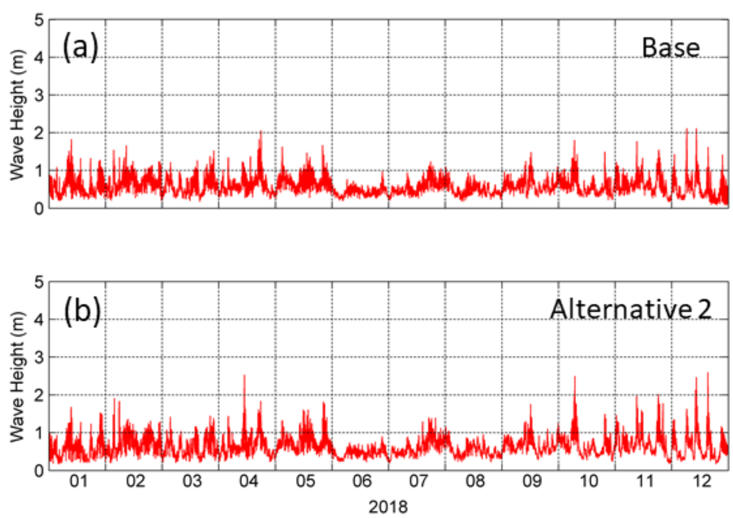

Figure 4-35. Comparison of significant wave heights between (a) base case and (b) Alternative 2 at S2 within the Stono Inlet Throat borrow area in 2018.

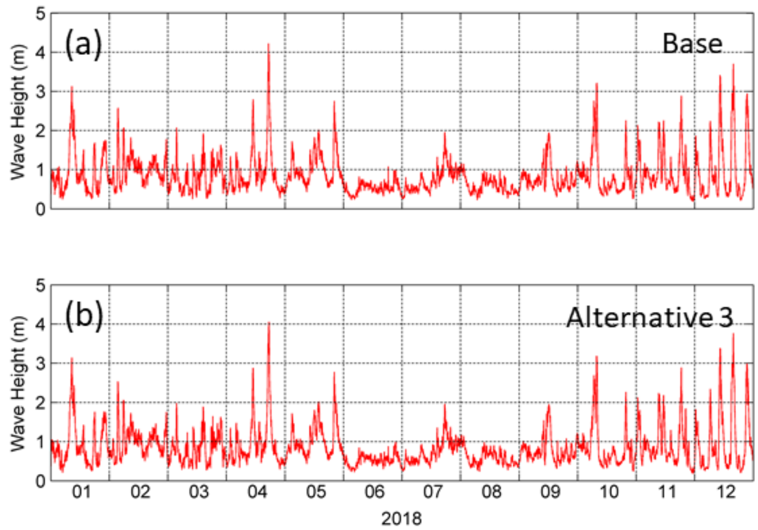

Figure 4-36. Comparison of significant wave heights between (a) base case and (b) Alternative 3 at S3 within the Stono Ebb Shoal 1 borrow area in 2018.

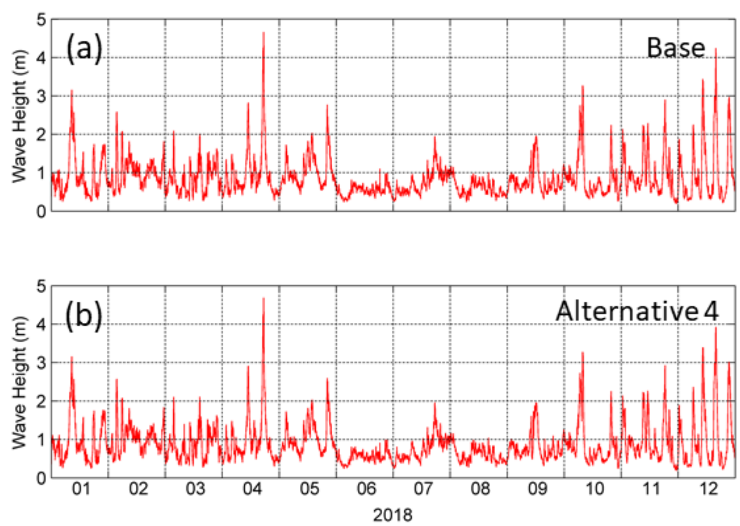

Figure 4-37. Comparison of significant wave heights between (a) base case and (b) Alternative 4 at S4 within the Stono Ebb Shoal 2 borrow area in 2018.

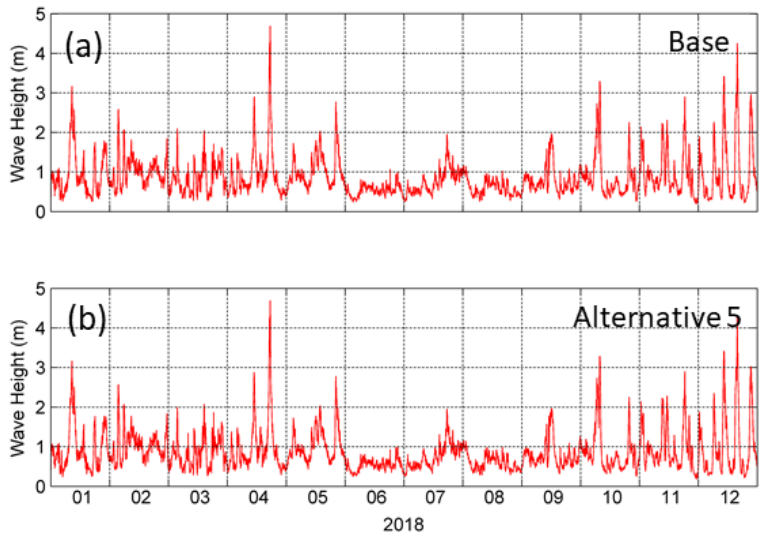

Figure 4-38. Comparison of significant wave heights between (a) base case and (b) Alternative 5 at S5 within the Stono Inlet borrow area in 2018.

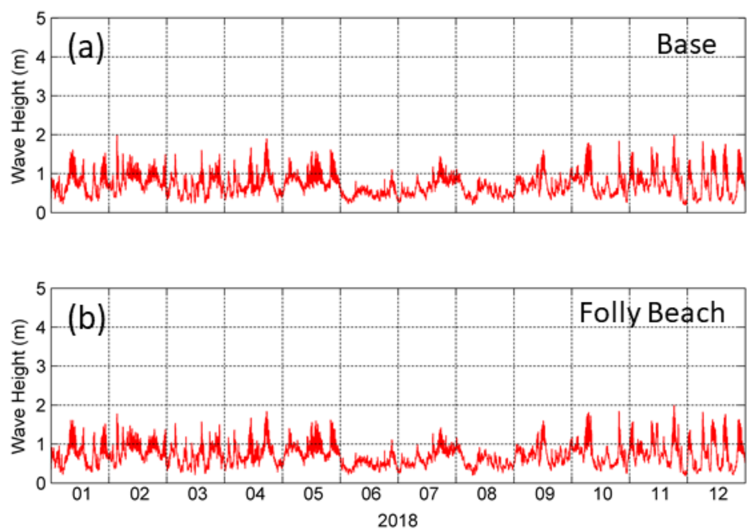

Figure 4-39. Comparison of significant wave heights between (a) base case and (b) Alternative 1 at S6 in the nearshore Folly Beach area in 2018.

Comparing the base case with the alternative cases, wave heights do not change much due to sand removal in the deeper borrow areas (S3 to S5). Because the original depth is small, sand removal in the Stono Inlet Throat area causes increase in peak wave heights. Figure 4-39 shows the comparison between the base case and Alternative 1 in the nearshore Folly Beach area at S6. Because no dredge activity happened at the location, wave heights between the two cases are very close over the year (Table 4-11).

Table 4-11. Annual averaged wave heights, current speeds, and sediment transport rates for the base and the alternative cases in the borrow areas and the nearshore Folly Beach area.

Area	Wave Height (m)		Current Speed (m/s)		Sediment Transport Rate (kg/(m.s))	
	Base	Alternative	Base	Alternative	Base	Alternative
Folly River (S1)	0.241	0.219	0.241	0.148	0.148	0.050
Stono Inlet Throat (S2)	0.591	0.637	0.239	0.254	0.219	0.336
Stono Ebb Shoal 1 (S3)	0.837	0.835	0.107	0.103	0.098	0.086
Stono Ebb Shoal 2 (S4)	0.844	0.853	0.077	0.068	0.064	0.058
Stono Inlet Throat (S5)	0.849	0.851	0.063	0.060	0.051	0.050
Folly Beach (S6)	0.721	0.720	0.102	0.101	0.094	0.094

Current

Figure 4-40 shows a snapshot of the depth-averaged flood and ebb current fields for the base case on 27 February 2018 at 22:00 and 28 February 2018 04:00 GMT, respectively. The maximum current speed is approximately 1.0-1.5 m/s in the Stono River channel. During both flood and ebb current the current speeds are relatively strong in the Folly River, Stono Inlet Throat, and Stono Ebb Shoal 1 borrow areas and weak in the Stono Ebb Shoal 2 and Stono Inlet borrow areas. The longshore current near Folly Beach shows flow direction from northeast to southwest periods although the current speed is much smaller during the ebb period.

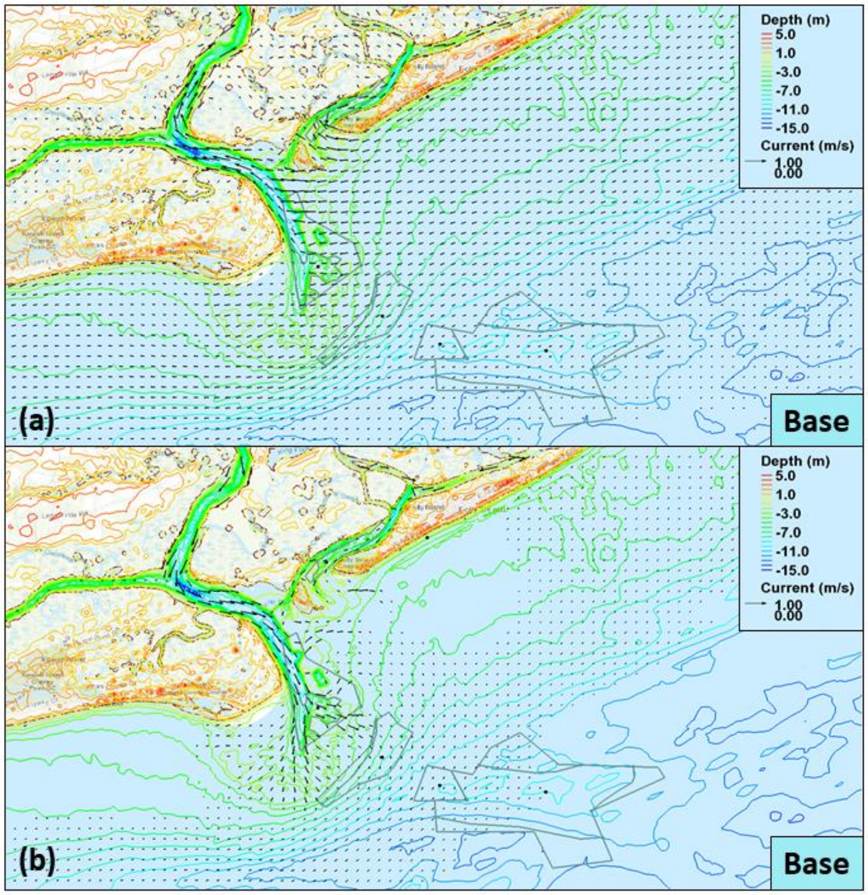

Figure 4-40. Calculated depth-averaged (a) flood and (b) ebb currents on 27 February 2018 at 22:00 and 28 February 2018 at 04:00 GMT, respectively.

Figure 4-41 to Figure 4-46, and Table 4-11 show that the nearshore Folly River (S1) and Stono Inlet Throat (S2) borrow areas have relatively strong currents with the peak speed above 0.8 m/s. In the offshore borrow areas (S4 and S5) the current speeds are generally less than 0.2 m/s. The nearshore Folly Beach area (S6) has shallow water depths and is greatly impacted by coastal processes and meteorological conditions. The peak current speeds there can be above 0.8 m/s during this annual simulation period.

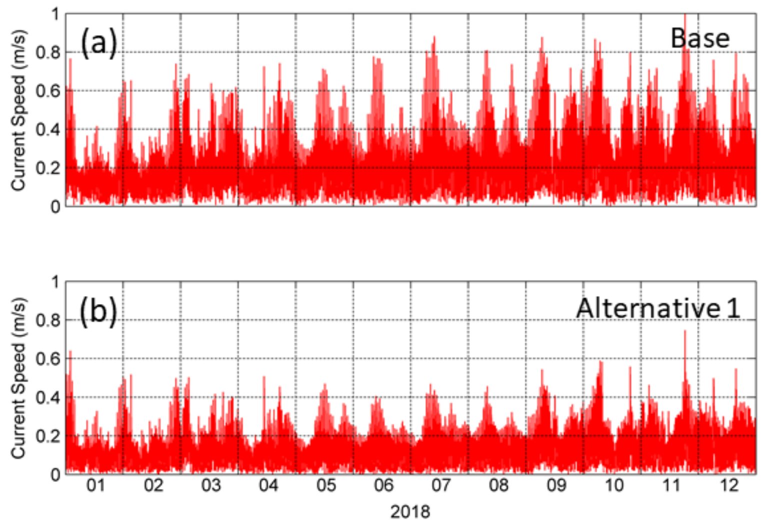

Figure 4-41. Comparison of current speeds between (a) base case and (b) Alternative 1 at S1 within the Folly River borrow area in 2018.

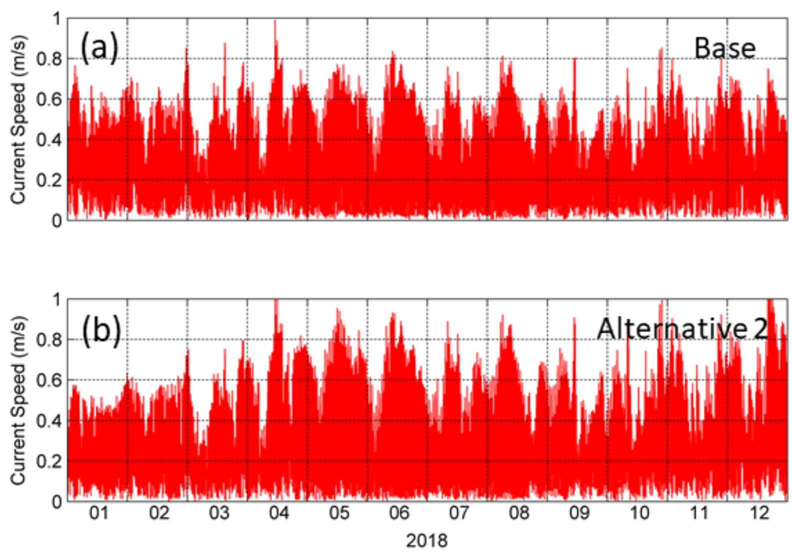

Figure 4-42. Comparison of current speeds between (a) base case and (b) Alternative 2 at S2 within the Stono Inlet Throat borrow area in 2018.

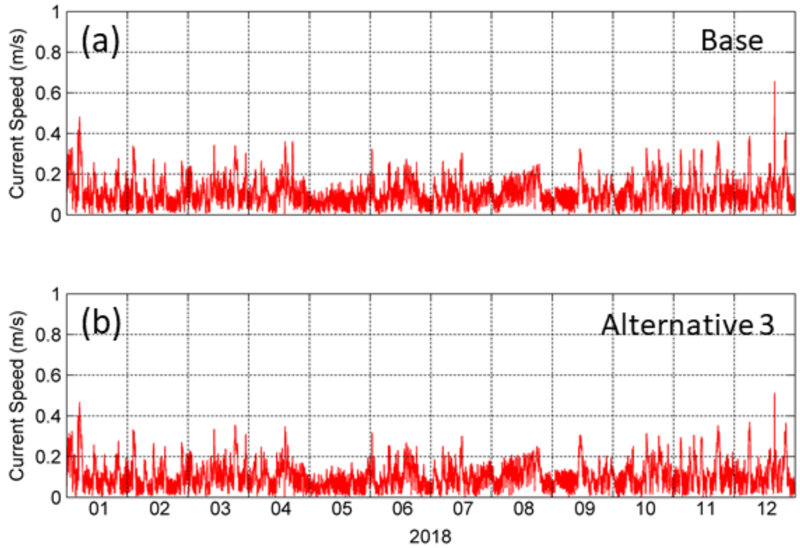

Figure 4-43. Comparison of current speeds between (a) base case and (b) Alternative 3 at S3 within the Stono Ebb Shoal 1 borrow area in 2018.

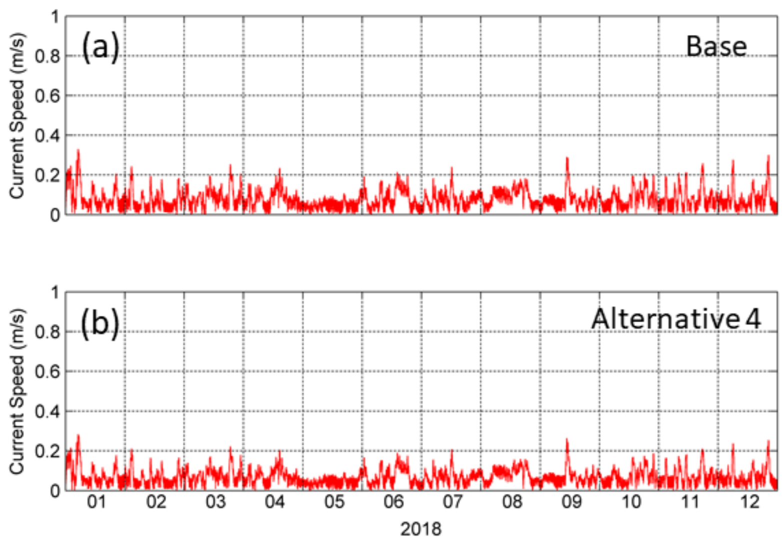

Figure 4-44 Comparison .of current speeds between (a) base case and (b) Alternative 4 at S4 within the Stono Ebb Shoal 2 borrow area in 2018.

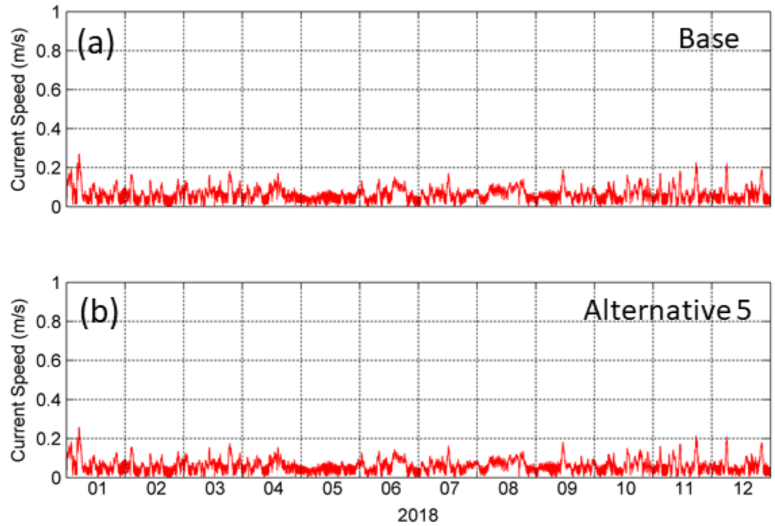

Figure 4-45. Comparison of current speeds between (a) base case and (b) Alternative 5 at S5 within the Stono Inlet borrow area in 2018.

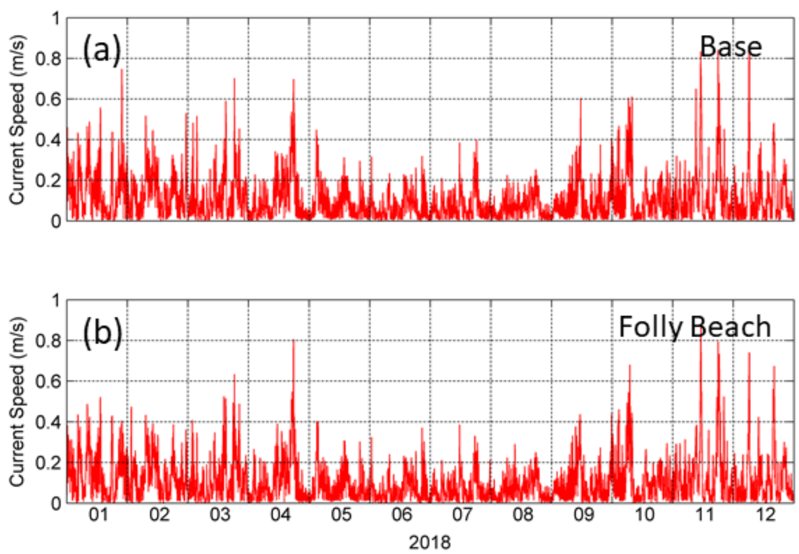

Figure 4-46 Comparison of current speeds between (a) base case and (b) Alternative 1 at S6 within the nearshore Folly Beach area in 2018.

Depth-averaged currents are closely associated with water depth changes. The Folly River borrow area was deepened from about an average depth of 2.5 m to 7.5 m and, therefore, the largest current speed changes can be seen in Figure 4-41. The offshore areas (S3 to S5) do not show significant changes in current speed because the original water depths are large and sand removal does not require large depth change due to the size of the borrow areas. Without sand borrowing in the nearshore Folly Beach area, Figure

4-46 shows the current speed changes through the year, which is probably due to depth changes related to sediment movement along the shore.

Sediment transport

Corresponding to wave and current forcing, the time series of sediment transport rates and annual averages are shown in Figure 4-47 to Figure 4-52, and Table 4-11. Similar to current variations in Figure 4-41 to Figure 4-46, the nearshore Folly River (S1), Stono Inlet Throat (S2) borrow areas, and Folly Beach area (S6) have relatively large sediment transport with the peak rate above 3 kg/(m·s). Comparing with those areas, the offshore sites in the Stono Ebb Shoal 2 (S4) and the Stono Inlet (S5) borrow areas show much smaller sediment transport rates, mostly less than 0.4 kg/(m·s).

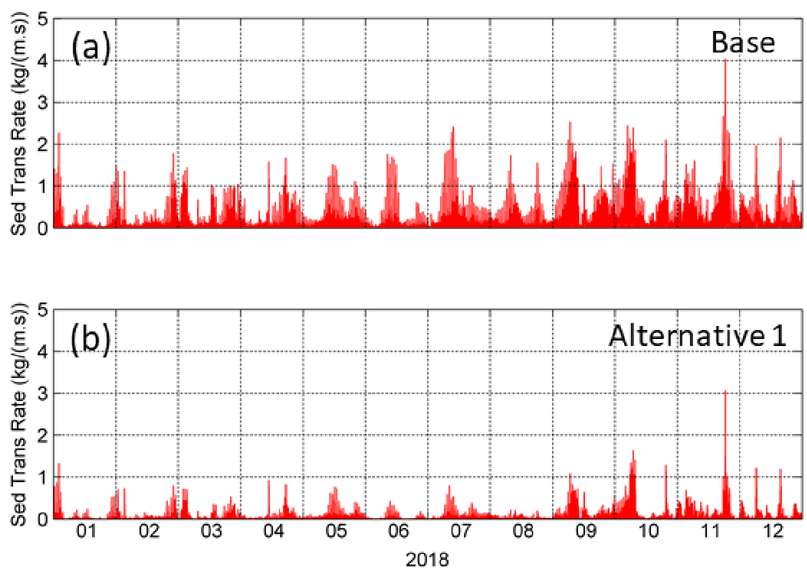

Figure 4-47. Comparison of sediment transport rates between (a) base case and (b) Alternative 1 at S1 within the Folly River borrow area in 2018.

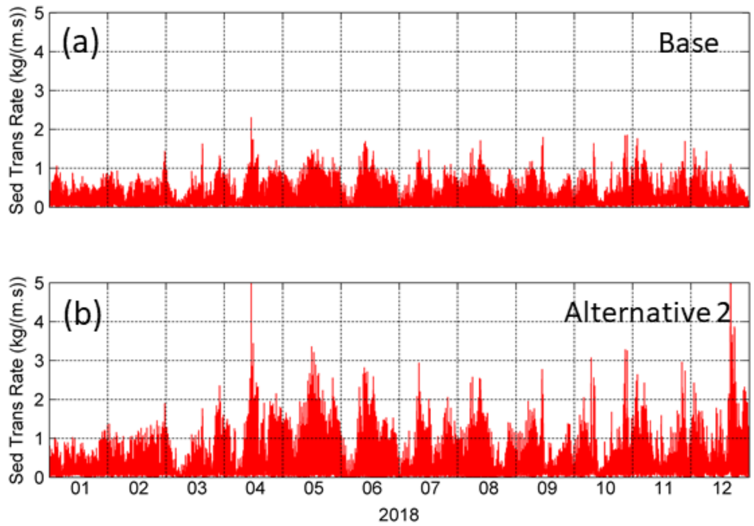

Figure 4-48. Comparison of sediment transport rates between (a) base case and (b) Alternative 2 at S2 within the Stono Inlet Throat borrow area in 2018.

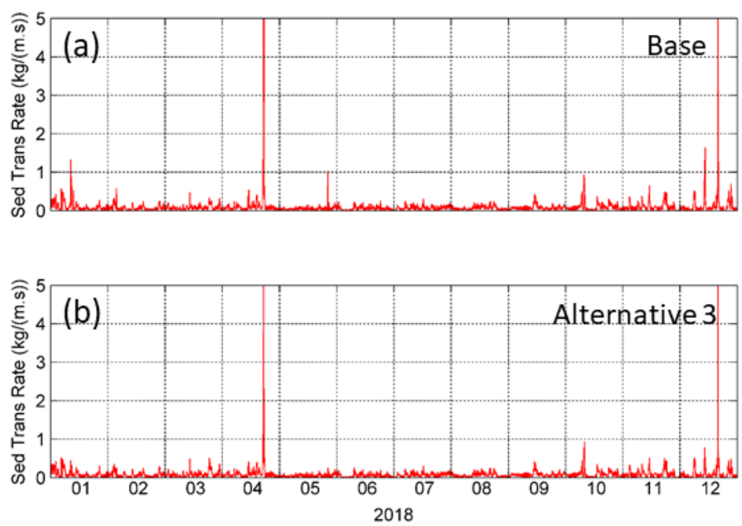

Figure 4-49. Comparison of sediment transport rates between (a) base case and (b) Alternative 3 at S3 within the Stono Ebb Shoal 1 borrow area in 2018.

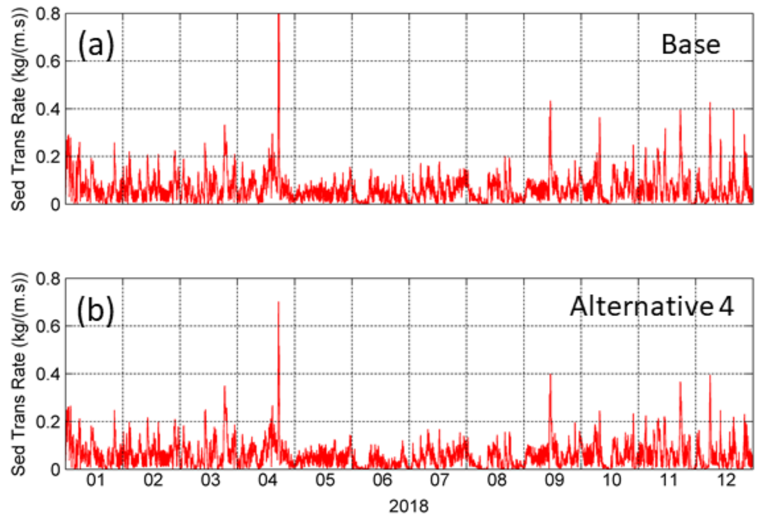

Figure 4-50. Comparison of sediment transport rates between (a) base case and (b) Alternative 4 at S4 within the Stono Ebb Shoal 2 borrow area in 2018.

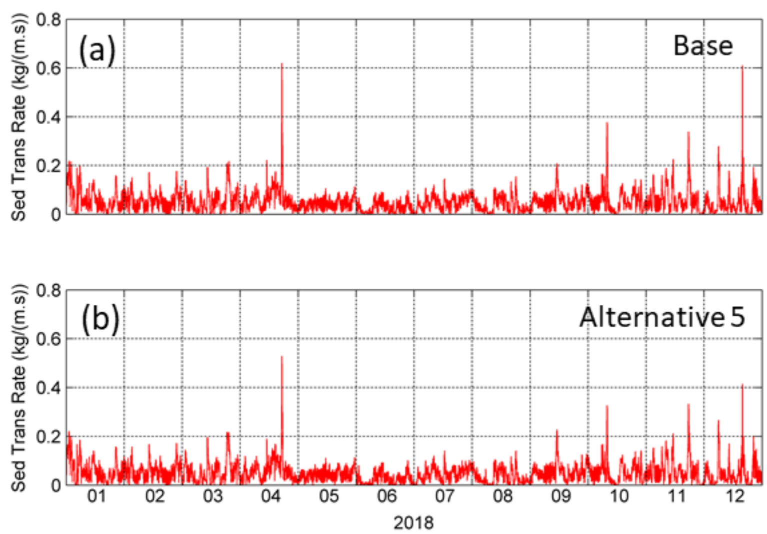

Figure 4-51. Comparison of sediment transport rates between (a) base case and (b) Alternative 5 at S5 within the Stono Inlet borrow area in 2018.

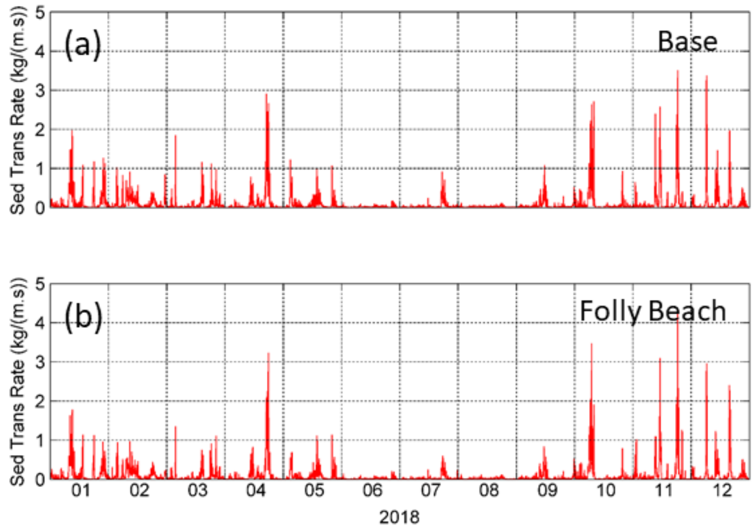

Figure 4-52. Comparison of sediment transport rates between (a) base case and (b) Alternative 1 at S6 in the nearshore Folly Beach area in 2018.

Current speeds and sediment transport rates have the greatest decrease in the Folly River borrow area (S1) (Figure 4-47). The offshore areas (S3 to S5) do not show significant changes in sediment transport. The annual averaged transport rates are all less than 0.1 kg/(m·s) both in the base and the alternative cases (Table 4-11). Without sand borrowing sediment transport changes in the nearshore Folly Beach area (S6) are responding to changes in water depth due to sediment erosion and deposition (Figure 4-52).

Morphology change

Annual morphology and volume changes within the five borrow areas and the nearshore Folly Beach area were calculated for the base and the alternative cases. For 2018, the changes were obtained by subtracting depth values at the end of simulation on 1 January 2019 at 00:00 from the initial depth values at the beginning of the simulation.

Comparisons of morphology changes before (base) and after (alternatives) sand removal from each of the borrow areas are shown in Figure 4-53 to Figure 4-57, respectively. The maximum erosion and deposition values within those areas are listed in Table 4-12.

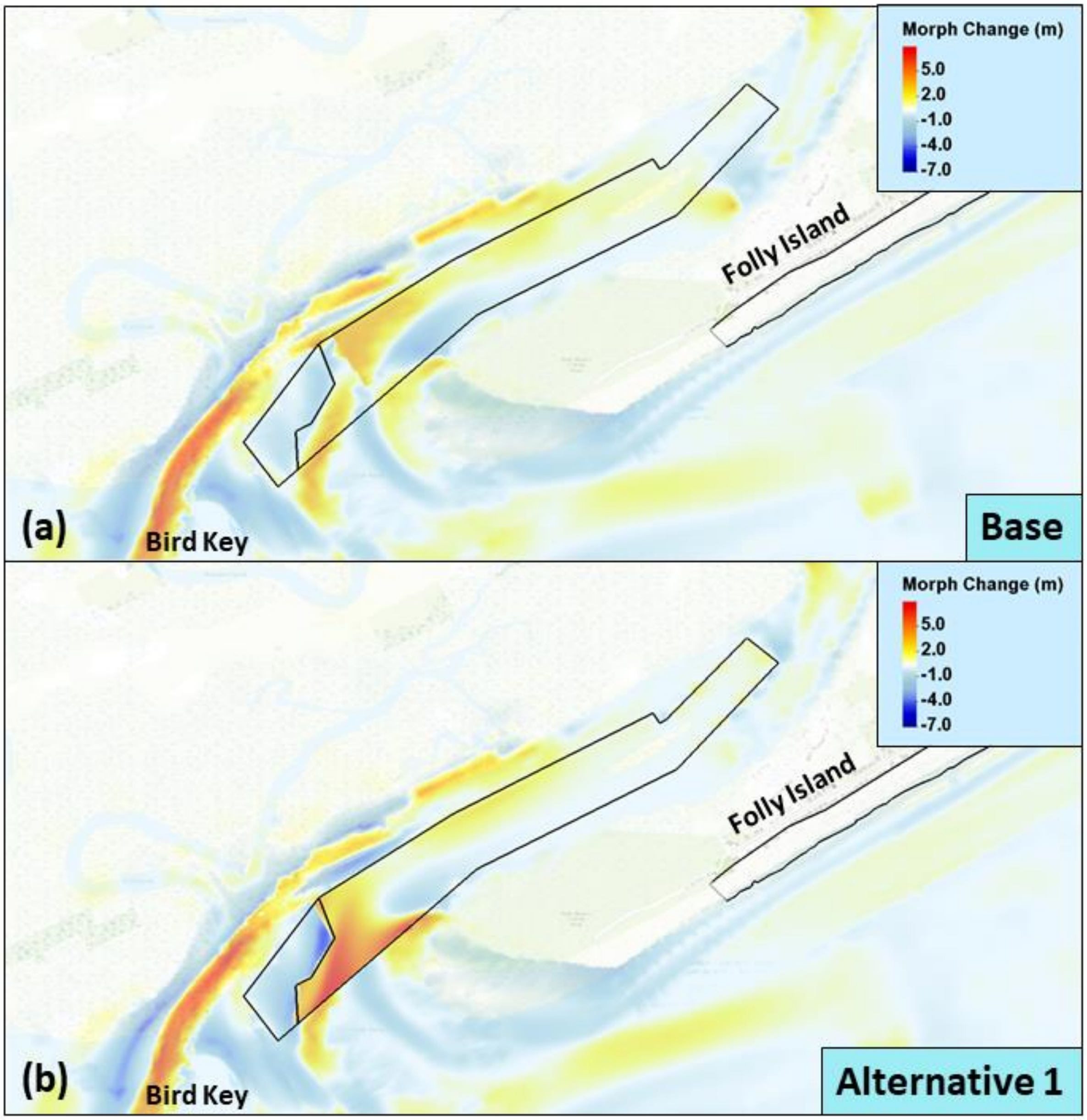

Figure 4-53. Comparison of morphology changes between (a) base case and (b) Alternative 1 within the Folly River borrow area in 2018. Warmer colors represent sediment accretion and cooler colors sediment erosion.

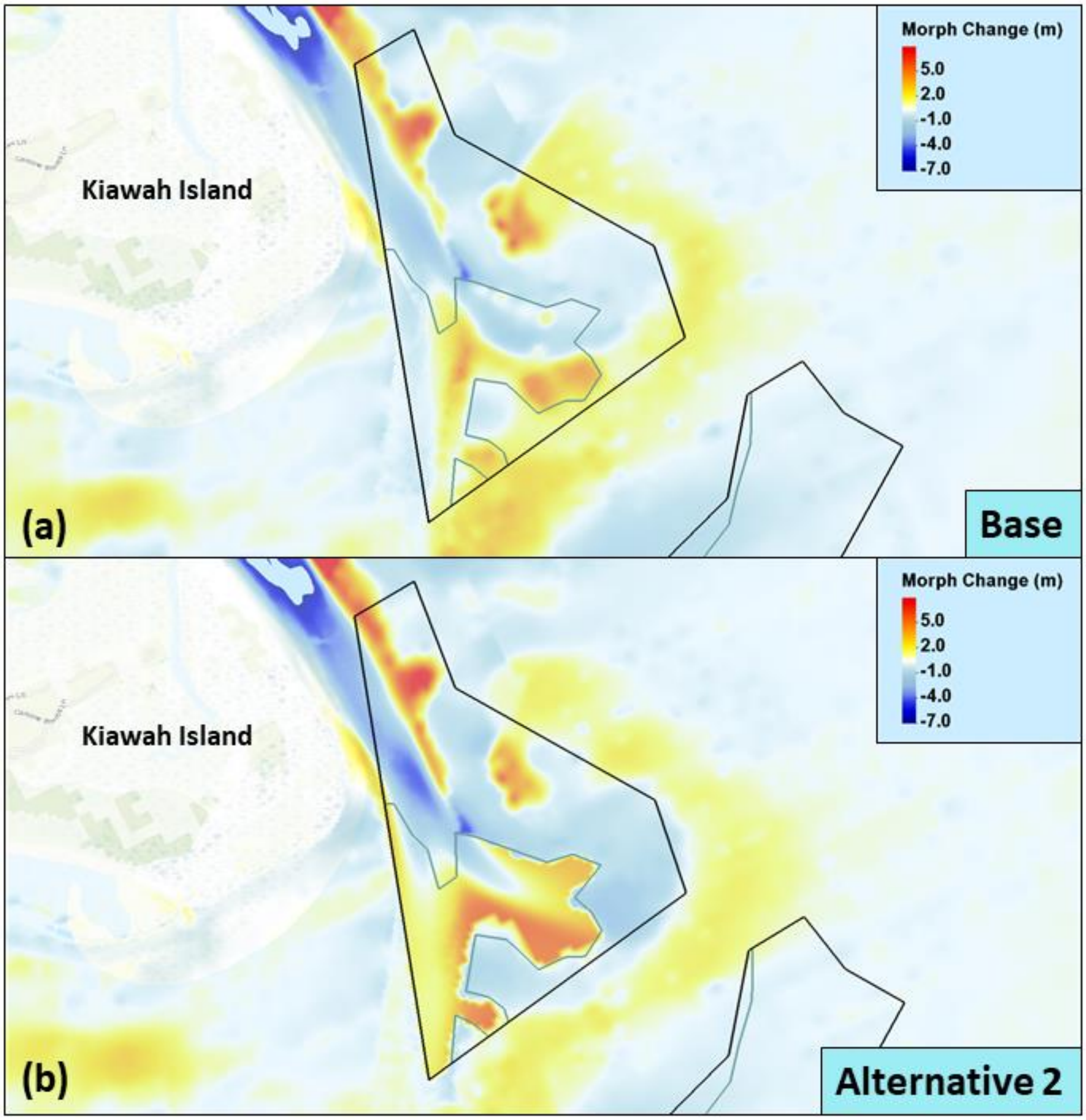

Figure 4-54. Comparison of morphology changes between (a) base case and (b) Alternative 2 within the Stono Inlet Throat borrow area in 2018. Warmer colors represent sediment accretion and cooler colors sediment erosion.

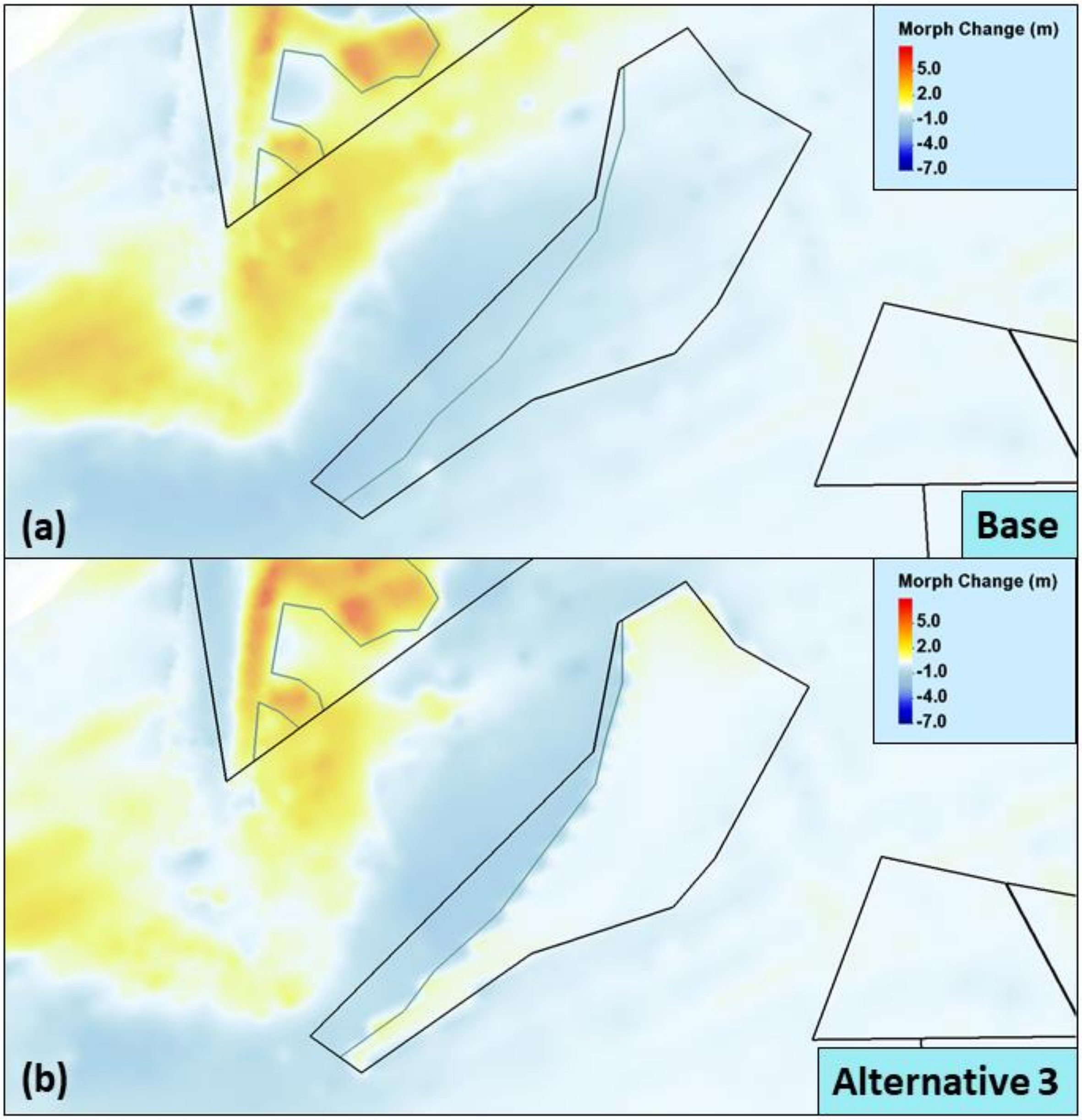

Figure 4-55. Comparison of morphology changes between (a) base case and (b) Alternative 3 within the Stono Ebb Shoal 1 borrow area in 2018. Warmer colors represent sediment accretion and cooler colors sediment erosion.

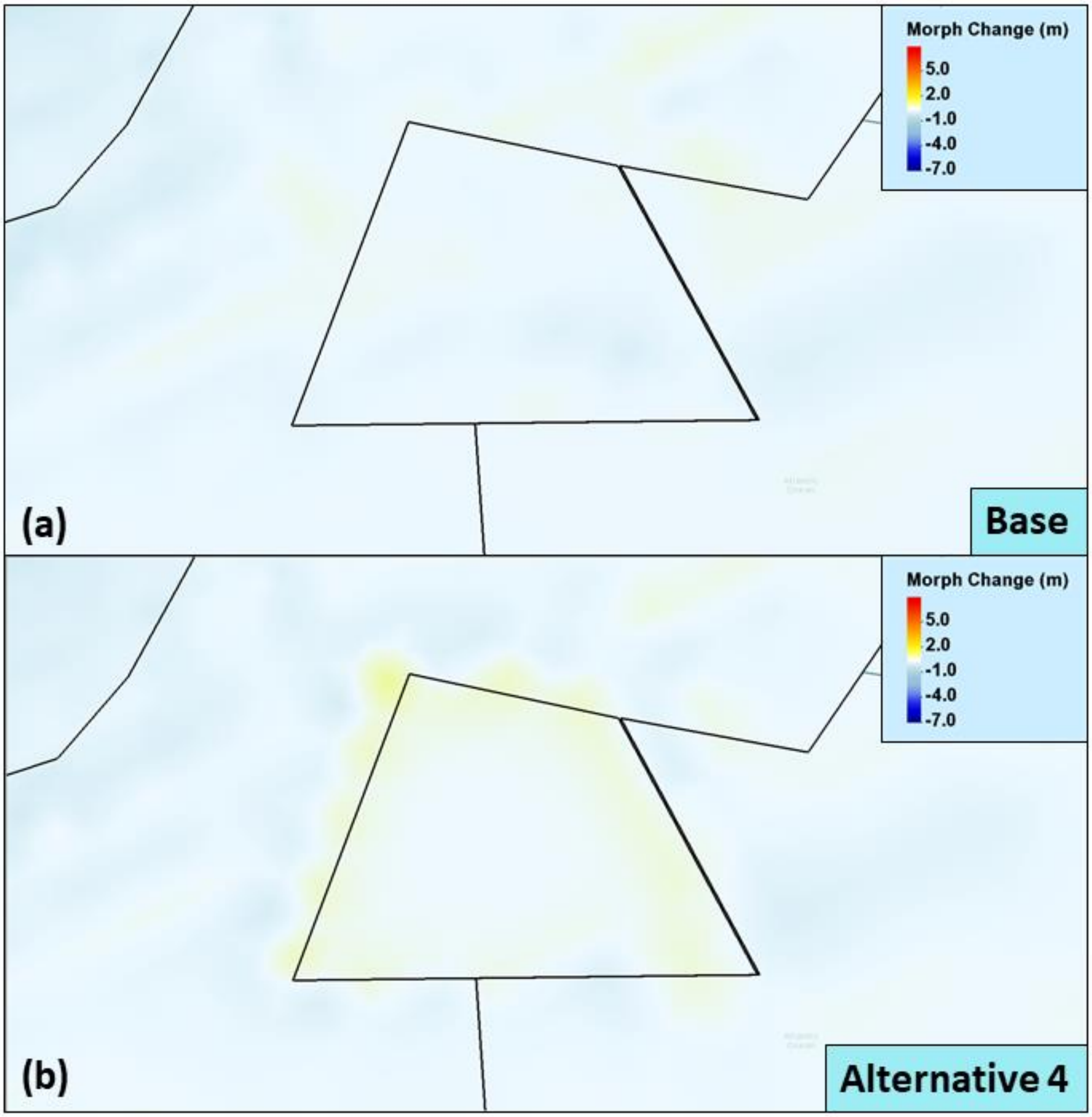

Figure 4-56. Comparison of morphology changes between (a) base case and (b) Alternative 4 within the Stono Ebb Shoal 2 borrow area in 2018. Warmer colors represent sediment accretion and cooler colors sediment erosion.

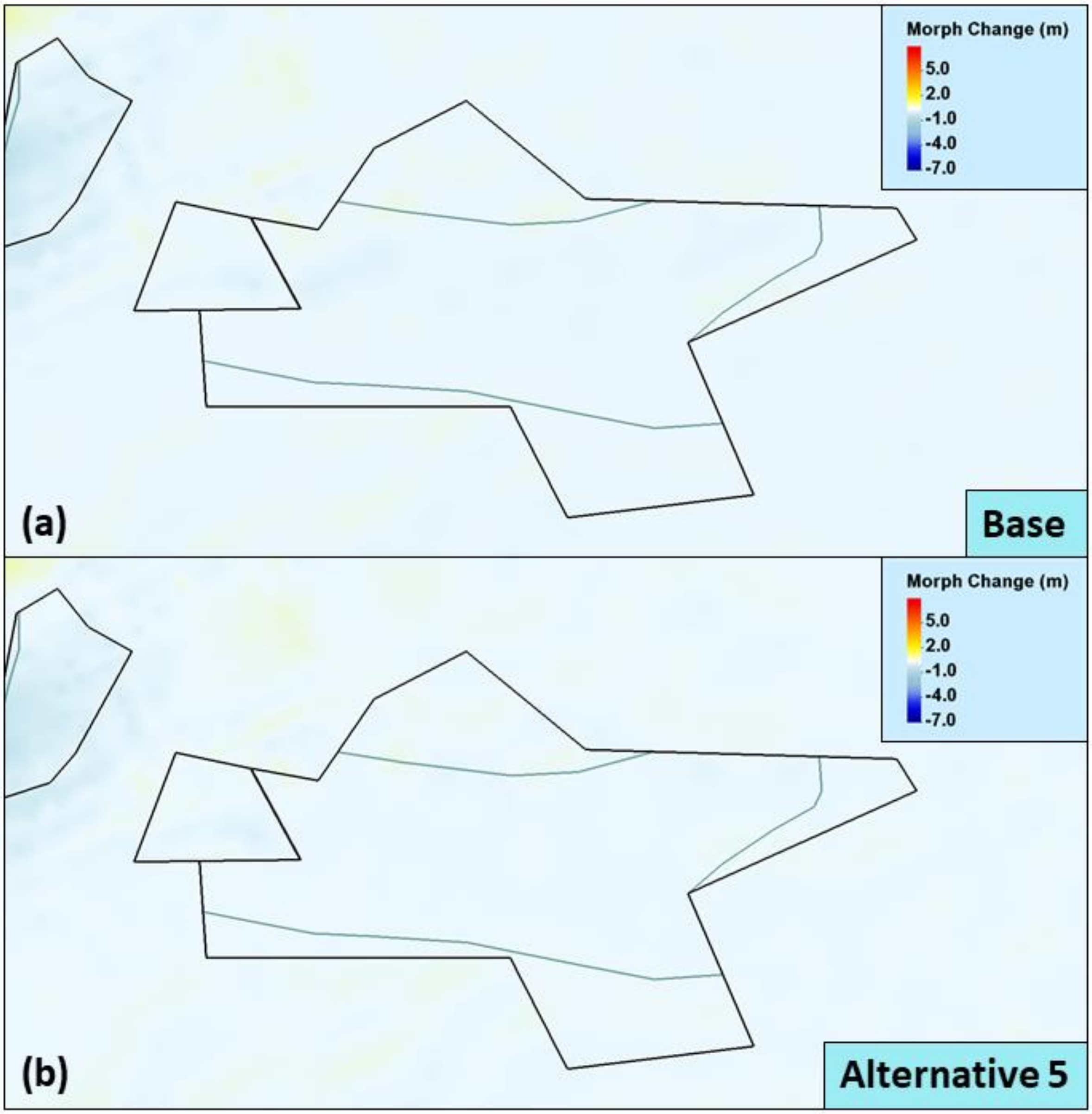

Figure 4-57. Comparison of morphology changes between (a) base case and (b) Alternative 5 within the Stono Inlet borrow area in 2018. Warmer colors represent sediment accretion and cooler colors sediment erosion.

Table 4-12. Comparisons of depth changes between the base case (before sand dredge) and alternatives (after sand dredge) in borrow areas in 2018.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Erosion (-) (m)	-3.2	-5.1	-5.0	-5.8	-1.9	-2.1	-0.4	-0.4	-0.4	-0.2

Deposition (+) (m)	3.3	6.8	6.2	7.1	0.1	0.9	0.1	0.3	0.3	0.2

The base case in the Folly River borrow area shows that sediment erosion and deposition mostly occurred in the southwest portion of the area, where water was shallow, and sediment was carried and moved by strong flows turning around southwest tip of Folly Island. The alternative case in the actual dredged area shows large deposition at the edge of the borrow area and no significant erosion inside the borrow area because of the decrease of currents due to sand removal (Figure 4-53). For the base case, the maximum erosion and deposition are up to 3.0 m, and for the alternative case, can be more than 5.0 m (Table 4-12). In the central and northern portion of the borrow area, sand movement is insignificant, and annual maximum erosion and deposition is less than 0.3 m.

For both the base and the alternative cases, the maximum erosion and deposition occurred in the undredged shoaling and the east part of the Stono Inlet Throat borrow area. In the dredged portion, although relatively small, comparable amount of sand was also moved around in the area. For the alternative case in the actual dredged area, sediment deposition was increased significantly (Figure 4-54 and Table 4-12). The maximum deposition is larger than 5.0 m and the maximum erosion is approximately 2.8 m.

Next to the shoaling area, the Stono Ebb Shoal 1 borrow area shows dominant erosional trend for both the base and the alternative cases (Figure 4-55). For the alternative case the eroded sand outside the actual dredged area was deposited inside the area. Table 4-12 lists that the maximum erosion in the borrow area is more than 1.0 m but the deposition is only around 0.1 m for the base case. For the alternative case, the maximum deposition values are increased to 0.9 m and the maximum erosion decreased to 0.1 m in the dredged area.

Similar to the erosion and deposition pattern in the Stono Ebb Shoal 1 borrow area, the Stono Ebb Shoal 2 borrow area also shows the dominant erosion for the base case and the increased deposition for the dredged case (Figure 4-56). As shown in Table 4-12 the erosion values are approximately 0.4 m for the base case and the deposition values increased from 0.1 to 0.34 m.

Located in deep offshore area, the currents in the Stono Inlet borrow area are weak and slight morphology changes are observed in Figure 4-57. As shown in Table 4-12, the maximum erosion in the area is around 0.37 m and the maximum deposition is about 0.25 m for the base case. Both the erosion and deposition values are a little smaller for the alternative case.

For the 2018 simulation, the four longshore erosion and deposition zones along Folly Beach as indicated in Figure 4-24 are all showing erosion in Figure 4-58. The average depths, depth changes, and volume changes during the year are listed in Table 4-13. Erosion Zone 1 goes along the shoreline all the way from the southwest to the northeast and is located in the nearshore area of Folly Island. This erosion zone has an average water depth change of 0.93 m and sand loss of 412,270 cu yd. Through the year Erosion Zone 2 at the southwest end of Folly Beach experiences a volume loss of 61,289 cu yd. In between these two zones is the original Deposition Zone 2, now the Erosion Zone 4 which shows an average depth change of 0.1 m, and a total sand loss of 10,202 cu yd. Erosion Zone 3 is the original Deposition Zone 1 extends from the north end of Erosion Zone 4 to the northeast of Folly Beach. The average depth change within this zone is close to 0.1 m and the loss of sand materials is 24,252 cu yd.

Figure 4-58. Morphology changes for the base case nearshore in front of Folly Beach in 2018. Warmer colors represent sediment accretion (delineated by red lines) and cooler colors sediment erosion (delineated by blue lines).

Table 4-13. Longshore erosion and deposition zones in front of Folly Beach during 2018. Positive depths are the land elevation above MSL.

Variables	Erosion Zone 1	Erosion Zone 2	Erosion Zone 3	Erosion Zone 4
Average Depth Change (m)	0.93	0.62	0.09	0.10
Average Depth (m)	-0.89	0.01	2.4	-1.43
Volume Change (cu yd)	412270	61289	24252	10202

Following the above analysis of sediment transport and morphology change, annual volume changes and sediment budget are examined in the borrow areas and nearshore Folly Beach area. Table 4-14 lists the comparisons of bed volume changes between the base and alternative cases in the designated and actual borrow areas in 2018.

Table 4-14. Comparisons of bed volume changes (cu yd) between the base case (before sand dredge) and alternatives (after sand dredge) in designated and actual borrow areas in 2018. The negative sign indicates the volume loss and the positive the volume gain.

Borrow Area	Folly River		Stono Inlet Throat (I)		Stono Ebb Shoal 1 (J)		Stono Ebb Shoal 2 (K)		Stono Inlet (E)	
Scenario	Base	Alt 1	Base	Alt 2	Base	Alt 3	Base	Alt 4	Base	Alt 5
Designated Area	214946	474928	1089153	849114	-2016446	-965885	-32588	146972	-162294	-141382
Actual Area	289642	645420	791135	2378339	-1150373	46760	-32588	146972	-259798	-209950

In the Folly River borrow area, the net sediment accretion occurred both for the base and the alternative cases. Comparing with the base case, the amount of accretion for the alternative case was more than doubled in the designated and the actual dredged areas, respectively. Comparing with the designated area, the amount of accretion in the actual dredged area increases by 35% and 36% for the base and the alternative cases, respectively (Table 4-14). The general trend of sediment accretion for different scenarios in different dredge areas indicates that (1) the Folly River borrow area is a sand trap zone, (2) sand removal in the borrow area results in more sediment infilling, and (3) more erosion occurs in the undredged portion of the borrow area.

The erosion and deposition values in Table 4-14 indicate that the Stono Inlet Throat borrow area is also located in a sand trap zone and has the largest sand accumulation among the other areas. Comparing with the base case, the accretion for the alternative case decreases by 22% in the designated dredged area and increases by a factor of three in the actual dredged areas. Comparing with the designated area, the amount of accretion in the actual dredged area is smaller for the base case and much larger for the alternative cases. Considering the much smaller actual dredged area within the designated area, the sediment accretion in this area is significant.

The Stono Ebb Shoal 1 borrow area is also partially dredged. Table 4-14 shows that net erosion occurs in the designated and actual dredged area, but smaller erosion and net accretion occur for the alternative case in the designated and actual dredged areas, respectively. Also, comparing with the base case, the alternative case changes the net erosion trend to the net accretion trend in the actual dredged area. Those values indicate that, associated with sand removal, large erosion happened in the undredged portion of the borrow area and a large amount of eroded materials moved into the actual dredged area. The trend of volume changes in this area is consistent with morphology change as shown in Figure 4-54.

The Stono Ebb Shoal 2 borrow area has the same designated and actual dredged area. Sand dredge changes the erosion and deposition pattern from erosion to deposition for the area. A large amount of material was removed from neighboring areas and deposited in the borrow area (Table 4-14).

In the designated Stono Inlet borrow area, Table 4-14 shows net erosion both for the base case and the alternative case. The amount of erosion for the alternative case is much smaller than that for the base case, meaning that sand removal still causes new deposition in this borrow area.

For the 2018 results, nearshore sediment budget and bed volume changes are analyzed around the polygons shown in Figure 4-30. Based on the total sediment transport across each polygon line, the transport directions are drawn, and sediment balance for each polygon is indicated by a positive (volume gain) or negative (volume loss) sign in Figure 4-59. Similar to the results by the hurricane impact, the 2018 results also show persistent longshore transport direction from the northeast to the southwest over all the polygon areas. Cross-shore transport direction is varying. The pattern of the net sand gain or loss within each polygon area is also the same as that from the storm simulation.

Figure 4-59. The calculated total sediment transport directions across polygon lines. The positive sign indicates bed volume gain and the negative the volume loss within the corresponding polygon area.

Based on the morphology changes, volume changes within each polygon are estimated and values are listed in Table 4-15.

Table 4-15. Bed volume changes (cu yd) for the base case (before sand dredge) and alternatives (after sand dredge) in polygon areas as shown in Figure 4-30 in 2018. The negative sign indicates the volume loss and the positive the volume gain.

Area	Alternative					
	Base	**1**	**2**	**3**	**4**	**5**
P1	-698	371	-493	-981	3044	-67
P2	12839	11847	13369	13131	11665	12364
P3	-65094	-64626	-65835	-65657	-62027	-65742
P4	-20576	-20959	-22014	-19710	-23561	-23078
P5	-27724	-27285	-28037	-26171	-24151	-26118
P6	6251	9217	3741	6675	14721	2263
P7	-23636	-23714	-14277	-27624	-20836	-31210
P8	19493	20376	19442	16460	23498	19120

For the base case, the net sediment loss in the southwest and the central sections of Folly Beach (P1 to P5) is approximately 101,253 cu yd in 2018. The northeast section of the beach (P6 to P8) shows small amount of sand gain of 2,108 cu yd. For the alternatives, the trend of the volume gain or loss within each polygon area is similar to the base case.

Corresponding to the changes in flow and sediment transport patterns related to the base and alternative cases, volume changes around Bird Key Island and Kiawah Island are examined for the 2018 simulation. The calculated results within the specified areas (Figure 4-32) are shown in Table 4-16.

Table 4-16. Bed volume changes (cu yd) for the base case (before sand dredge) and alternatives 1, 2, and 3 (after sand dredge) in polygon areas around Bird Key Island and Kiawah Island in 2018. The negative sign indicates the volume loss and the positive the volume gain.

Area		Alternative			
		Base	1	2	3
Bird Key Island		-100494	-99119	-98526	-101646
Kiawah Island	East	-622754	-583562	-565354	-574211
	SE	-283691	-327545	-313469	-319679
	South	-28076	-17436	-12695	-4981

Similar to the hurricane impact all the base and alternative cases show material loss around Bird Key Island and Kiawah Island. Within the polygon area surrounding Bird Key Island, the sediment volume changes due to the three alternatives are less than 2%.

The volume changes within three polygon areas along the shoreline of Kiawah Island show different responses to the dredges in the three borrow areas. The polygon area on the east side of the island shows the largest sediment loss during this one-year simulation. The three alternatives reduce the sediment loss by 6-9% within the area. On the southeast side of the island, the sediment loss is around 300k cu yd. Comparing with the base case the alternatives promote the sediment loss by more than 10% within this area. The alternatives also reduce the sediment loss within the polygon on the south side of the island but the total volume changes are insignificant comparing with the changes within the other two areas.

5 Conclusions

With the implementation of a field survey program, a coupled wave, hydrodynamic, sediment transport model, the Coastal Modeling System, was developed and applied to investigate sediment transport and morphology changes around the Stono Inlet estuarine system and adjacent Folly Beach, South Carolina. Field data collection included the deployment of two AWACs to measure waves, water level, and current within the Stono River channel and the open water in nearshore Folly Beach area. Driven by tide, waves, and wind, the CMS simulations include a 40-day calibration and validation, an 8-day storm simulation (Hurricane Hugo), and a one-year simulation (2018). For the storm and 2018 simulations, the base case and five alternative cases were configured, in which sand materials were dredged from the designated borrow areas and placed on Folly Beach for beach protection. By comparing the base case with each of the alternative cases, the model results were evaluated for sediment movement around the sand borrow and nearshore areas. From this modeling application, the major conclusions are drawn as follows:

1) Field data program is an integral component to the successful implementation of the numerical model and to the proper validation of the physical forces driving sediment transport in the coastal zone at the Stono Inlet estuarine system. Tidal flushing was captured through spatial and temporal field data collection of water levels and currents at the Stono River channel and the nearshore open ocean. Measured waves and currents provided a strong validation for numerical simulation of sediment transport.

2) The calibration of the CMS provided a close representation of physical forcing factors that drive sediment transport in the nearshore zone at the Stono Inlet system. Primary driving forcing in the areas are tide, wind and waves. Tidal currents are the dominant flow component in the estuary and around the inlet, and storm-/wave-driven currents are dominant in the open ocean area.

3) Among five sand borrow areas, relatively large backfilling occurs in nearshore areas, the Folly River and the Stono Inlet Throat areas. In offshore areas, sand removal does not have significant impact on sediment transport fields due to weak currents. Comparing the base

case with each of the alternative cases, sand dredge in a borrow area always induces more material accumulation.

4) Dredging a borrow area creates large depth gradients between dredged and undredged parts of the ocean, which tends to cause large current changes and more sediment deposition. Based on the model results, sediment supplies to the Folly River borrow area mainly come from the nearshore Folly Beach area, which are carried by the longshore current turning around the southwest tip of Folly Island. The actual dredged portion in the Stono Inlet Throat borrow area receives large amount of sediment from neighboring undredged shallow area.

5) In the nearshore Folly Beach area, model results show dominant longshore current and sediment transport directions from the northeast to the southwest. The net sand loss occurs in the southwest and the central sections and the net sand loss in the northeast section of Folly Island.

6) The Hurricane Hugo and the 2018 simulations show similar trend in morphology and volume changes, and erosion and deposition patterns around the sand borrow areas and in the nearshore Folly Beach area.

7) Both the base and the alternative cases of the Hurricane Hugo and the 2018 simulations indicate net sediment losses in the specified areas around Bird Key Island and Kiawah Island. During Hurricane Hugo dredging in the Folly River borrow area causes the largest sediment volume change, reducing the sediment loss by more than 10%, around Bird Key Island. Comparing the alternative with the base cases, the 2018 simulation shows significant increase and decrease of sediment loss on the southeast and the east side of Kiawah Island, respectively.

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